

UGL REGIONAL LINX



STRUCTURES ASSESSMENT

CRN-MAN-CVL-713026361-706

CRN CM 305

LINKING
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Table of Contents

Document Control.....	vi
Summary of changes made from previous version.....	vi
Chapter 1 General	1
C1-1 Purpose	1
C1-2 Context	1
C1-3 How to read this manual.....	1
C1-4 References	2
C1-4.1 Australian and International Standards	2
C1-4.2 CRN documents	2
C1-4.3 Other documents.....	2
C1-5 Definitions, abbreviations and acronyms	2
Chapter 2 Management requirements.....	4
Chapter 3 Competencies	5
Chapter 4 Structural appreciation	6
C4-1 Introduction	6
C4-2 Loads on structures	6
C4-2.1 Dead loads.....	6
C4-2.2 Live loads.....	6
C4-3 Structural behaviour	6
C4-3.1 Structure response to applied loads.....	6
C4-4 Railway loads for bridges	9
C4-5 Effects of dynamic loading	10
C4-6 Load rating.....	10
C4-6.1 Underbridges	10
C4-6.2 Overbridges	12
C4-6.3 Footbridges.....	13
C4-7 Structural Drawings	13
C4-8 Bearings.....	14
C4-8.1 Functions of bearings	14
C4-8.2 Types of bearings	15
C4-8.3 Bearing assessment issues.....	15
Chapter 5 Assessment of steel structures.....	16
C5-1 Deterioration modes in steel structures.....	16
C5-1.1 General	16
C5-1.2 Corrosion	16
C5-1.3 Impact damage	17
C5-1.4 Fatigue cracking	17
C5-1.5 Loose or missing connections	17

C5-2	Assessment of deterioration	17
C5-2.1	General	17
C5-2.2	Assessment procedures	17
C5-2.3	Other detection methods	18
C5-2.4	Steel repair issues	18
Chapter 6	Assessment of concrete structures	19
C6-1	Deterioration modes in concrete structures	19
C6-1.1	General	19
C6-1.2	Corrosion	19
C6-1.3	Other factors	19
C6-2	Deterioration of concrete	20
C6-2.1	Factors affecting deterioration	20
C6-2.2	Causes of deterioration	20
C6-3	Repairs of prestressed girders	23
C6-3.1	General	23
C6-3.2	Causes and Associated Symptoms.....	23
C6-4	Assessment of deterioration	24
C6-4.1	General	25
C6-4.2	Assessment procedures	25
C6-4.3	Other detection methods	27
C6-4.4	Concrete repair issues.....	27
Chapter 7	Assessment of masonry structures	29
C7-1	Deterioration modes in masonry structures	29
C7-1.1	General	29
C7-1.2	Causes of deterioration	29
C7-1.3	Cracking.....	29
C7-2	Causes of deterioration	30
C7-2.1	Ground movement.....	30
C7-2.2	Thermal movement.....	30
C7-2.3	Sulphate attack.....	30
C7-2.4	Expansion on wetting	30
C7-2.5	Corrosion of embedded iron or steel	31
C7-2.6	Unsound materials.....	31
C7-2.7	Salts.....	31
C7-2.8	Abrasion.....	31
C7-2.9	Impact force	31
C7-2.10	Overloading	32
C7-2.11	Vegetation and marine organisms.....	32
C7-3	Strength and stability	32
C7-3.1	Strength of arch bridges	32

C7-4	Assessment of deterioration	33
C7-4.1	General	33
C7-4.2	Assessment procedure	33
Chapter 8	Assessment of timber structures	35
C8-1	Deterioration modes in timber structures	35
C8-1.1	General	35
C8-2	Causes of deterioration	35
C8-2.1	Biological attack.....	35
C8-2.2	Non-biological deterioration.....	36
C8-3	Assessment of deterioration	36
C8-3.1	General	36
C8-3.2	Assessment procedures	36
C8-3.4	Timber repair issues	37
C8-3.5	Capacity reduction.....	37
Chapter 9	Assessment of timber poles.....	39
C9-1	Introduction	39
C9-2	Risk Assessment	39
C9-3	Final assessment of unsupported poles.....	40
C9-3.1	Introduction	40
C9-3.2	Assessment procedure	41
Chapter 10	Selecting repair actions	44
C10-1	Introduction	44
C10-2	Process for selecting repair action	44
C10-2.2	Is a repair to restore full strength necessary?	44
C10-2.3	Is a standard repair procedure available for each defect?	44
C10-2.4	What is the total cost of repairing all defects?.....	45
C10-2.5	How significant will the repair be in extending the life of the structure?	45
C10-2.6	Can the cost of the repair be justified on the basis of benefits derived?	45
C10-2.7	Is a detailed engineering assessment needed?	45
C10-2.8	Has partial replacement of the structure been considered?	45
C10-2.9	Will the repair have any adverse effects on the structure?	46
C10-3	Testing for and measuring defects	46
C10-3.1	General	46
C10-3.2	Measuring section loss in steel	46
C10-3.3	Testing for and measuring cracks in steel.....	46
C10-3.4	Observation under load	46
C10-3.5	Detecting defects in concrete	46
C10-4	Engineering assessments	47
C10-5	Avoiding recurrence of defects.....	47
C10-6	Steel repairs	48

C10-6.1	Steel repair issues	48
C10-6.1.1	Methods of connection	48
C10-6.2	Selecting the appropriate repair procedure	49
C10-6.3	Repair materials	49
C10-7	Repair materials for steel structures.....	50
C10-7.1	Steel.....	50
C10-7.2	Fasteners.....	50
C10-7.3	Epoxies for filling voids	50
C10-7.4	Epoxies for sealing interfaces	50
C10-7.5	Patch painting systems.....	50
C10-7.6	Paint systems for galvanised surfaces	51
C10-7.7	Bearing pad materials.....	51
C10-8	Repairing impact damage to steel structures.....	51
C10-8.1	Engineering discussion.....	51
C10-8.2	Sub-procedures required	51
C10-8.3	Inspection of damage	52
C10-8.4	Assessment of damage	52
C10-8.5	Selection of repair methods.....	55
C10-8.5.1	General	55
C10-9	Management of lead paint on steel structures	57
C10-9.1	Procedure	57
C10-9.2	Determination of the presence of lead paint	58
C10-9.3	Selection of painting strategy	58
C10-9.4	Selection of paint removal method	59
C10-10	Welding old steels	60
C10-10.1	Introduction	60
C10-10.2	Wrought and cast iron	61
C10-10.3	Welding wrought iron.....	61
C10-10.4	Mild steel.....	62
C10-10.5	Welding procedures for mild steel	62
C10-10.6	Structural steel.....	63
C10-10.7	Material identification.....	63
C10-10.8	Identification of wrought iron	63
C10-11	Concrete repair materials	67
C10-11.1	Introduction	67
C10-11.2	Material properties.....	67
C10-11.3	Types of materials for concrete repairs	68
C10-11.4	Questions to consider before choosing a repair material.....	77
C10-12	Repair options for concrete structures	79
C10-12.2	Repair options	79

C10-12.3	Selection of repair methods.....	79
C10-13	Repair materials for masonry structures	80
C10-13.1	General	80
C10-13.2	Function of mortar	80
C10-13.3	Problems with strong mortars.....	80
C10-13.4	Importance of lime in mortars	81
C10-13.5	Basic principles.....	81
Chapter 11	Certifying structures after maintenance and minor installation	82
C11-1	Introduction	82
C11-2	What to Certify	82
C11-3	How to Certify	82
C11-3.1	Planning.....	82
C11-3.2	Structures repair work	82
C11-3.3	Install transoms	83
C11-3.4	Install minor structures	83
C11-3.5	Install temporary track support	83
Chapter 12	Certifying structures after examination	85
C12-1	Introduction	85
C12-2	What to Assess.....	85
C12-3	When to Assess.....	85
C12-4	How to Assess	85
C12-4.1	General	85
C12-4.2	After detailed examination	85
C12-4.3	After special examination	86
C12-5	After underwater examination	86
C12-6	After monthly broad flange beam examination.....	87
C12-7	How to Certify	87

Document Control

Function	Position	Name	Date
Approver	A&E Manager	Lucio Favotto	24.01.2022

Revision	Issue Date	Revision Description
1.2	30.11.2021	UGLRL Operational Standards Template applied
2.0	14.12.2021	First approved and issued UGLRL version
3.0	24.01.2022	Issued for publish to intranet and webpage.

Summary of changes made from previous version

Section	Summary of change
All	This document is based on the previous rail infrastructure maintainer (RIM). Full revision history is available on request from UGLRL

Chapter 1 General

C1-1 Purpose

This Manual details the requirements for the assessment and certification of structures on the Country Regional Network (CRN).

Assessment includes:

- The evaluation of defects, and
- The selection of appropriate repair methods and repair materials for the rectification of defects.

Certification includes determination that a structure is safe for trains after

- Detailed structures examination and
- Maintenance work on the structure that affects or may affect track system integrity.

Relevant engineering background to structures repair procedures is included in the Manual to assist those responsible for selecting the appropriate repair action.

C1-2 Context

The manual is part of UGLRL CRN's engineering standards and procedures publications.

More specifically, it is part of the Civil Engineering suite that comprises standards, installation and maintenance manuals and specifications.

Manuals contain requirements, processes and guidelines for the management of track, structures, geotechnical and right of way assets and for carrying out examination, construction, installation and maintenance activities.

The manual is written for the persons undertaking installation and maintenance activities.

It also contains management requirements for the Civil Maintenance Engineer and Structures Superintendent needing to know what they are required to manage assessment and certification of structures on their area.

C1-3 How to read this manual

The best way to find information in the manual is to look at the Table of Contents starting on page 3. Ask yourself what job you are doing? The Table of Contents is written to reflect work activities.

When you read the information, you will not need to refer to CRN Engineering standards. Any requirements from standards have been included in the sections of the manual and shown like this:

The following design requirements are extracted from CRN Engineering Standard CRN CS 310 “Underbridges”			
	Track Class	Design Load configuration	
	Main Lines		
	Heavy Haul Coal Operations	350-LA plus DLA	

Reference is however made to other Manuals.

Throughout this manual reference is made to the following levels of Engineering Authority:

- Principal Track and Civil Engineer
- Civil Maintenance Engineer
- Area Manager
- Structures Superintendent

These are general descriptors only. For an explanation of the positions in the UGLRL CRN organisation that perform these functions, refer to CRN Engineering Manual CRN CM 001 “Civil Technical Competencies and Engineering Authority”.

C1-4 References

C1-4.1 Australian and International Standards

AS 1214	Hot-dip galvanized coatings on threaded fasteners (ISO metric coarse thread series)
AS 1252	High strength steel bolts with associated nuts and washers for structural engineering
AS 1580.501.1 - (withdrawn)	Paints and related materials – Methods of test – Soluble lead content
AS 1554.1	Structural steel welding – Welding of steel structures
AS 3678	Structural steel – Hot-rolled plates, floor plates and slabs
AS 3679.1	Structural steel – Hot-rolled bars and sections
AS 3679.2	Structural steel – Welded I sections
AS 5100	Bridge Design

Unless otherwise specified, all references relate to the latest standard versions, including amendments and relevant superseding standards.

C1-4.2 CRN documents

CRN CS 310	Underbridges
CRN CM 001	Civil Technical Competencies & Engineering Authority
CRN CM 101	Civil Service Schedules
CRN CM 211	Track Geometry & Stability
CRN CM 302	Structures Examination
CRN CM 303	Structures Repair

C1-4.3 Other documents

Department of Transport (UK) Departmental Advice Note BA 16/84	The Assessment of Highway Bridges and Structures”
Department of Transport (UK) Departmental Standard BD 21/84	The Assessment of Highway Bridges and Structures”
National Co operative Highway Research Program Report 271	Guideline for Evaluation and Repair of Damaged Steel Bridge Members
Welding Technology Institute of Australia (WTIA) Technical Note 11	Commentary on the structural steel welding Standard AS 1554

C1-5 Definitions, abbreviations and acronyms

Fracture Critical Member: Tension members or tension components of members whose failure would be expected to result in collapse of the bridge or inability of the bridge to perform its design function.

Tension	Force acting to stretch a structural member.
Compression	Force acting to compress a structural member.
Flexural Strength	Strength of a structural member in bending.

Shear Force

Force acting perpendicular to the axis of a structure component.

Alkali Aggregate Reaction

Reaction which occurs over time in concrete between the cement paste and aggregates. This reaction can cause expansion of the aggregate, leading to spalling and loss of strength of the concrete.

Chapter 2 Management requirements

The Area Manager is responsible for ensuring that the following tasks are carried out by persons with appropriate competencies and engineering authority:

- Determining repair actions for maintenance work on structures
- Certifying structures safe for trains after maintenance work on structures that affects or may affect track system integrity
- Certifying structures safe for trains after detailed structures examination

Chapter 3 Competencies

NOTE: These competencies may enable activities to be carried out in other manuals. For a comprehensive list of all activities that are covered by a given competency see CRN CM 001.

To carry out this work	You need these competencies
Maintain structures - selecting repair actions	TLIB2096 - Repair concrete/masonry structures TLIB2081 - Repair timber structures TLIB2082 - Repair steel structures AND Structures assessment (EA Level 3)
Certify structure safe for trains after repair work	TLIB2096 - Repair concrete/masonry structures TLIB2081 - Repair timber structures TLIB2082 - Repair steel structures AND Structures assessment (EA Level 3) Separate certification of track: TLIB3094 - Check and repair track geometry
Certify structure safe for trains after installation of transoms	TLIS2028 - Install and replace transoms AND Structures assessment (EA Level 3) Separate certification of track: TLIB3094 - Check and repair track geometry
Certify structure safe for trains after installation of minor structures	TLIS2013 - Install minor structures AND Structures assessment (EA Level 3) Separate certification of track: TLIB3094 - Check and repair track geometry
Certify structure safe for trains after installation of temporary track supports	TLIS2033- Install and repair temporary track supports AND Structures assessment (EA Level 3) Separate certification of track: TLIB3094 - Check and repair track geometry
Certify structure safe for trains after detailed examination of structures	Detailed examination of structures competencies AND Structures assessment (EA Level 3)
Certify structure safe for trains after special examination of structures	Detailed examination of structures competencies AND Structures assessment (EA Level 3) Separate certification of track: TLIB3094 - Check and repair track geometry

Chapter 4 Structural appreciation

C4-1 Introduction

This section provides information about the basic loadings that a structure is subjected to and how the structure responds as load is applied.

It does not provide any in-depth discussion of design loadings or structural analysis.

C4-2 Loads on structures

C4-2.1 Dead loads

Dead loads are permanent loads on a structure. Dead loads include the weight of the following:

- The structure components
- Permanent attachments such as the track, service pipes, protection screens, traffic barriers and, walkways

C4-2.2 Live loads

Live loads are temporary loads imposed on a structure. Live loads include:

- Rail vehicles on an underbridge
- Road vehicles on an overbridge
- Pedestrians on an overbridge or footbridge
- Dynamic load: the effects of moving trains or road vehicles
- Centrifugal forces
- Earth pressure
- Wind
- Flood
- Braking
- Ground movement (subsidence)
- Earthquake
- Collision
- Thermal effects: the effects of heating and cooling.
- Time dependent effects (creep & shrinkage etc.)

C4-3 Structural behaviour

C4-3.1 Structure response to applied loads

When structures are subjected to loads, the structure components respond in a number of ways to the applied loads.

The main structural responses to loads are:

- Compression
- Tension
- Reactions at supports or where fixed
- Bending and deflection

- Shearing
- Torsion
- Buckling

C4-3.1.1 Compression

When the component is loaded, the length is shortened and the member is in compression.

C4-3.1.2 Tension

When the component is loaded, the length is extended and the member is in tension.

C4-3.1.3 Reactions at supports

Remember Newton's law: for every action (load), there is an equal and opposite reaction.

Example

Reactions for dead loads on bridges:

- If W is the total weight of the bridge deck of a single span bridge, the reaction is $W/2$ at each of the 2 abutments.

Reactions for live loads on bridges:

- Reactions change as the train moves over the bridge, but reaction at Abutment 1 plus the reaction at Abutment 2 always equals the total train load.

C4-3.1.4 Bending

Loads on a bridge superstructure cause it to bend, resulting in a deflection. The bottom of the superstructure extends and is in tension. The top of the superstructure shortens and is in compression. Refer to Figure 1.

A single span bridge responds to load by deflecting. The further out from the abutment, the greater the deflection, until the midpoint of the span is reached. It is here that the deflection is greatest.

For a bridge with 2 spans and the superstructure continuous over the pier, then the load on the bridge causes deflection in both spans, with the same effect as in Figure 1, but causes the reverse effect over the pier. Hence, over the pier, the top of the superstructure is in tension and the underside is in compression. Refer to Figure 2.

This type of structural action results in bending stress. The maximum bending stress occurs at mid-span and over the pier.

Note that rail underbridges are generally not continuous at piers and act as separate single spans.

The tension and compression must be carried by the material – steel, concrete or timber. If either are too high for the particular beam, the beam will fail in tension (fracture) or compression (crushing).

Concrete is weak in tension and strong in compression. The main reinforcing steel is placed in the tension area.

Steel is strong in tension and compression.

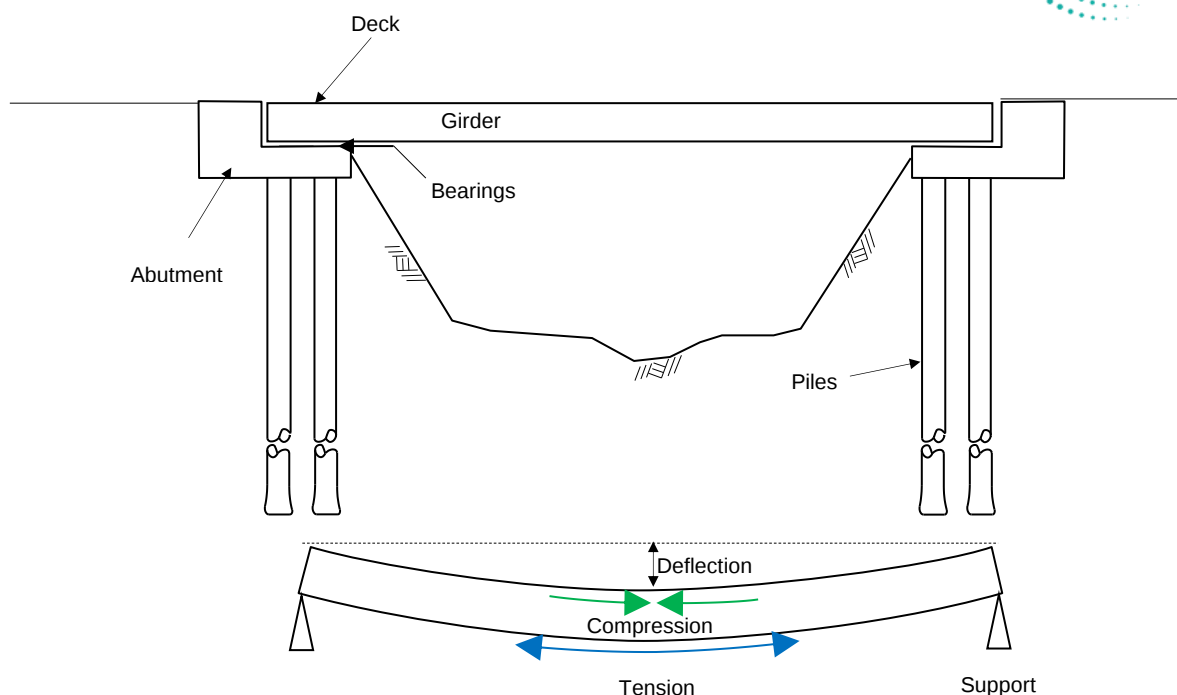


Figure 1 - Simple Span – Response to load

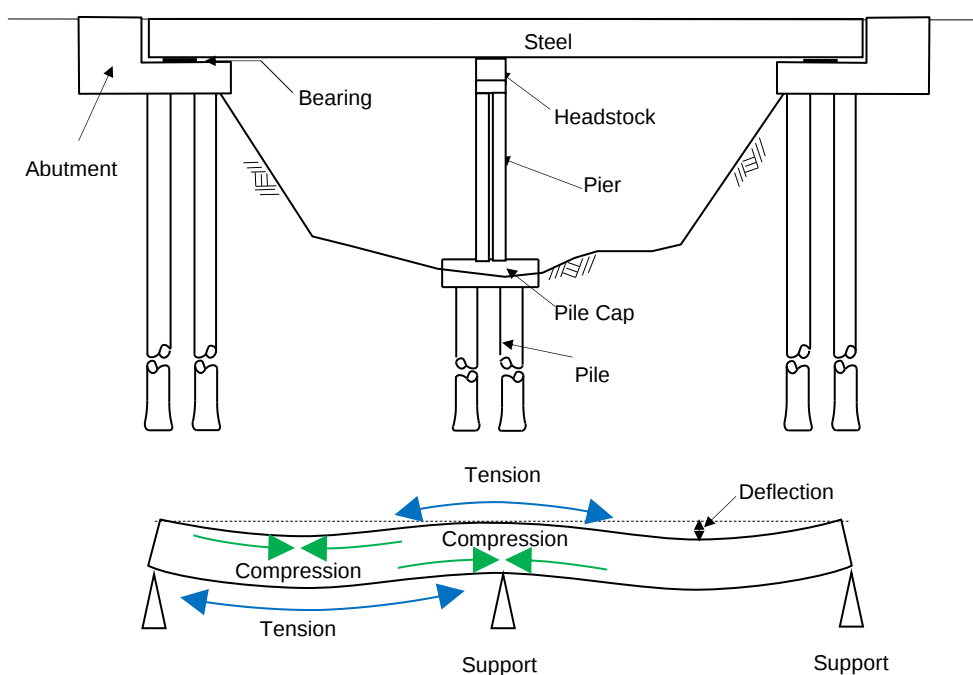


Figure 2 – Multiple Span – Response to load

C4-3.1.5 Shearing

Shear force acts perpendicular to the member axis. In bridges it is principally the vertical force at support points – piers and abutments. Refer to Figure 3.

There is a tendency for the beam to fail by the shearing action of reaction and loads.

The maximum shear loading occurs near the pier. This is the reason for the use of headstocks. Headstocks spread the load from the deck over a greater area, hence dissipating its intensity.

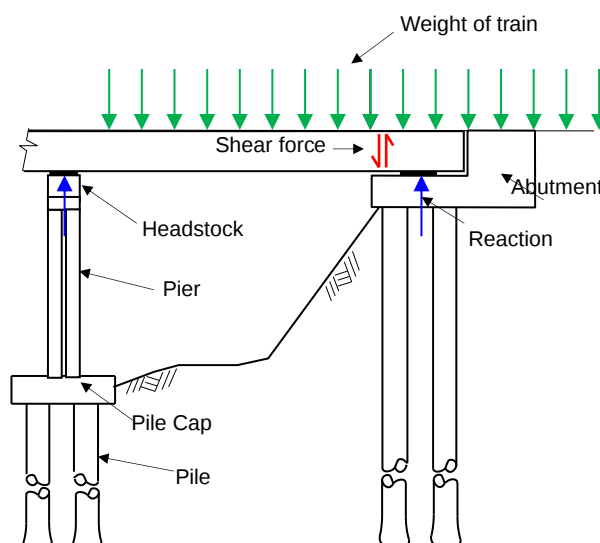


Figure 3 - Shear Force

C4-3.1.6 Torsion

Torsion, or twisting, can be caused by application of eccentric (off-centre) loads.

C4-3.1.7 Buckling

When the load on a compression member is continuously increased, the member at some point will suddenly displace sideways and fail by bending. This is known as buckling.

Examples of buckling are:

- Crushing a can is a buckling failure. The load is increased until the walls of the cylinder fail in buckling
- Buckling of the top chord of a truss with inadequate or no bracing when subject to a high compression load

Buckling is not caused by a material failure. It is related to the load and physical properties of the member.

The load at which buckling occurs can be increased by providing lateral support to the member. If the member is restrained laterally at mid-length, the free length of the member is reduced and the buckling load is increased e.g. lateral brace between the two trusses of a bridge to prevent buckling of the top chord.

Webs of steel beams are usually thin so they are prone to buckling. Web stiffeners are added to prevent buckling.

C4-4 Railway loads for bridges

Figure 4 shows the 300LA loading which is the design load from the current bridge design code (AS 5100).

This is a standard design loading (live load) and is meant to represent the worst case loading and load configuration that a bridge will be subjected to.

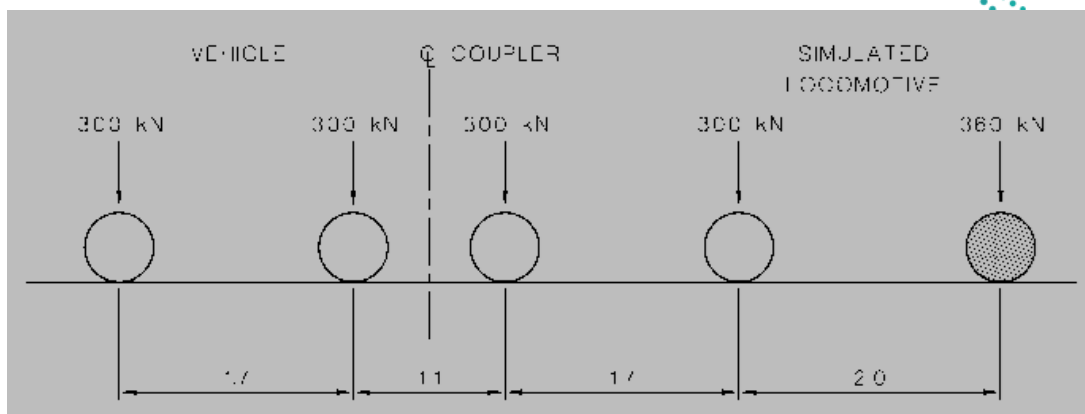


Figure 4 - 300LA Railway Traffic Loads – Axle Loads

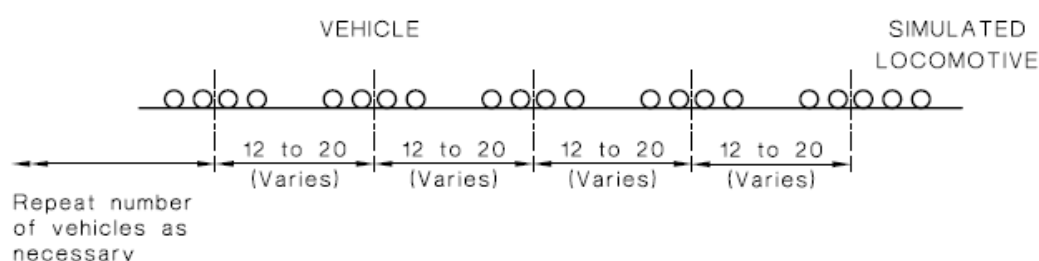


Figure 5 - 300LA Railway Traffic Loads – Axle Group Spacing

C4-5 Effects of dynamic loading

Fatigue of structural members is an outcome of applied loadings on bridges and structures. Fatigue is due to repeated cycles of loading and unloading and occurs when a live load, such as trains travelling over a bridge, subjects the structural elements to cycles of loading and unloading (load reversals).

In the case of bridge structures the materials that are susceptible to fatigue induced failure are steel components and the elastomeric material in bridge bearings.

A possible outcome of fatigue loading is that it causes an otherwise ductile material to fail suddenly with little warning. This can have severe consequences as the failure usually occurs under the imposition of live load (i.e. with a train and/or people on the structure).

Fatigue results in the formation of fine, even minute, cracks that usually propagate from a defect in the material, a bad detail such as a square opening with sharp corners or damage to the structure, in a part of the bridge or structure that is subject to large loading reversals (e.g. the bottom chord of a steel truss at mid-span with flame-cut holes for drainage).

C4-6 Load rating

Load rating is a design process for determining the safe load capacity of a bridge. The initial rating of a bridge is the design load. The rating changes over the life of the asset due to deterioration of its components.

C4-6.1 Underbridges

Over the years, design loads have changed as design codes have developed. Underbridge design loads have been expressed as:

1. Cooper E (imperial).
2. Metric Cooper M.

3. 300-A-12.
4. 300LA.

Ratings for CRN underbridges are generally expressed as M or LA loadings.

Most bridges have been designed to older design codes and do not necessarily comply with the current design code.

Details of the changes in loadings are given below:

2004 AS 5100 Bridge Design

The current design load is 300LA as shown in C4-4.

1996 Australian Bridge Design Code (300-A-12)

The 300-A-12 loading consists of groups of four axles each having a load of 300 kN, and having axle spacings of 1.7 m, 1.1 m and 1.7 m as shown in Figure 6.

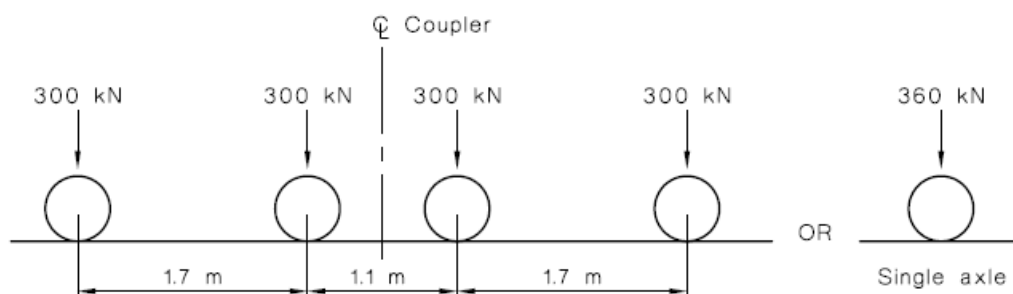


Figure 6 - 300-A-12 Axle Loads

The spacing between the centres of each axle group should be taken as 12m (see Figure 7).

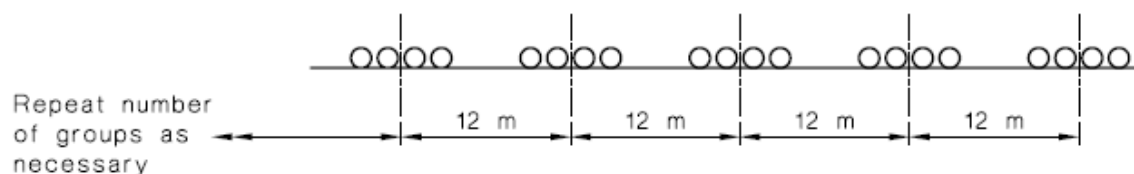


Figure 7 - 300-A-12 Axle Group Spacing

The 300-A-12 also includes a single axle load of 360 kN. The single axle load is not applied concurrently with other vertical railway live loading.

1974 Australian and New Zealand Railway Conferences (ANZRC), Railway Bridge Design Manual (Metric Cooper M)

The ANZRC Metric Cooper M loading is an approximate metrication of the American Railway Engineering Association (AREA) Cooper E loading for Iron and Steel Structures, Concrete Structures and Foundations, which was imperial. The maximum design live load in the state railway systems was AREA E 60. This was approximately metricated to ANZRC M 267 that was usually rounded off to M 270.

The ANZRC gave the recommended design load as M 250 as shown in Figure 8.

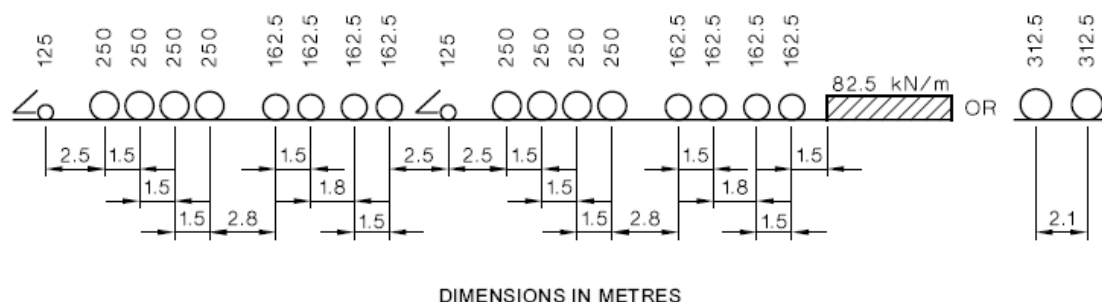


Figure 8 - M250 Live Load

Irrespective of the code or standard referred to, the higher the number the stronger the bridge i.e. it can carry higher loads and has more ability to withstand the effects of defects.

The design loads given below cover only the major vertical loads. They do not include dynamic load allowance (impact).

Note that dynamic load allowance generally increases with older codes as older non-dynamically balanced steam locomotives generated higher dynamic loads.

For underbridges, the current minimum design loads for the various lines are as follows:

The following design requirements are extracted from CRN Engineering Standard CRN CS 310 "Underbridges"		
Track Class	Design Load configuration	
Main Lines		
Heavy Haul Coal Operations	350-LA plus DLA	
Class 1 and 2 lines	300-LA plus DLA	
Class 3 and 5 lines	280-LA plus DLA	
Sidings		
General Yard	300-LA plus 50% DLA	
Sidings (includes unloading bins)	330-LA plus 0% DLA	
Passenger operations/ or maintenance	180-LA plus 0% DLA	

C4-6.2 Overbridges

For overbridges, design loads are specified in terms of 'T' or 'R' loading (see Figure 9). Newer bridges would carry the B-double truck configuration and would be rated as carrying the T44 load. Older bridges are rated using the rigid truck configuration and are nominally in the range R20 to R30.

The 'R' vehicle is a rigid truck with the same configuration as the prime mover portion (first 3 axles) of the 'T' vehicle and the numerical portion is the vehicle's weight in tonnes.

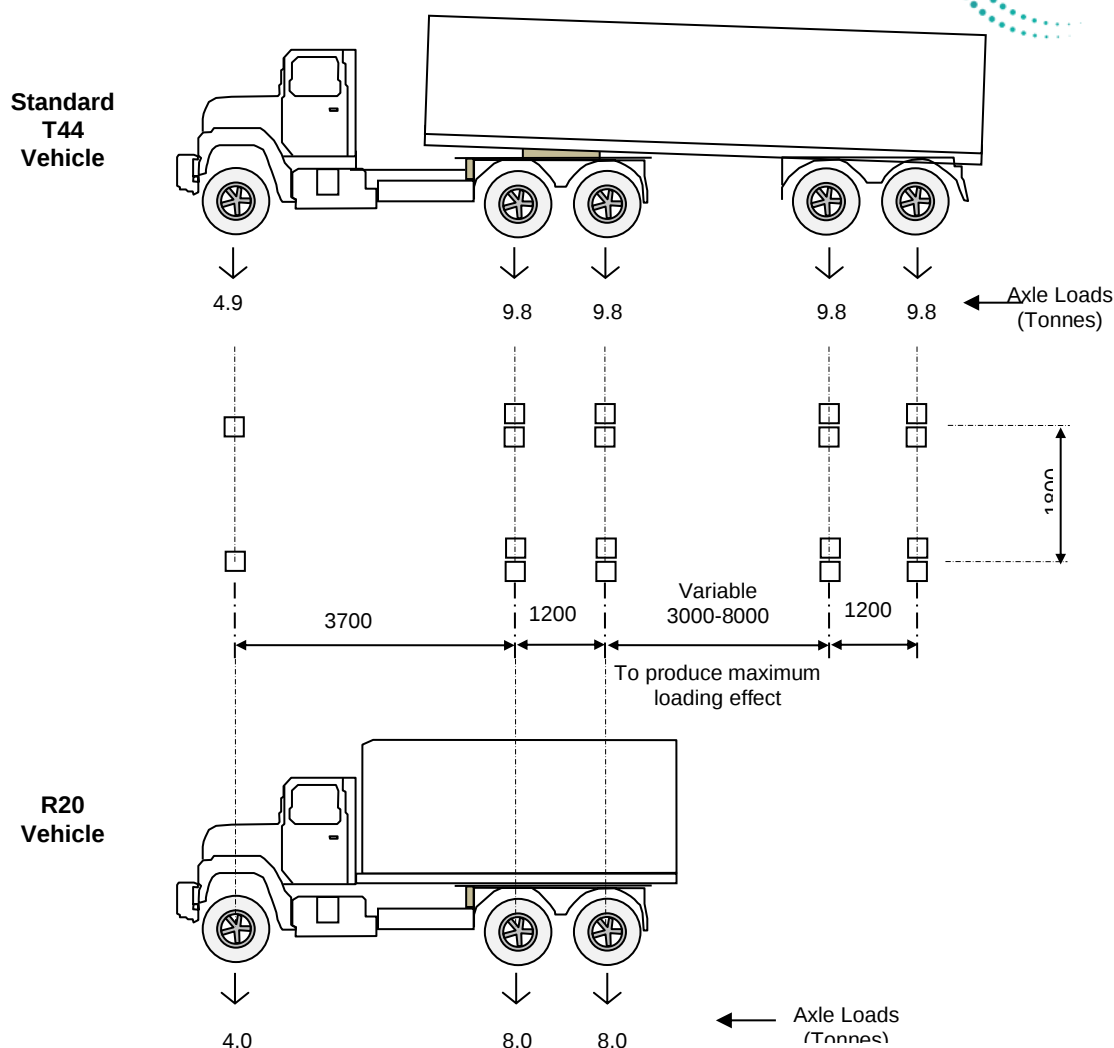


Figure 9 - Design Road Vehicle Configurations

C4-6.3 Footbridges

For footbridges, design loads are specified in terms of a uniformly distributed load i.e. the load per square metre of loaded area.

The design load for rail footbridges is 5 kPa i.e. 5 kN per m².

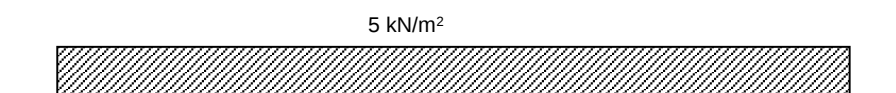


Figure 10 – Pedestrian Load on Footbridge

C4-7 Structural Drawings

Where available, reference should be made to the drawings for a structure. These will provide details of the structure, components, connections, bearings, foundations etc.

For modern structures, these drawings are usually well laid out with details and sections being easy to find read and interpret.

Unfortunately for structures that were designed many years ago, the drawings can be very difficult to read as at that time they tried to minimise the number of sheets by including as much information as possible on each sheet.

Older drawings may not contain important information such as design loads or as-built details such as founding depth of piles.

C4-8 Bearings

C4-8.1 Functions of bearings

Bearings are a critical part of a bridge. If they do not operate as intended, structural damage may occur to the bridge.

Bearings provide both a support function for and a resisting function to the range of loads applied to a bridge.

They provide support for vertical loads and lateral loads:

- Dead loads
- Live loads, including dynamic effects
- Impact loads from vehicles and ships
- Wind loads
- Flood loads, including debris and stream flow
- Loads caused by track superelevation – centrifugal force.

They provide resistance to longitudinal horizontal loads and horizontal loads:

- Earth pressure loads transmitted from abutment – earth pressure moves the abutment and applies a force to the bearing as the girder will move
- Braking effects of live loads – braking load is transferred to the abutment through the fixed bearing.

They also resist other effects:

- Thermal expansion and contraction
- Shrinkage and creep of concrete (shortening). This is similar to thermal contraction but permanent
- Differential settlement of supports may cause excessive rotation of the bearing
- Mining subsidence may cause excessive rotation of the bearing
- Earthquake loads may generate forces in excess of the bearing capacity.

A bearing has to withstand the following types of movements:

- Rotation
- Longitudinal movements
- Transverse movements

When a girder bends under load, the bearing must be able to accommodate the rotation caused by the deflection of the girder. How the bearing rotates depends on the type of bearing used.

It is essential that movement can occur at expansion bearings. Loss of movement is likely to cause damage to the superstructure and substructure. Expansion bearings must have sufficient capacity for the expected movement.

As loads are applied to bridge spans, each span either slides, shears or rolls on the bearing.

All longitudinal loads go to the fixed bearings if the other end is supported on free bearings.

Transverse loads are usually resisted at piers and abutments.

C4-8.2 Types of bearings

The types of bearings used for CRN bridges are:

1. Sliding.
2. Rocker.
3. Roller.
4. Elastomeric rubber.
5. Elastomeric rubber with steel plates.

C4-8.3 Bearing assessment issues

The primary requirement is the ability of the bearing to accommodate movement and rotation in the bridge superstructure.

Bearings must be kept clean and, where required, lubricated so that they can continue to perform as designed.

Free bearings must be able to move freely. Poorly maintained bearings may become frozen i.e. they are unable to move.

Frozen bearings can result in forces being applied to the substructure that it was not designed for. The most common outcome is cracking of the bearing pad or of the substructure components.

Common defects in bearings are:

1. Corrosion at the interface of steel components.
2. Accumulation of dirt.
3. Lack of lubrication.
4. Deformation of rubber bearings.

Chapter 5 Assessment of steel structures

C5-1 Deterioration modes in steel structures

C5-1.1 General

The main indicators of deterioration of steel or iron structures are section loss, cracking, loss of protective coating, deformation of members and loose or missing connections.

Other factors to be taken into consideration may include the age of the structure, vulnerability to impact, location of the defect and importance of affected member(s).

The main modes of deterioration in steel, cast iron and wrought iron members are:

- Breakdown of the corrosion protection system
- Corrosion of exposed surfaces or at interfaces with concrete or steel
- Loose or missing connectors
- Impact damage
- Buckling of members
- Fatigue cracking
- Cracking of welds
- Delamination (wrought iron)

Of the above modes, corrosion is the most prevalent factor affecting steel structures. In assessing the significance of the corrosion it is necessary to determine its extent, severity and location. This significance may vary from superficial surface corrosion only through to a defect condition resulting in a loss of load carrying capacity and even possible failure.

Cracking of welds or members, buckling and impact damage are other defects that can lead to sudden collapse or a reduction of load carrying capacity. Cracks in tension flanges must always be regarded as serious and requiring urgent action.

C5-1.2 Corrosion

The majority of deterioration in steel and iron components results from the breakdown or loss of the protective system. Without adequate protection, steel and iron components are vulnerable to corrosion and hence loss of section.

Corrosion may be prevented by any of the following systems:

- Durable protective barriers such as painting, encasing in concrete or galvanising to prevent oxygen and moisture reaching the steel
- Inhibitive primers which hold off attack on the steel substrate
- Provision of sacrificial anodes such as zinc rich paints or galvanising
- Provision of cathodic protection by use of an external current to suppress the anodic reaction. This process is also used for concrete bridges for arresting corrosion in reinforcement

The protective system usually adopted for bridges is painting or galvanising, however the loss or partial loss of either of these systems will see the onset of deterioration. The accumulation of debris around bearings, on flanges or the base of the substructure will further hasten the corrosion process by providing a moist environment. It is therefore important for these areas in particular to be regularly examined and cleaned.

C5-1.3 Impact damage

The next most common cause of deterioration of steel and iron members comes as a consequence of impact loading. Steel trestles are particularly vulnerable to major deformation or even failure from train or vehicle impacts. The bottom flange of girders, bracings and cross girders are also exposed to risk of impact from high vehicles or protruding loads. Damage can include scraping, shearing of bolts or rivets, buckling of members, loss of protective barrier and notching (which can lead to crack propagation).

C5-1.4 Fatigue cracking

Repetitive loading cycles and or overstressing of steel and iron members can eventually lead to fatigue cracking. A continuation of the loading cycle can result in the propagation of cracks and finally failure. Fatigue cracking is usually initiated at high stress concentration points such as bolt and rivet holes, welds, re-entrant corners, change of sections or areas of restraint.

C5-1.5 Loose or missing connections

Loose or missing connections are another common cause of deterioration of steel or iron structures. These may result from vehicle impacts, severe corrosion, incorrect initial installation, vibration and tensile failure of the connector.

C5-2 Assessment of deterioration

IMPORTANT

The most important matter in any assessment of damage is to establish if the strength and stability of the structure are adversely affected. Where safety of a structure is in question, professional advice should be sought immediately for the protection of the structure as well as its users against further damage, collapse or injury.

C5-2.1 General

To successfully repair a deteriorated steel structure it is essential to identify the cause, extent and rate of deterioration, and whether or not the cause is still active.

A step by step procedure for assessing deterioration is given below. This procedure includes a number of simple tests that can be easily carried out on site. It is not essential to carry out all the tests and judgement should be used in applying the tests according to the severity of the problem at hand.

It is also recognised that resources for carrying out these tests may not be available and detailed investigation may have to be entrusted to specialist firms or consulting engineers who have appropriate expertise to establish the causes of deterioration and advise on what repair action should be taken.

C5-2.2 Assessment procedures

- Before proceeding, assess if detailed examination of the damage or deterioration will require track closure, pedestrian and traffic restrictions, assistance from police and utility authorities, worksite protection, special equipment (ladders, cherry pickers) and any special safety measures
- Study previous investigation and repair reports available, if any. Examine the condition of the past repairs to determine whether they have been successful or if the deterioration is growing worse
- Carry out a visual inspection of the structure and, if necessary, use hand magnifiers, binoculars and telephoto photography to record the type and extent of deterioration. Estimate the crack widths. If possible, ascertain the obvious causes of deterioration such as corrosion, poor drainage, environmental conditions, accidental damage etc.

- Ascertain if the cracks are "live", that is, their width changes under thermal or structural loading. This can be detected with a mechanical strain gauge held on gauge discs glued to the steel surface. Cracks that are due to applied load will move immediately the load is changed (e.g. under traffic passing over a bridge). Cracks due to thermal movement move when the temperature of the element alters. Measurements made three or four times a day should establish whether a crack is live or not
- Observe members under load and note any excessive movement in members or fastenings
- Check for section loss using an ultrasonic thickness meter to measure section thickness
- Check thickness of paint using dry film thickness gauge

C5-2.3 Other detection methods

Listed below are additional tests that require special equipment and significant skills and experience to obtain usable results. Such testing methods would have to be undertaken by specialist personnel skilled in the field of diagnostic testing.

- X-Rays
- Ultrasonic testing
- Magnetic particle testing
- Acoustic emission
- Laboratory analysis of steel samples
- Thermal imaging

C5-2.4 Steel repair issues

The most important issue in the repair of any structure is to ensure adequate strength and stability at all times. In such cases the strength and stability of the structure during repairs should be checked by a structural engineer before commencing repairs. Also, load restrictions should be applied and the structure temporarily supported as necessary.

The repair of steel structures requires knowledge of the following issues:

- Types of defects which can occur due to deterioration, e.g. cracks, corrosion, section loss, loose connections
- Causes of deterioration, e.g. water build-up (poor drainage), impact damage, shrinkage and thermal effects, foundation movements, environmental effects etc.
- Test methods for assessing the severity of deterioration
- Ability to interpret test results
- Selection of appropriate repair materials
- Selection of appropriate repair procedure

The above issues are discussed in detail in Section C10-6 of this manual.

Chapter 6 Assessment of concrete structures

C6-1 Deterioration modes in concrete structures

C6-1.1 General

The main indicators of deterioration of concrete and masonry structures are corrosion of the steel reinforcement, spalling, cracking, fretting and loss of mortar at joints.

Other factors to be taken into consideration may include the age of the structure, vulnerability to chemical attack, vulnerability to impact and foundation movements (including mine subsidence), environmental effects etc.

Concrete members deteriorate in service in the following ways:

- Weathering or spalling at exposed faces, resulting from erosion, poor quality concrete, chemical action, water action, corrosion of reinforcement, insufficient cover to rebars, crushing at bearing surfaces and drumminess
- Cracking from loading changes, including settlement
- Mechanical damage, especially from road or rail vehicles

Common defects that occur in concrete structures and therefore require checking during examination are as follows:

- Corrosion of reinforcement, with subsequent cracking and spalling
- Scaling – cement render breaking away
- Delamination
- Leaching and water penetration
- Rust stains
- Honeycombing or other construction deficiencies
- Fire damage
- Dampness
- Leaking joints
- Breaking up of existing repairs
- Shattering and crushing of bearing pads

C6-1.2 Corrosion

The major failure mode in concrete structures is corrosion of the reinforcement. The product of this corrosion has a volume many times larger than the parent metal. This results in a build-up of internal pressure that leads to de-bonding, cracking and eventual spalling. When a crack develops the rate of deterioration accelerates and this can lead to defects such as leaching, water penetration and rust staining.

Corrosion can be caused by many means ranging from construction deficiencies to mechanical weathering or chemical action. All of these threaten the protective barrier the concrete provides for the reinforcement. Once this process has been initiated and the reinforcement protection is lost, the rate of deterioration is accelerated dramatically. The physical properties of the concrete, environmental conditions, concrete cover and other design or construction practices will all influence the rate of deterioration.

C6-1.3 Other factors

Other factors that cause concrete structures to deteriorate include:

- Impact loading
- Overload
- Foundation movements
- Seizure of bearings
- Differential thermal strains
- Freeze/thaw cycles
- General wear and abrasion
- Leaching
- Chemical attack (carbonation, chloride contamination, sulphate attack and alkali aggregate reactivity)

C6-2 Deterioration of concrete

C6-2.1 Factors affecting deterioration

Deterioration of concrete is affected by the following factors:

- Quality of the constituent materials (cement, sand, aggregate, admixtures and water used in the manufacture of concrete)
- Environmental conditions (exposure to air, water, chemicals, industrial pollutants, marine condition, frost etc.)
- Physical properties of concrete (permeability, compressive strength, density, cement content, shrinkage and creep characteristics)
- Standard of workmanship during construction (fixing of formwork and reinforcement, placing, compaction and curing of concrete)
- Concrete cover to steel reinforcement
- Design practice (mix design, detailing of reinforcement and concrete cover, practicality of construction)

C6-2.2 Causes of deterioration

The main causes of deterioration of concrete are summarised below:

C6-2.2.1 *Corrosion of reinforcement*

Rusting or corrosion of reinforcement is one of the major causes of deterioration of reinforced concrete. Corrosion involves a combination of processes, mainly carbonation and chloride contamination, leading to de-passivation of steel and its subsequent corrosion by electrolytic reaction.

In theory, the steel reinforcement is protected from corrosion by a film of oxide that is stable in the alkaline environment of the surrounding concrete. The alkalinity is provided by the free calcium hydroxide (lime) present in the Portland cement. The process of corrosion is initiated by de-passivation of the steel, i.e., breakdown of the protective oxide layer due to degradation of the alkaline environment. The alkalinity in concrete may be reduced by either of the following causes:

- Leaching out of the free lime by water if the concrete is porous.
- **Carbonation:** Penetration into the concrete of carbon dioxide present in the atmosphere. The carbon dioxide dissolves in the pore water of the concrete and reacts with the free calcium hydroxide to form neutral calcium carbonate. This reaction progressively lowers the alkalinity of concrete and results in removal of the passive oxide layer from the steel. The carbonation rate

is very dependent on the concrete quality. Concrete with high water/cement and high porosity carbonates very rapidly.

- **Chloride contamination:** The chlorides can come from a number of sources including contaminated aggregates, admixtures such as calcium chloride and exposure to sea water, salt spray or saline water. The chlorides in the pore water within the concrete form an electrolyte and the chloride ions locally de-passivate the steel reinforcement by breaking down the protective oxide layer even in highly alkaline concrete.

The steel corrosion process is usually slow, but always progressive. The products of corrosion occupy a volume greater than the parent metal. This volume increase generates high internal pressures that cause de-bonding, cracking and eventually spalling of the concrete.

C6-2.2.2 Sulphate attack

Sulphate attack is initiated by sulphates that may be present in ground water or are formed by penetration into concrete of sulphur dioxide from the air, particularly in areas of industrial pollution. The sulphates react with calcium hydroxide to form gypsum (CaSO_4) that subsequently reacts with tricalcium aluminate (C3A) of the cement to form a swelling sulpho-aluminate substance known as ettringite. Concrete affected by sulphate attack expands, initiating cracking and spalling that provides access to reinforcing steel for the very aggressive sulphate ions and results in corrosion of the steel.

C6-2.2.3 Alkali aggregate reactivity

Certain aggregates can react with the alkali present in cement to form a gel that swells by absorbing moisture and cracks the surrounding concrete. As the reaction advances cracks extend over the surface in a random "mud crack" pattern with pop-outs, appearing first on the most weathered surfaces. Alkali aggregate reaction (AAR) also manifests itself as closing up of gaps in concrete and development of cracks along stress lines in the vicinity of concentrated loads such as bearings and prestressed anchorage zones. In the long term the gel exudes through the cracks as white efflorescence and the concrete shatters into small blocks or along stress lines, but may be held together by steel reinforcement. The effects of AAR are unsightly and structurally debilitating.

C6-2.2.4 Creep, shrinkage, thermal and load effects

Cracking is induced in concrete structures when the free movement due to shrinkage of concrete and thermal expansion and contraction is restrained, even under simple loads. Thermal effects include those occurring during the heat of hydration in fresh concrete poured in restrained locations (e.g. against "cold" joints). Cracking results when the new concrete cools and shrinks.

In continuous structures, temperature gradients may also cause flexural cracking due to large sagging moments produced over the supports. Similarly, differential shrinkage between the precast girders and the cast-in-place deck of continuous bridges may result in cracks in the region of the hogging moments.

Cracks produced by these effects should have been allowed for in the design, and are most likely to occur due to inadequate design, poor detailing and poor construction. The structural significance of these cracks should be checked by a structural engineer prior to any treatment that may range from no action to external strengthening. If the concrete is sound such cracks should have little effect on corrosion unless they run along reinforcing steel or are very large.

C6-2.2.5 Frost and salt attack

Frost attack is unusual in Australia but salt attack is more common and will occur where concrete is in intimate contact with sea, salt lakes or high salinity soils.

Under frost the water in the pores of concrete freezes and expands generating high internal pressures that shatter the concrete surface. Salt in a saturated solution can also seep into concrete pores and crystallise by evaporation of the water. The crystals expand and generate high pressures that spall the surface of the concrete. Both effects are pronounced in poor quality concrete and lead to corrosion of steel.

C6-2.2.6 *Impact forces*

Impact damage can be caused in concrete structures by:

- Collision of, or glancing blows from, motor vehicles against piers, abutments, parapets and walls
- Derailment of trains
- Overheight vehicles striking against the underside of a bridge superstructure
- Impact of heavy floating logs carried by rapid flowing streams against a bridge structure

This type of damage generally causes cracking and spalling of concrete with or without exposure of reinforcing steel. Severe impact may also result in rupture or fracture of members and collapse of the structure.

Impact damage should be repaired promptly before the reinforcement has started to rust and before the damaged surfaces are affected by carbonation or contamination.

C6-2.2.7 *Overloading*

Overloading of bridge structures may occur due to vehicles with above legal limit weight, increase in train loads since the construction of bridges, extremes of temperature causing excessive movements, high temperature differentials within the structure, high winds, excessive build-up of road metal or ballast on the deck or build-up of flood debris against the structure. Overloading can cause cracking of concrete members. Excessive overloading may result in fracture of members and collapse of bridge. Cracks that have formed as a result of accidental overload will tend to be very fine after the load has been removed and often need no treatment. Cracks wider than 0.3 mm may need to be sealed.

For increased train loads, the strength of all components of the bridges should be assessed by structural calculations.

C6-2.2.8 *Faulty construction*

Faulty construction is one of the most common causes of early deterioration of concrete.

Common construction faults include:

- Formwork not cleaned out properly (pieces of timber, nails and debris embedded in finished concrete)
- Formwork not made watertight (honeycombed concrete due to loss of cement grout)
- Inadequate compaction (voids in concrete)
- Over compaction (laitance on top surface leading to scaling)
- Lack of sufficient concrete cover to steel either by failure to fix the reinforcement correctly or due to poor design
- Inadequate and insufficient curing

C6-2.2.9 *Deterioration at joints*

Joints in concrete structures are especially vulnerable to deterioration for several reasons:

- They can be difficult to construct and the concrete at a joint may lack compaction
- They may act as paths for the entry of salt water or carbon dioxide
- They may fail to work as joints forcing the concrete to crack at an adjacent plane of weakness (e.g. at the end of dowels or at the fin of a waterstop)
- They may not be intended to be active joints (e.g. construction joints) but may subsequently become active without having any provision for sealing

These faults will eventually result in rusting of reinforcement after the concrete has become carbonated. If repair action is taken quickly, before the reinforcement has started to corrode, it is possible to prevent or greatly reduce the extent of damage from these causes.

C6-3 Repairs of prestressed girders

C6-3.1 General

Maintenance on prestressed concrete bridges is generally carried out when repairs are needed. Such repairs could be initiated by:

- Damage from overloading, impact, etc.
- Spalling concrete due to rusting reinforcement
- Cracks in concrete
- Ground movements
- Servicing needs of bearings
- Damage or malfunctioning of deck joints
- Problems with deck drainage

Due to the high compressive stresses applied longitudinally, the girder should never develop transverse cracking (cracks normal to longitudinal axis of bridge) and the concrete may tend to spall explosively if subjected to severe disturbance. Any transverse cracking should, therefore, be the subject of further investigation and care should be taken during any repairs or modifications to the girder.

The tendons are enclosed in steel ducts, located in the webs and filled with grout. The strands should not be disturbed or damaged. Similarly the prestressing strands in pre-tensioned girders should not be disturbed or damaged.

C6-3.2 Causes and Associated Symptoms

Cause	Symptom	Remarks
Overload	Cracking in deck soffit at mid span. When load is removed, cracks should close due to the prestressing force.	Confirm that the girders were designed for no cracking under service loads. Monitor and document.
Ship impact	Easily visible as scrapes, chips and damaged concrete	Possible collisions from light vessels. Negligible consequence.
Derailment (minor)	The top slab cantilevers have been designed to support wheel loads.	Unlikely to cause damage to the structural elements.
Fire	Concrete has a good resistance against fire damage.	Unlikely to be a major problem unless inside the box.
Durability	Spalling Concrete	Check concrete surfaces for a drummy sound (when hit with a hammer) at an early stage

Cause	Symptom	Remarks
Concrete Cracks		
End blocks bursting	Cracks normally run radially to the prestressing strands/tendons over a short distance.	No concerns since they are usually initiated during or soon after construction (jacking).
Girder webs	Shear cracks usually are inclined sloping outwards from a point of support.	This is very serious.
Top and bottom slabs	Shear lag cracks in (due to uneven distribution of prestress) most likely to appear as diagonal cracks in the top slab cantilevers	Unlikely due to high compressive stress. No great consequence.
Shrinkage	Cracks normally longitudinal (and parallel sided), generally with a regular pattern.	No great consequence.
Derailment	Transverse flexural cracking would appear as longitudinal cracks on one surface only.	Not serious.
Flexure	Longitudinal flexural cracking would appear transverse to the superstructure.	Unlikely to occur given the level of prestress.
Other causes	Cracks are surface mainly and less than 0.25mm wide.	Not serious. No short-term action required.

C6-4 Assessment of deterioration

IMPORTANT

The most important matter in any assessment of damage is to establish if the strength and stability of the structure are adversely affected. Where safety of a structure is in question, professional advice should be sought immediately for the protection of the structure as well as its users against further damage, collapse or injury (see Figure 11).

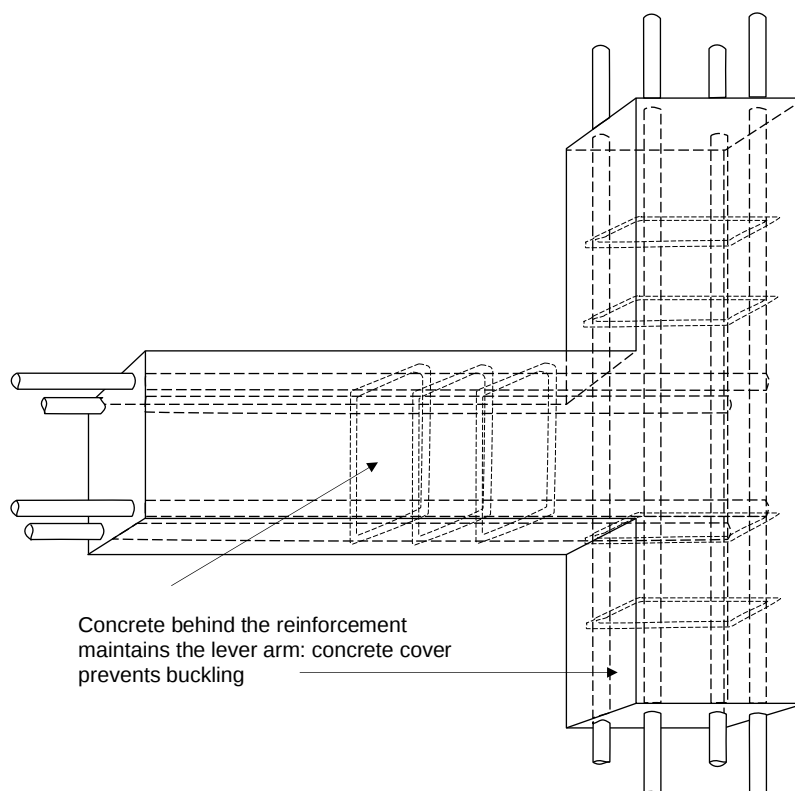


Figure 11 - Importance of concrete to stability

WARNING

Even the most basic joint depends on the concrete cover for its strength.

Before removing any concrete from a load bearing structure carefully consider whether the concrete you propose to remove is providing essential support for the structure.

If it could be, support the structure first.

C6-4.1 General

To successfully repair a deteriorated concrete structure it is essential to identify the cause, extent and rate of deterioration of concrete and whether or not the cause is still active.

A step by step procedure for assessing deterioration is given below. This procedure includes a number of simple tests that can be easily carried out on site. It is not essential to carry out all the tests and judgement should be used in applying the tests according to the severity of the problem at hand.

It is also recognised that resources for carrying out these tests may not be available and detailed investigation may have to be entrusted to specialist firms or consulting engineers who have appropriate expertise to establish the causes of deterioration and advise on what repair action should be taken.

C6-4.2 Assessment procedures

- Before proceeding, assess if detailed examination of the damage or deterioration will require track closure, pedestrian and traffic restrictions, assistance from police and utility authorities, worksite protection, special equipment (ladders, cherry pickers) and any special safety measures

- Study previous investigation and repair reports available, if any. Examine the condition of the past repairs to determine whether they have been successful or if the deterioration is growing worse
- Carry out a visual inspection of the structure and, if necessary, use hand magnifiers, binoculars and telephoto photography to record the type and extent of deterioration. Estimate the crack widths. If possible, ascertain the obvious causes of deterioration such as reinforcement corrosion, poor drainage, environmental conditions, accidental damage etc
- Examine for hollowness and delaminations by tapping with a hammer, medium size spanner or a steel rod. Use chain-drags on slabs. Assess and mark out suspected areas of hollowness and delamination
- Ascertain if the cracks are "live", that is, their width changes under thermal or structural loading. This can be detected with a mechanical strain gauge held on gauge discs glued to the concrete surface. Cracks that are due to applied load will move immediately the load is changed (e.g. under traffic passing over a bridge). Cracks due to thermal movement move when the temperature of the element alters. Measurements made over a range of temperature and loading conditions should establish whether a crack is live or not
- Estimate the concrete cover to reinforcement using electromagnetic cover meters or by actual measurement where concrete is broken and reinforcement is exposed. Check if the cover provided is adequate for the exposure conditions, or is as per drawings (if the drawings are available)
- Testing for carbonation:

Break off small pieces of concrete from different areas of the structure using a hammer and cold chisel and test freshly exposed concrete surfaces by spraying with 2 per cent solution of phenolphthalein in alcohol. This pH indicator solution will change colour according to the alkalinity of the concrete. The solution remains pink and is easily visible on concrete that has retained its alkalinity but becomes colourless on concrete that has lost its alkalinity by carbonation. The test will thus indicate the depth to which the concrete has been carbonated from the surface.

- Testing for chloride contamination:

To determine the chloride content of concrete, samples are obtained by drilling holes in the concrete and collecting the dust produced (NB. if there is any surface salt built up, it must be removed before drilling). The dust samples are collected at different depths, e.g. 0-10 mm, 10-25 mm, 25-50 mm and so on to determine how the chloride content changes with depth from the surface. It also helps to establish whether the chloride was present in the concrete when it was cast, or whether it penetrated the concrete from the surroundings (see Figure 12).

The concrete samples are treated with acid to dissolve the cement and the chloride content is determined by titration against silver nitrate.

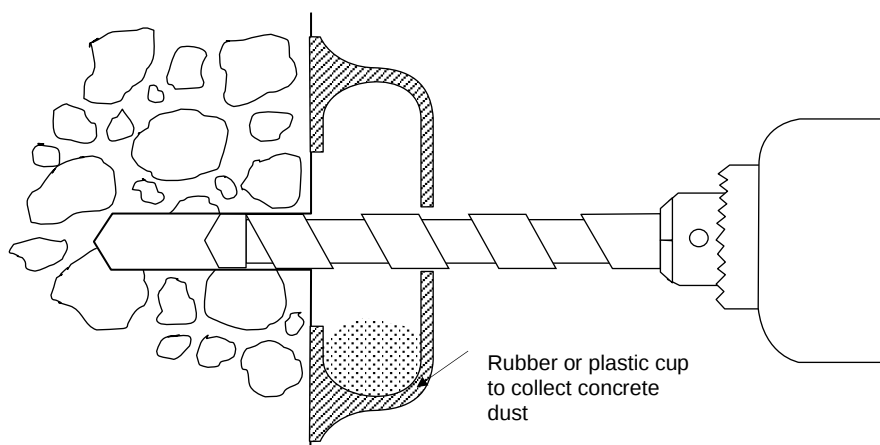


Figure 12 - Sampling for chloride content

C6-4.3 Other detection methods

Listed below are additional tests that require special equipment and significant skills and experience to obtain usable results. Such testing methods would have to be undertaken by specialist personnel skilled in the field of diagnostic testing.

- Half-cell potential measurements to assess corrosive activity in concrete and the probability of corrosion in steel reinforcement
- Ultrasonic pulse velocity measurements to locate areas of delamination and honeycombed concrete
- Electrical resistivity measurements to assess the rate of corrosion qualitatively
- Permeability tests to measure the water absorption of concrete
- PH testing of concrete
- Covermeter testing
- In-situ compressive strength measurements using Schmidt Hammer
- Core sample testing for strength, permeability, contamination, composition and density
- Measuring deflection of structural members under known applied loads

C6-4.4 Concrete repair issues

The most important issue in the repair of any structure is to ensure adequate strength and stability at all times. This is particularly relevant in repair of reinforced concrete elements where significant areas of concrete are to be removed. In such cases the strength and stability of the structure with the concrete removed should be checked by a structural engineer before commencing repairs. Load restrictions should also be applied and the structure temporarily supported as necessary.

The repair of concrete structures requires knowledge of the following issues:

- Types of defects which can occur due to deterioration, e.g. cracks, spalls, delamination, scaling, honeycombing etc
- Causes of deterioration, e.g. chloride penetration, carbonation, alkali - aggregate reaction, shrinkage and thermal effects, foundation movements etc.
- Test methods for assessing the severity of deterioration
- Ability to interpret test results
- Selection of appropriate repair materials, from ordinary Portland cement to synthetic polymers, resins and acrylics according to particular requirements of a repair project



- Selection of appropriate repair procedure

There are also other more complex and advanced methods for repairing concrete such as cathodic protection, chloride removal and re-alkalisation. These should only be entrusted to organisations specialising in this work and are not covered in this manual.

Chapter 7 Assessment of masonry structures

C7-1 Deterioration modes in masonry structures

C7-1.1 General

Masonry or stone is rarely used as a construction material for modern structures, except for facing or ornamentation. However, many structures within the CRN network were built from masonry construction and are still in service, owing to the general longevity of the material. Most deterioration can be attributed to weathering, migration of water, impact damage and foundation movements.

Common defects that occur in masonry structures and therefore require checking during examination are as follows:

- Loose, drummy or missing blocks
- Fretting of blocks and mortar joints
- Splitting or cracking of blocks and or mortar
- Cracking due to subsidence or relative movements
- Mortar loss
- Scrapes and spalls from impact
- Water penetration and leaching

C7-1.2 Causes of deterioration

Many different factors lead to the deterioration and development of defects in masonry structures. Most of these are very slow acting and require repeated occurrence.

Seasonal expansion and contraction causes repeated volume changes that lead to the development of seams and fine cracks. These may grow over time to a size that allows other factors to contribute to further deterioration. Frost and freezing in these cracks, seams or even in pores can split or spall blocks. Plant stems and roots growing in cracks or crevices can exert a wedging force and further split open blocks or mortar.

Plants such as lichen and ivy will chemically attack masonry surfaces in the process of attaching themselves.

Abrasion also leads to deterioration of masonry and this may be due to water or wind borne particles.

Fretting of bricks, blocks and mortar can be caused by the loss of the connecting or binding agents via leaching through the structure. The water can either be drawn up from the footings and backfill by capillary action or leak down through the fill.

Gases or solids dissolved in water can chemically attack the masonry. Some of these may dissolve the cementing material between the blocks and lead to mortar loss.

C7-1.3 Cracking

Differential settlement of parts of the structure or subsidence of foundations can lead to extensive and sometimes severe cracking. It is important to distinguish those cracks that relate to the stability and load carrying capacity of the structure from those that do not. Cracking or movement is especially significant if it is recent in origin and is still active.

The major factors affecting the stability of a masonry arch bridge are summarised below:

- Differential settlement across an abutment or pier. This may cause longitudinal cracks along an arch ring, indicating that the arch has broken up into separate rings;

- Movement or settlement of the foundations of an abutment or pier. This may cause lateral cracks across an arch ring and settlement in the deck, indicating that the arch has broken up into separate segments.
- Settlement at the sides of an abutment or pier. This may cause diagonal cracks starting near the side of the arch at the springing and extending to the centre of the arch at the crown.
- Flexibility of the arch ring. This may cause cracks in the spandrel walls near the quarter points.
- Outward movement of the spandrel walls due to the lateral pressure of the fill, particularly if the live loads can travel close to the parapet. This may cause longitudinal cracking near the edge of the arch.
- Movement of the wingwalls. This may cause cracking and if adjacent to the deck, loss of the surface material

C7-2 Causes of deterioration

The principal causes of deterioration of masonry structures are:

C7-2.1 Ground movement

All masonry structures are vulnerable to cracking from excessive or differential settlement of the foundations. Ground movements may be caused by:

- Variations in soil compressibility due to changes in groundwater levels or site drainage
- Loss of support due to soil erosion
- Swallow holes, mining subsidence or local vibration effects
- Desiccation of shrinkable clays in dry summers due to adjacent trees
- Rehydration of clays following removal of adjacent trees
- Inadequately compacted fill

If significant foundation movement is suspected, it should be referred to a geotechnical engineer for investigation and advice.

C7-2.2 Thermal movement

Fluctuations in temperature may set up stresses in walls restrained at both ends that are sufficient to cause fracture or distortion of the wall. Movement of bridge superstructure elements, e.g. steel girders, concrete beams and slabs can crack the supporting masonry if sliding bearings have not been provided or if the bearings have corroded and "seized".

C7-2.3 Sulphate attack

Cement-based mortars may be attacked by sulphates derived from clay bricks themselves or from sulphate bearing soils. The attack is gradual and occurs only when the masonry remains wet for long periods. Sulphate attack causes expansion of the jointing mortar that may result in spalling of the brick edges, progressive cracking and deformation of the masonry.

If the wall is rendered, sulphate attack will produce horizontal cracking of the rendering. Later, the rendering becomes hollow and more badly cracked and portions may fall away.

Suspected sulphate attack can be confirmed by chemical tests.

C7-2.4 Expansion on wetting

Clay bricks undergo a slight "growth" after leaving the kiln, resulting in expansion of brickwork. In load bearing walls vertical expansion rarely causes problems, but horizontal expansion can give rise to movement and vertical cracking of brickwork. These cracks often occur at or near the quoins of short returns, at setbacks or changes in height or thickness of walls.

The bulk of the expansion and movement will occur quite early in the life of the structure but can continue for up to 20 years.

Another cause of outward movement of masonry and cracking is the swelling of materials used as hardcore or filling behind the walls. Some shales and clays used as fill may swell when wetted and cause outward movement of walls.

C7-2.5 Corrosion of embedded iron or steel

Corrosion of iron and steel embedded or enclosed in masonry can cause opening of the masonry joints, cracking of the masonry and rust staining. Where a steel beam is in contact with a brick cladding, rusting of the steel may displace the immediately adjacent courses of bricks beyond the main face of the masonry. Rusting of a stanchion similarly covered may produce a vertical crack. Brick pedestals built around steel columns often crack due to rusting of the steel. The rusting of unprotected steel cramps, brackets or reinforcement embedded in the masonry can also cause problems to occur.

C7-2.6 Unsound materials

Occasionally, masonry suffers damage because of unsoundness of the mortar or of the bricks themselves. For example, the presence of imperfectly slaked lime in a mortar can produce effects ranging from minor pitting of the mortar to general expansion with deformation and cracking of the masonry. Similarly, unslaked nodules of lime in clay bricks may cause "blowing" or spalling of brickwork when first wetted.

Weak mortars are often porous; they are damaged by frost and flowing water, become friable and are easily eroded. The weakness may be due to low cement content or to unsuitable sand in the mortar mix, or occasionally it is caused by dirty mixing water.

C7-2.7 Salts

Salts in masonry may be derived from the bricks or stones themselves, from mortar, from soil in contact with the masonry or from sea water.

Salts in clay bricks or mortar produce an efflorescence that although unsightly is usually temporary and harmless to bricks.

Salts dissolved in ground water penetrate masonry from the foundations and backfill by capillary action or leak down from the fill materials above. When water reaches the face of the wall in contact with air, it evaporates away leaving behind the dissolved salts that crystallise just below the surface of the wall. The pressure developed due to crystallisation is sufficient to spall the surface layer of the masonry units. If this process continues unchecked for a length of time, considerable loss of material may occur. The deterioration is more pronounced in lime mortar joints that are more porous than brick or stone, draw more water and therefore fret more easily.

Masonry abutments and piers built in coastal water streams suffer damage by ingress of salts from the sea water. The continuous cycles of wetting and drying in the tidal zone cause disintegration of the masonry surface by formation of salt crystals within its pores and subsequent eroding away of the disintegrated material by tides.

C7-2.8 Abrasion

Water borne abrasive materials may abrade soft masonry. This can be particularly significant if the base flow of the watercourse is permitted to run along the faces of piers, abutments or walls.

C7-2.9 Impact force

Impact damage can be caused to masonry structures by:

- Collision of or glancing blows from motor vehicles against masonry abutments, piers, wingwalls and retaining walls
- Derailment of trains

- Overheight vehicles striking against the intrados of masonry arches
 - Impact of heavy floating logs carried by rapidly flowing streams against a bridge structure
- Impact damage must be investigated immediately to check if the strength and stability of the structure are affected and repairs should be organised accordingly.

C7-2.10 Overloading

Overloading of masonry structures may occur due to the following:

- Road and rail vehicles above the legal weight limit
- Increased road traffic or train loads since the original construction
- Excessive build-up of road metal or ballast on the deck of a bridge
- Build-up of flood debris against a structure
- Excessive hydraulic pressures behind abutments and retaining walls due to lack of drainage
- Use of heavy machinery on a station platform causing failure of the platform wall

Overloading can cause cracking of the structure. Excessive overloads may even cause collapse.

C7-2.11 Vegetation and marine organisms

The fill retained between spandrel walls over arch structures and behind abutments, retaining walls and wing walls often contains sufficient water and nutrients to support a large mass of vegetation. While growth of grass and small bush may help to stabilise backfill slopes and prevent erosion, large trees may damage the masonry by exerting pressure through their trunks and roots.

Roots and stems growing in crevices and joints exert a wedging force that can prise open and dislodge the masonry. Lichen and ivy can chemically attack the surface while attaching themselves to masonry.

Rock boring molluscs can attack masonry by means of chemical secretions.

C7-3 Strength and stability

It is not always easy to judge from appearance how far the strength and stability of the structure will be affected by damage or deterioration. In general, simple cracks, even wide cracks, may not be too serious provided the masonry is not distorted or too far out of plumb.

If it is established that the strength and stability are not affected, repairs can safely be delayed until movement has ceased and the weather is favourable.

If in doubt, engineering advice should be sought for assessing the strength and stability. Engineering advice should also be sought before undertaking any major repairs and for supporting and strengthening severely damaged structures.

C7-3.1 Strength of arch bridges

For assessing the strength of masonry arch bridges, reference should be made to the following publications:

- Department of Transport (UK) Departmental Advice Note BA 16/84 - "The Assessment of Highway Bridges and Structures"
- Department of Transport (UK) Departmental Standard BD 21/84 - "The Assessment of Highway Bridges and Structures"

C7-4 Assessment of deterioration

IMPORTANT

The most important matter in any assessment of damage is to establish that the strength and stability of the structure are not affected. Where safety of a structure is in question, professional advice should be sought immediately for protecting the structure as well as its users against further damage, collapse or injury.

C7-4.1 General

Before any repairs are initiated, it is essential to identify the cause and extent of the damage and whether or not the cause is still active.

C7-4.2 Assessment procedure

- Before proceeding, assess if detailed examination of the damage would require track closure, pedestrian and traffic restrictions, assistance from police and utility authorities, worksite protection, special equipment for access and any special safety measures
- If the damage is old and has been repaired before, study previous investigation and repair reports if available. Examine the condition of the past repairs and determine if they have been successful or if the deterioration is growing worse
- Carry out a visual inspection of the structure. If necessary, use hand magnifiers, binoculars and telephoto photography to assess and record the type and extent of damage or deterioration
- Assess the possible causes for deterioration or damage (refer to C7-2 "Causes of Deterioration")
- Map the location, direction and extent of cracks. Examine if the cracks zig-zag through the joints or run through the masonry units. Measure the width of cracks
- Ascertain if the cracks are moving by fixing strain gauges across them at suitable locations. Cracks that are due to applied load will move immediately when the load is changed (e.g. under traffic passing over a bridge). Cracks may also move under temperature variations. Measurements should be made with and without traffic loads at varying temperature conditions to establish whether a crack is live or not
- Check if any distortions have occurred in the load bearing elements such as walls, abutments and piers as well as retaining structures
 - Check verticality with a plumb line and measure any out-of-plumb distortion
 - Check bulging in both horizontal and vertical directions using a 3 metre long straight edge
- Assess the effect of these distortions on the strength and stability of the structure using appropriate structural analysis methods
- Examine the bearing areas under steel, concrete and timber beams and concrete slabs supported on top of the masonry walls, abutments and piers. Examine the extent of corrosion of steel bearings or deterioration of other slip joints provided. Look for cracks in the masonry immediately under the bearings. Also ensure that there has not been a significant loss of support to the superstructure
- If there are signs of efflorescence, leaching and percolation of water through masonry, assess the deterioration of masonry units and jointing mortar by fretting. Investigate if the earth fill behind abutments and retaining walls and fills between spandrel walls of the arches are properly drained. Check if the weep holes (if provided) are functioning. Dry weep holes indicate they may be blocked



- Examine the substructures that are located in waterways for damage by abrasion, salts or marine organisms above the low tide mark. Check if there is a significant loss of section (more than 15%) and jointing mortar
- Check the structure for deterioration by growth of vegetation in joints, near foundations and behind retaining walls and abutments.

Chapter 8 Assessment of timber structures

C8-1 Deterioration modes in timber structures

C8-1.1 General

The main indicator of deterioration of timber members is the section loss caused by one or more outside agents.

Timber generally does not deteriorate significantly in service without being attacked by some outside agent. This can take the form of a biological attack or non-biological deterioration.

In general, timber deteriorates in one of five ways:

- Fungi and insect attack (termites or borers)
- Weathering at exposed surfaces
- Decay or rot
- Fire
- Mechanical damage from impact

Of the above categories, decay and insect attack usually cause deterioration inside a member and are therefore the most difficult to accurately measure.

The most common defects that occur in timber structures and therefore require checking during examination are as follows:

- Decay
- Troughing or bulging (indicates internal decay)
- Insect infestation
- Weathering - abrasion, cracks, shakes, checks and splits
- Loss of section due to fire
- Vehicle impact damage
- Crushing
- Loose or missing bolts/connections
- Corroded connections

C8-2 Causes of deterioration

C8-2.1 Biological attack

Timber structures and their individual components are vulnerable to biological attack from fungi, termites and marine borers.

Fungal attack is the main cause of deterioration in timber bridges, however certain conditions are necessary for the development of fungi. These include:

- A temperature range suitable to their life cycle
- A moisture content suitable for their development
- An adequate oxygen supply
- A food supply on which they can grow (i.e. timber)

Fungi attacks both sapwood and heartwood (under favourable conditions) causing breakdown of the wood substance and this is known as decay.

There are several types of insects in Australia that attack timber, however the termite is the only one that attacks seasoned heartwood. Termites work along the grain eating out large runways. In the early stages much sound wood is left between the runways, however in the long term only the thin outer layer of wood may remain.

Marine borers are of several types and the danger from these is dependent upon geography and water salinity. Although borers attack different sections of piles (depending upon the type of borer), the simple rule is to protect from below mud line to above high water level. Borers may make only a few small holes on the surface and yet the pile interior may be practically eaten away.

C8-2.2 Non-biological deterioration

Timber is also vulnerable to non-biological deterioration from weathering, abrasion, fire, impact and overload.

Weathering is the most common form of non-biological deterioration. Exposure to the elements can lead to continual dimensional changes in the wood from repeated wetting/drying, or it may result in drying and shrinkage. These processes can lead to cracks, shakes, checks, splits (particularly at member ends or at bolted connections) or warping and loose connections.

Impact and overloading may result in damage to members such as shattered or injured timber, sagging or buckled members, crushing or longitudinal cracking. The action of vehicles passing over decking can cause abrasion and subsequent loss of section.

C8-3 Assessment of deterioration

IMPORTANT

The most important matter in any assessment of damage is to establish if the strength and stability of the structure are adversely affected. Where safety of a structure is in question, professional advice should be sought immediately for protecting the structure as well as its users against further damage, collapse or injury.

C8-3.1 General

To successfully repair a deteriorated timber structure it is essential to identify the cause, extent and rate of deterioration and whether or not the cause is still active.

A step by step procedure for assessing deterioration is given below. This procedure includes a number of simple tests that can be easily carried out on site. It is not essential to carry out all the tests and judgement should be used in applying the tests according to the severity of the problem at hand.

It is also recognised that resources for carrying out these tests may not be available and detailed investigation may have to be entrusted to specialist firms or consulting engineers who have appropriate expertise to establish the causes of deterioration and advise on what repair action should be taken.

C8-3.2 Assessment procedures

- Before proceeding, assess if detailed examination of the damage or deterioration will require track closure, pedestrian and traffic restrictions, assistance from police and utility authorities, worksite protection, special equipment (ladders, cherry pickers) and any special safety measures
- Study previous investigation and repair reports available, if any. Examine the condition of the past repairs to determine whether they have been successful or if the deterioration is growing worse
- Review the detailed examination report
- Carry out a visual inspection of the structure and confirm existence and extent of main defects. If possible, ascertain the obvious causes of deterioration such as termites, environmental conditions, accidental damage etc.

- Hammer test and bore and probe members as necessary
- Observe members under load and note any excessive movement in members or fastenings. If necessary, conduct deflection tests on girders

C8-3.3 Other detection methods

Listed below are additional tests that require special equipment and significant skills and experience to obtain usable results. Such testing methods would have to be undertaken by specialist personnel skilled in the field of diagnostic testing.

- Shigometer
- Ultrasonic
- X-Rays

C8-3.4 Timber repair issues

The most important issue in the repair of any structure is to ensure adequate strength and stability at all times. In such cases the strength and stability of the structure during repairs should be checked by a structural engineer before commencing repairs. Also, load restrictions should be applied and the structure temporarily supported as necessary.

The repair of timber structures requires knowledge of the following issues:

- Types of defects which can occur due to deterioration, e.g. pipes, troughs, crushing, loose bolts
- Causes of deterioration, e.g. biological attack, impact damage, foundation scouring
- Test methods for assessing the severity of deterioration
- Ability to interpret test results
- Selection of appropriate repair materials
- Selection of appropriate repair procedure

C8-3.5 Capacity reduction

Assess the capacity reduction of girders and corbels using Figure 13 below.

The figure shows the capacity (bending & shear) of piped timber beams as a percentage of a solid 300 x 300 timber beam.

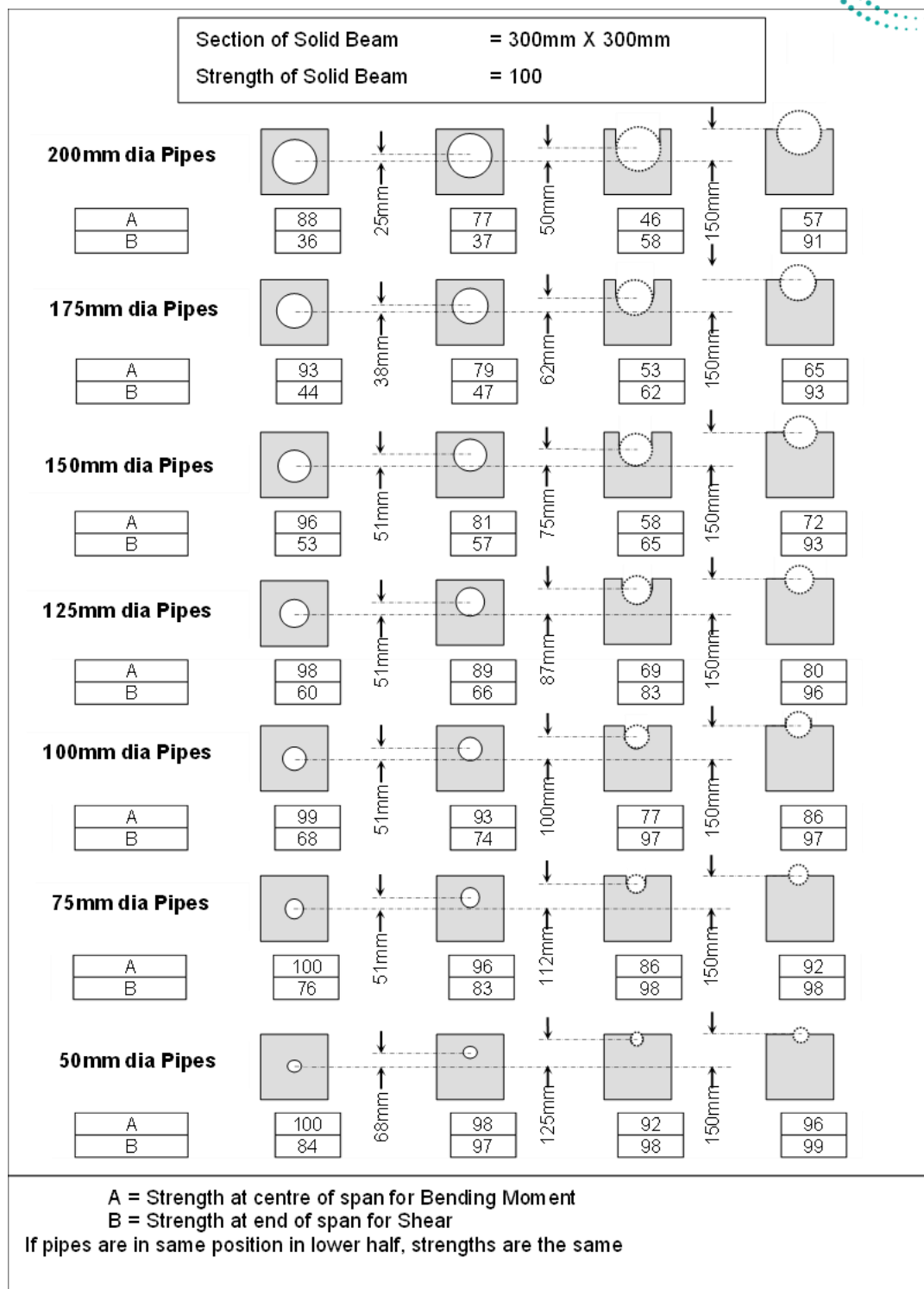


Figure 13 - Comparison of strength of solid and piped timber beams

Chapter 9 Assessment of timber poles

C9-1 Introduction

When timber poles have been inspected in accordance with CRN CM 302 and assessed by the examiner to be in other than “serviceable condition, they are forwarded to the Structures Superintendent to arrange Risk Assessment and Final Assessment.

Note: Risk assessment and Final Assessment may only be undertaken by competent Structures Assessors who have been granted Engineering Authority by the Principal Track and Civil Engineer.

C9-2 Risk Assessment

Undertake a risk assessment after a pole has been categorised as “conditionally serviceable” or “condemned”.

The determination of the risk classification, and the actions required for each risk class, are set out in Table 1 for supported poles and Table 2 for unsupported poles. Depending on the extent and magnitude of other pole degradation indicators (see Chapter 8), a pole may be assigned a higher risk category after the application of engineering assurance processes. Document the outcome of this assessment process.

Depending on the costs of pole replacement, and the costs and resource availability for additional maintenance, it may be appropriate to have a condemned or conditionally serviceable pole replaced sooner than the allowable time specified in Table 1 and Table 2.

Risk	Serviceability	Location of Pole	Action	Defect / Repair categories
Very high	Dangerous with $t < 20\text{mm}$, and assessed to be at risk of imminent failure	All	Replace pole within 48 hours; Carry out other actions set out in CRN CM 302 Section C4-5.1 Table 5	A / Within 48 hrs
High	Condemned with $t < 18\text{mm}$	All	Replace pole within 1 month; Inspect pole weekly until pole is replaced	B / Rm1
High	Condemned with $18\text{ mm} \leq t < 20\text{ mm}$	Pole may fail across railway line or roadway upon failure	Replace pole within 1 month; Inspect pole weekly until pole is replaced	B / Rm1
Medium	Condemned with $18\text{ mm} \leq t < 20\text{ mm}$	All except as noted above	Replace pole within 6 month; Inspect pole every 2 months until pole is replaced	C / Rm6
Low	Conditionally serviceable with $20\text{ mm} < t \leq 30\text{ mm}$	All	Carry out pole base examination every 3 years	D
Very low	Conditionally serviceable with $30\text{ mm} < 40\text{ mm}$	All	Maintain per Technical Maintenance Plan	E

Table 1 - Supported pole risk categories for conditionally serviceable and condemned

Note: t = least thickness of sound timber in the wall of the annulus of the pole

Risk	Serviceability	Location of Pole	Action	Defect / Repair categories
Very high	Dangerous	All	Replace pole within 48 hours; Carry out other actions set out in CRN CM 302 Section C4-5.1 Table 5	A / Within 48 hrs
Very high	Condemned	Pole may fail across railway line or roadway upon failure	Replace pole within 48 hours; Carry out other actions set out in CRN CM 302 Section C4-5.1 Table 5	A / Within 48 hrs
High	Condemned	Pole is located outside of rail corridor and not in remote area	Replace pole within 1 month; Inspect pole weekly until pole is replaced	B / Rm1
High	Condemned	Pole may fail across railway line or roadway upon failure	Replace pole within 1 month; Inspect pole weekly until pole is replaced	B / Rm1
Medium	Condemned	Pole may fail across railway line or roadway upon failure	Replace pole within 6 month; Inspect pole every 2 months until pole is replaced	C / Rm6
Medium	Conditionally Serviceable	All locations other than those in the High risk areas noted above	Replace pole within 6 month; Inspect pole every 2 months until pole is replaced	C / Rm6
Low	Conditionally Serviceable	Pole may fail across railway line or roadway upon failure	Carry out pole base examination every 3 years	D
Low	Conditionally Serviceable	All locations other than those in the Medium risk areas noted above	Carry out pole base examination every 3 years	D
Very low	Conditionally Serviceable	All except as noted above	Maintain per Technical Maintenance Plan	E

Table 2 - Unsupported pole risk categories for conditionally serviceable and condemned

C9-3 Final assessment of unsupported poles

C9-3.1 Introduction

If, during the examination undertaken in accordance with CRN CM 302, it is determined that further assessment is required, undertake the following procedure to determine the risk ranking, then by using Table 2 decide the Defect/ Repair categories.

The following information from the examiner is required for the detailed assessment:

- Pole diameter at the defective section level “D”
- Pipe diameter “d”
- Least thickness of sound timber in the wall of the annulus of the pole “t”
- The height of the defective section “H”, from ground level
- The total pole height “L” from ground level

C9-3.2 Assessment procedure

- Find the original section modulus “Z” at the defective section.

$$Z = \frac{\pi D^3}{32}$$

- Find the defective section Modulus “Z_d” at the defective section.

$$Z_d = \frac{I_d}{y'}$$

Where $y' = \frac{(A_1 y_1 - A_2 y_2)}{(A_1 - A_2)}$

$$A_1 = \frac{\pi D^2}{4}$$

$$A_2 = \frac{\pi d^2}{4}$$

$$y_1 = \frac{D}{2}$$

$$y_2 = t + d/2$$

And $I_d = I_1 - I_2$

Where $I_1 = \frac{\pi D^4}{64} + \frac{\pi D^2}{4} (y' - y_1)^2$

$$I_2 = \frac{\pi d^4}{64} + \frac{\pi d^2}{4} (y' - y_2)^2$$

- Find the safety factor FS.

$$FS = \frac{Z_d}{Z}$$

- Using Table 3, determine the Risk ranking.

H	FS	Risk Ranking
H ≤ L/4	0.9 ≤ FS < 1.0	Very Low
	0.8 ≤ FS < 0.9	Low
	0.7 ≤ FS < 0.8	Medium
	0.6 ≤ FS < 0.7	High
	FS < 0.6	Very High
L/4 < H ≤ L/2	0.8 ≤ FS < 1.0	Very Low
	0.7 ≤ FS < 0.8	Low
	0.6 ≤ FS < 0.7	Medium
	0.5 ≤ FS < 0.6	High
	FS < 0.5	Very High
H > L/2	0.7 ≤ FS < 1.0	Very Low
	0.6 ≤ FS < 0.7	Low
	0.5 ≤ FS < 0.6	Medium
	0.4 ≤ FS < 0.5	High
	FS < 0.4	Very High

Table 3 - Risk ranking based on safety factor and defect location

- Using Table 2, determine the defect and repair categories.

Example 1

An examiner found a 75mm pipe at 1.5m height from ground level in a 5m high pole of 250mm diameter and the least sound thickness of the pole is 87.5 mm.

Complete the detail assessment and find the risk ranking and the correlated Defect/Repair categories.

$$D = 250 \text{ mm}$$

$$d = 75 \text{ mm}$$

$$t = 87.5 \text{ mm}$$

$$H = 5 \text{ m}$$

$$L = 1.5 \text{ m}$$

1. Determine the original section modulus Z .

$$Z = \frac{\pi D^3}{32} = \frac{3.14 * 250^3}{32} = 1,533,203 \text{ mm}^3$$

2. Second, you should find the defective section modulus Z_d .

$$A_1 = \frac{\pi D^2}{4} = \frac{3.14 * 250^2}{4} = 49,062.5 \text{ mm}^2$$

$$A_2 = \frac{\pi d^2}{4} = \frac{3.14 * 75^2}{4} = 4,415.6 \text{ mm}^2$$

$$y_1 = \frac{D}{2} = \frac{250}{2} = 125 \text{ mm}$$

$$y_2 = t + \frac{d}{2} = 87.5 + \frac{75}{2} = 125 \text{ mm}$$

$$y' = \frac{(A_1 y_1 - A_2 y_2)}{(A_1 - A_2)} = \frac{(49,062.5 * 125 - 4,415.6 * 125)}{(49,062.5 - 4,415.6)} = 125 \text{ mm}$$

$$I_1 = \frac{\pi D^4}{64} + \frac{\pi D^2}{4} (y' - y_1) = \frac{3.14 * 250^4}{64} + \frac{3.14 * 250^2}{4} (125 - 125)^2 = 191,650,390 \text{ mm}^4$$

$$I_2 = \frac{\pi d^4}{64} + \frac{\pi d^2}{4} (y' - y_2) = \frac{3.14 * 75^4}{64} + \frac{3.14 * 75^2}{4} (125 - 125)^2 = 1,552,368 \text{ mm}^4$$

$$I_d = I_1 - I_2 = 191,650,390 - 1,552,368 = 190,098,021 \text{ mm}^4$$

$$Z_d = \frac{I_d}{y'} = \frac{190,098,021}{125} = 1,520,784 \text{ mm}^3$$

3. Find the safety factor FS

$$FS = \frac{Z_d}{Z} = \frac{1,520,784}{1,533,203} = 0.99$$

4. From Table 3, with $H = 1.5$ and $L = 5$, thus $(L/4 < H \leq L/2)$

Risk Ranking = Very Low

5. From Table 2,

Defect category E. Maintain per TMP

Example 2

An examiner found 100mm pipe at 1.0m height from ground level in a 5m height pole of 200mm diameter and the least sound thickness of the pole is 25 mm.

Complete the detail assessment and find the risk ranking and the correlated Defect/Repair categories.

If the pole were to fall it may fall across a railway track.

$D = 200 \text{ mm}$

$d = 100 \text{ mm}$

$t = 25 \text{ mm}$

$H = 5 \text{ m}$

$L = 1 \text{ m}$

1. Determine the original section modulus Z ,

$$Z = \frac{\pi D^3}{32} = \frac{3.14 * 200^3}{32} = 785,000 \text{ mm}^3$$

2. Find the defective section modulus Z_d ,

$$A_1 = \frac{\pi D^2}{4} = \frac{3.14 * 200^2}{4} = 31,400 \text{ mm}^2$$

$$A_2 = \frac{\pi d^2}{4} = \frac{3.14 * 100^2}{4} = 7850 \text{ mm}^2$$

$$y_1 = \frac{D}{2} = \frac{200}{2} = 100 \text{ mm}$$

$$y_2 = t + d/2 = 25 + 100/2 = 75 \text{ mm}$$

$$y' = \frac{(A_1 y_1 - A_2 y_2)}{(A_1 - A_2)} = \frac{(31400 * 100 - 7850 * 75)}{(31400 - 7850)} = 108.3 \text{ mm}$$

$$I_1 = \frac{\pi D^4}{64} + \frac{\pi D^2}{4} (y' - y_1)^2 = \frac{3.14 * 200^4}{64} + \frac{3.14 * 200^2}{4} (108.3 - 100)^2 = 80,663,146 \text{ mm}^4$$

$$I_2 = \frac{\pi d^4}{64} + \frac{\pi d^2}{4} (y' - y_2)^2 = \frac{3.14 * 100^4}{64} + \frac{3.14 * 100^2}{4} (108.3 - 75)^2 = 13,611,036 \text{ mm}^4$$

$$I_d = I_1 - I_2 = 80,663,146 - 13,611,036 = 67,052,110 \text{ mm}^4$$

$$Z_d = \frac{I_d}{y'} = \frac{67,052,110}{108.3} = 619,133 \text{ mm}^3$$

3. Find the safety factor FS

$$FS = \frac{Z_d}{Z} = \frac{619,133}{785,000} = 0.78$$

4. From Table 3, with $H = 1.0$ and $L = 5$, thus ($H \leq L/4$)

Risk Ranking = Medium

5. From Table 2,

Defect category C, Repair Category Rm6, replace pole within 6 months, inspect pole every 2 months until pole is replaced.

Chapter 10 Selecting repair actions

C10-1 Introduction

The primary outcome of repairing a structure is an extension to its service life. It is most important that the repair actions selected satisfy this outcome at a cost commensurate with the benefits derived. Inappropriate repair action may actually reduce the life expectancy of a structure. It is also possible that money spent on extensive and costly repairs will not extend the life of the structure significantly and would be better put towards a new structure.

Where possible, life cycle costing principles should be applied to a structure to determine the most appropriate course of action (maintenance, rehabilitation or replacement). The objective should be to provide a serviceable structure at minimum annual cost.

If a life cycle costing analysis cannot be carried out, the next best thing is to determine the appropriate repair action by applying a process of logical assessment to the structure as a whole.

Section C10-2 below presents a series of questions to be considered in the process of selecting appropriate repair actions.

C10-2 Process for selecting repair action

In determining the appropriate repair action to be carried out on a structure, the following questions should be answered:

- C10-2.1 What is the nature, severity, location and extent of each defect?
- To enable a considered logical assessment of repair actions, full data about each defect must be known and recorded from site inspections, measurements and testing. Section C10-3 deals with testing for and measurement of defects.

C10-2.2 Is a repair to restore full strength necessary?

A structure may have been designed with a load capacity greater than currently required or envisaged. In this case the strength reduction caused by the defect may be acceptable.

Even if the full design load capacity is needed, a small overstress - say 10% - resulting from the defect can usually be accepted.

Some elements may be able to tolerate considerable deterioration before repair or strengthening is needed. For example, the full flange area of a rolled girder may not be necessary near the end of a span where the bending moment is small.

The necessity for and extent of a repair should be determined, where appropriate, by a full engineering assessment. The time and effort required for such a task may be repaid by minimising the extent of repairs needed or by determining that the repairs are not structurally necessary. Section C10-4 discusses the role of engineering assessments.

If it is found that repair or strengthening is not needed, then the only action required is to protect the structure from further deterioration.

The serviceability requirements, particularly fatigue in steel, should be considered in a similar manner to strength.

Note that a repair may be warranted to improve the appearance of a structure, even if it is not structurally essential.

C10-2.3 Is a standard repair procedure available for each defect?

It is not normally appropriate to repair one defect in a structure, if other significant defects are left unrepaired because satisfactory or cost effective repair procedures are not available.

It should be confirmed that a standard repair procedure proposed is actually appropriate to the defect. The engineering discussion accompanying each repair procedure will describe where the procedure should and should not be used.

C10-2.4 What is the total cost of repairing all defects?

The total cost of repairing all defects in a structure, even if only determined crudely, should be estimated and compared with the expected benefits. The benefits are usually either extending the life of the structure or eliminating an immediate dangerous situation. The costs of any track possessions etc. that are required should be included.

If the costs of repair are substantial and the repairs are not urgent for safety reasons, it may be better to replace the structure and to divert the repair funds to more appropriate cases.

The initial cost estimate will be a factor in deciding if a detailed engineering assessment is worthwhile. When the total cost is high - say more than 5% to 10% of the structure replacement cost - a detailed engineering assessment needs to be carried out. The extent of repair is likely to be optimised as are sizes and numbers of connections for strengthening elements.

C10-2.5 How significant will the repair be in extending the life of the structure?

The proposed repair may not provide value for money if the life of the structure is not extended because of other factors.

Most steel structures have a finite life, governed by fatigue of the steelwork. It may not be appropriate to spend large amounts of money on repairs if the structure is near the end of its predicted fatigue life. Replacement is probably a better option.

In concrete structures, defects such as corrosion of reinforcement may not be apparent at the present time, but the effects may show up in the form of concrete cracking and spalling in the future. It may not be appropriate to spend money on repairing some localised defects if much more extensive defects are likely to show up in the near future. Investigation by specialists into the complete structure may be warranted, prior to undertaking costly repairs of concrete.

As another example, the benefits of repairing corroded bottom flanges of jack arch bridges may not be certain because the condition of the remainder of the steel section is usually not known. The effort in repairing the bottom flange would be wasted if the top flange and web were also severely corroded.

C10-2.6 Can the cost of the repair be justified on the basis of benefits derived?

The answer to this question will normally be evident from the answers to the preceding questions. For some structures, there may be additional factors that influence the decision on whether to implement a particular repair.

Taking advantage of planned track possessions may often be a significant factor in deciding whether to implement a repair or not. In many repairs, particularly smaller ones, the cost of the track possession is the major component. If the repair can be carried out under a possession provided for other reasons, the actual cost of the repair drops significantly for the same benefit. A repair that could not normally be justified on a cost/benefit basis may become cost effective as a result.

C10-2.7 Is a detailed engineering assessment needed?

Situations in which an engineering assessment may be warranted are described above. It should be noted however that a longer lead time to implement a repair will usually be required if an engineering assessment is to be undertaken.

C10-2.8 Has partial replacement of the structure been considered?

As an alternative to repairing defects or strengthening structures, consideration should be given to partial replacement of the structure. This may involve replacement of individual structural members

(e.g. damaged bracing members, stringers, cross girders, concrete components etc.) up to complete replacement of major sub-components of the structure.

The decision will be based on time, cost and effectiveness of the repair compared to that of a partial replacement. If the effectiveness or life expectancy of repairs or strengthening is limited, then complete replacement of a member or sub-component may be a better option, particularly if the cause of the defect can be eliminated at the same time.

C10-2.9 Will the repair have any adverse effects on the structure?

Any adverse effects that the repair may have on the performance of the structure should be considered, e.g.:

- Will traffic clearances be reduced?
- Will the structure become more vulnerable to damage or deterioration?
- Will other potential defects be hidden by the repair?

C10-3 Testing for and measuring defects

C10-3.1 General

As stated in Section C10-2 above, it is necessary to determine the nature, severity, extent and location of defects to determine appropriate repair actions. Most defects are initially detected by visual inspection. The severity, extent and precise location are determined by subsequent measurements and tests.

C10-3.2 Measuring section loss in steel

In steel structures, corrosion of steel leading to a loss of section is a common defect. Measurements are to be taken to determine the thickness of remaining sound steel for comparison with the original thickness. All loose rust and corrosion product must be removed at the point of measurement to allow an accurate reading. Vernier callipers or preferably a micrometer should be used to obtain accurate measurements to at least an accuracy of $\pm 0.25\text{mm}$.

The number and location of measurements required will depend on the defect under consideration. Usually the minimum requirement is to measure the remaining cross-sectional area of an element (flange, web, stiffener) at the location of the greatest corrosion loss and location of greatest stress.

C10-3.3 Testing for and measuring cracks in steel

Although many cracks in steel can be detected by visual inspection, it is usually necessary to use techniques such as magnetic particle testing to determine their exact extent. This is particularly important in mapping fatigue cracks where the end point must be found.

Examination for cracks and other defects in new welds will require the use of non-destructive techniques such as ultrasonic or X-ray examination.

Further discussion of non-destructive methods is beyond the scope of this manual, but reference should be made to the appropriate Australian Standards.

C10-3.4 Observation under load

The severity of some defects is best determined by observing the structure during the passage of a heavy load. Loose rivets and bolts may be detected by this means. The behaviour of girder bearing plates and bed plates under load will aid in determining the necessity for repair or replacement.

C10-3.5 Detecting defects in concrete

While most defects in concrete such as cracking, spalling and rust staining are detected by visual examination, the likelihood of further deterioration can be determined using specialist testing techniques. Tests to determine the extent of carbonation, chloride penetration, cover to reinforcing etc. can be carried out to determine the life expectancy of concrete. As outlined above, this

information is useful in planning the long term maintenance and repair strategy for a concrete structure.

Chapter 6 of this manual deals with assessment of concrete, including test methods.

C10-4 Engineering assessments

As stated above, a detailed engineering assessment is normally necessary only when the estimated total cost of repairs is more than 5% to 10% of the replacement cost of the structure. In such cases, the cost of the investigation could be more than saved in the reduced extent of repair.

The purpose of the engineering assessment is to determine the appropriateness of the proposed repair and the repair details. The assessment is to be based on:

- The defects as measured and recorded during inspection
- The structural drawings where available
- The load carrying requirements

IMPORTANT

The aims of an engineering assessment are to:

- Determine the effect of the defects on the strength of the structure
- Determine the effects of the defects on the serviceability of the structure, including its fatigue performance
- Determine or confirm that each proposed standard repair is both necessary and suitable to address the defect
- Determine the extent of all the required repairs
- Determine the engineering details of the repairs where required – e.g. plate sizes, fastener sizes and types, new connection details etc.

CRN Engineering Manual CRN CM 303 “Structures Repair” provides some detailed assessment guidelines for specific repair procedures. Refer to CRN CM 303 when determining the appropriate repair action.

C10-5 Avoiding recurrence of defects

In conjunction with carrying out the repairs, action should be taken where possible to avoid a recurrence of the defects. This has been considered in devising repair procedures. To protect against recurrence of the defect, the original cause must be identified and eliminated where possible.

The causes of defects are often built into the structures and are difficult to eliminate. For example, deterioration of concrete due to insufficient cover to the reinforcement or poor concrete compaction cannot be easily addressed. Details in steel structures which are prone to corrosion because of collection of water and dirt cannot be readily eliminated without changing the structure significantly.

Often the only effective means of avoiding recurrence of the defect is preventive maintenance of the structure to remove dirt and debris and to maintain the integrity of the paint system.

Where possible, galvanised strengthening or replacement elements should be used to ensure long term corrosion protection with minimum maintenance requirements. The standard repair procedures specify galvanised elements where possible.

C10-6 Steel repairs

C10-6.1 Steel repair issues

C10-6.1.1 Methods of connection

In addressing defects in steel structures resulting from steel corrosion, it is clearly not possible to reinstate the steel to its original condition. Similarly, restoring physically damaged steel to its original condition is often difficult. Most repairs therefore involve fitting new steel elements to compensate for the reduction in strength or serviceability caused by the section loss.

The two standard methods of connecting new elements are welding and mechanical fastenings (bolts etc.). Of the two, mechanical fastenings have less potential problems although welding is usually easier and cheaper.

Mechanical fasteners have been adopted as the standard method in all steel repair procedures.

IMPORTANT

The two main potential problems with field welded connections are:

- Satisfactory welds may be difficult or impossible to achieve in steels of older structures because of their metallurgical properties
- the fatigue life of a structure subject to cyclic loading may be adversely affected by welding. The effect may be severe

The two main potential problems with field welded connections are:

- Satisfactory welds may be difficult or impossible to achieve in steels of older structures because of their metallurgical properties
- The fatigue life of a structure subject to cyclic loading may be adversely affected by welding. The effect may be severe

Connection by welding can only be permitted if satisfactory welds are proven to be achievable and any effects on fatigue life are acceptable. Most standard repairs, detailed with bolted connections, can be readily adapted for welded connections.

C10-6.1.2 Weldability of steel

While modern steels can be readily repaired by welding using appropriate procedures, the steels found in older steel structures are often considered "not weldable" or very difficult to weld because of their metallurgical properties. The high sulphur and phosphorous contents largely contribute to the difficulty in welding.

Unfortunately, it is not possible to classify steels as weldable or "not weldable" based on their age. From the 1940's to the 1960's, some imported steels were from plants producing steels designed to be weldable, while others were from plants producing unweldable steels.

IMPORTANT

It is essential that, where a welded repair is proposed on steels of unknown weldability, the steels be tested by a certified laboratory to determine whether or not they are weldable by normal welding procedures.

It is sometimes technically possible to achieve satisfactory welds in older style (non-weldable) steels, but only by careful adherence to particular welding procedures designed for such steels. A high level of operator skill and good welding conditions are required. Unfortunately, most steel repairs must be carried out in conditions which are anything but conducive to good welding.

Section C10-10 provides details of techniques for welding old steel, so-called "non-weldable" steels. The information is intended for accredited welders/boilermakers already skilled in normal welding techniques. If such welding is attempted, it is essential to carry out test welds to confirm

that acceptable results can be achieved under site conditions. It is also necessary to carry out non-destructive testing of the completed welds to confirm their integrity.

Careful consideration must also be given to the ramifications of a failed attempt at a welded repair on "non-weldable" steels. The cost of undoing the damage may be significantly greater than if a bolted connection repair had been carried out.

C10-6.1.3 *Effect on fatigue life*

As in the design of new steel structures, the details of welded connections used in steel repairs may have an effect on the fatigue life of a structure that is subject to cyclic loading. Although the remaining life of a steel structure that has been in service for many years is not expected to be as great as a new structure, it is best to avoid any repair actions that would adversely affect fatigue life. This is particularly so if viable bolted connection alternatives are available.

IMPORTANT

Properly maintained riveted girder bridges with low stress levels may never fail by fatigue. Repairs involving welded connections on such bridges often set a finite fatigue life and so should be avoided.

If a welded repair is proposed, it should always be accompanied by an engineering assessment of the effect on the fatigue life of the component. The AREA Manual for Railway Engineering should be used to assess the effects. The proposed welded repair should only be implemented if the effects on fatigue life are acceptable.

C10-6.1.4 *Redundancy of riveted bridge girders*

Riveted bridge girders comprising multiple elements for flanges, have beneficial redundancy that should not be removed by the repair process. Redundancy means that if one of the plate or angle elements of a flange fails, say by a fatigue crack, the whole girder will not immediately fail as the fatigue crack cannot propagate to other elements. Inspection can detect the failed component for repair prior to complete failure of the girder. The stress levels in the uncracked elements obviously increase, causing overstress and reducing the overall fatigue life, but the situation is better than in welded or rolled girders where a crack in a flange can quickly propagate all the way through the section, leading to collapse.

IMPORTANT

The elements of riveted girder flanges must not be joined by welding as this would create paths to allow fatigue cracks to propagate from one element to another.

Redundancy is of particular benefit in bridges of a considerable age because it provides a degree of protection against sudden failure. The redundancy must be preserved.

C10-6.2 **Selecting the appropriate repair procedure**

Earlier sections of this chapter deal with selecting the appropriate actions or strategies for the repair of a structure, looking at it as a whole.

Because of the generic nature of some of the repair procedures given, it is also necessary to ensure that the procedure proposed is appropriate and applicable for the particular defect. Careful engineering assessment of the defects, on a case by case basis, is strongly recommended to assist in ensuring that the appropriate repair procedure is selected.

Discussion of the engineering considerations of each repair is given within the procedure. Where appropriate, the discussion includes guidelines for determining the necessity for the repair based on the severity of the defect.

C10-6.3 **Repair materials**

In the repair procedures, repair materials are referred to by their generic name. Specific brand and material names of epoxies, paints etc. are usually avoided as availability may vary from time to time and new, superior materials may become more appropriate. Lists of suitable material types

are given in C10-7, together with the specifications for standard repair materials such as steel and high strength bolts.

Where new steel parts are to be fitted as part of the repair, it is generally recommended that those parts be galvanised. Use of galvanised steel can reduce long term maintenance requirements and minimise the amount of on-site painting required. Where the appearance of galvanised steel is not acceptable and painting over galvanised surfaces is impractical, the steel parts may be painted instead using a high quality paint system. Refer to CRN CM 303 steel repair sub-procedure C4.3 for further information.

CAUTION

Corrosion protection systems such as Denso Tape wrapping and epoxy filling may hide critical defects such as fatigue cracks. Such defects may be difficult to detect during normal inspections and may result in collapse of the structure.

Corrosion protection systems such as these should not be used on fatigue-critical elements unless appropriate procedures to regularly check for and detect cracks are implemented.

C10-7 Repair materials for steel structures

NOTE

Specific product brand names have not been used. The product characteristics mentioned would apply to a number of products and are likely to be available from a number of suppliers.

C10-7.1 Steel

- Unless noted otherwise all steel sections and plates used for strengthening in repair shall be grade 250 to AS 3678 and AS 3679
- Unless noted otherwise all steel sections and plates used in replacement members or elements shall be grade 250 to AS 3678 and AS 3679
- Other steels specified for particular repairs are:
 - Grade 350 to AS 3678 and AS 3679
 - Bisalloy 80

C10-7.2 Fasteners

High strength bolts, nuts and washers shall conform to AS 1252 and shall be galvanised in accordance with AS 1214.

C10-7.3 Epoxies for filling voids

- High strength two part epoxy
- High strength non-sag epoxy for overhead or vertical surfaces

C10-7.4 Epoxies for sealing interfaces

- Single component polyurethane sealant suitable for being painted over
- Non sag sealer for overhead or vertical surfaces
- Epoxy primer

C10-7.5 Patch painting systems

- Two part surface tolerant epoxy mastic
- Polyurethane primer containing zinc phosphate

C10-7.6 Paint systems for galvanised surfaces

- Two part high build epoxy mastic suitable for galvanised surfaces

C10-7.7 Bearing pad materials

- High strength non-shrink cementitious grout
- High strength non-shrink epoxy flow grout

C10-8 Repairing impact damage to steel structures

C10-8.1 Engineering discussion

Steel bridge elements including major components such as girders or truss members and minor components such as bracing members are frequently damaged by vehicle impact. Once the damage has been detected repair of the damaged components must be undertaken.

In order to ensure that the repair method adopted is cost effective and restores the required bridge capacity, the following sequence of actions is required:

- Inspection of damage
- Assessment of damage
- Selection of cost effective repair method

A range of repair methods is currently available. These are shown in Table 5 which compares appropriateness of repair method with type of impact damage.

Many of the available repair methods are excluded for "Fracture Critical Members" (FCM's).

FCM's include tension flanges of girders and truss tension members.

Tension components of steel bridges include all portions of tension members and those portions of flexural members subjected to tension stress. Any attachment having a length in the direction of the tension stress greater than 100mm that is welded to a tension component of a FCM shall be considered part of the tension component and therefore, shall be considered fracture critical.

Broad flange beam (BFB) spans over roadways are subject to a significant risk of fatigue and/or brittle fracture if damaged by road vehicle impact and shall be considered fracture critical.

The majority of cases of impact damage encountered on the CRN network would be to FCM's.

The applicable range of repairs in most cases will therefore be reduced to full or partial replacement of the damaged member, or flame straightening of the member followed by installation of bolted cover plates to fully replace the damaged section.

C10-8.2 Sub-procedures required

Repairs to impact damage can involve all of the sub-procedures defined in Chapter 4 CRN CM 303. Techniques associated with the following procedures from CRN CM 303 may also be appropriate with minor modification, depending on the form of the impact damage:

- Repairing flange corrosion in riveted girders
- Repairing flange corrosion in rolled or welded girders
- Repairing webs with localised corrosion
- Repairing corroded bottom flanges of jack arch bridges
- Repairing intermediate web stiffeners with localised corrosion
- Repairing corrosion of bottom flange bracing connection
- Intercepting fatigue cracks

- Repairing corroded angle columns (temporary support available)
- Repairing corroded 4-angle columns (no temporary support)
- Replacing members or elements of riveted members

C10-8.3 Inspection of damage

C10-8.3.1 Initial inspection and action

Carry out an initial inspection to ensure safety to the user and to reduce further damage to the bridge. When damage is severe, an experienced structural engineer should make the initial inspection and determine whether to restrict traffic or close the bridge. Preliminary strengthening should be made immediately to prevent further damage. Preliminary strengthening may also be made to allow traffic on the bridge. These preliminary actions are normally based on judgment supplemented by brief calculations. If a severely damaged member is fracture critical, immediate steps should be taken to prevent bridge collapse. When a member is damaged beyond repair, the engineer may recommend at this time to partially or wholly replace the member. When safety of the user is in question, the bridge should be closed until it is conclusively determined that traffic can be safely restored.

C10-8.3.2 Inspection sequence and record

Commence inspection with the most critically damaged area first, followed by inspection of other damage in descending order of severity. Inspect the main supporting members first. Tension members should be inspected for indication of cracking. Compression members should be inspected for indications of buckling. When more than one member has been damaged a complete description of damage for each member should be given.

Painted surfaces should be visually inspected for cracks. Cracks in paint and rust staining are indications of cracking in the steel. Heavy coatings of ductile paint may bridge over cracks that are tight. When there is any doubt about ability to inspect for cracks, the paint should be removed. Damaged fracture critical members should be blast cleaned and magnetic particle inspected.

All areas inspected, including those areas inspected that did not suffer damage, should be recorded. This procedure aids the decision-making process of what, if anything, should be done to repair a member.

C10-8.3.3 Monitoring of repairs

Follow up inspection of repairs shall be made on a regular basis. Members that have complete restoration should be inspected with the same frequency as the complete bridge. Member repairs where there is some doubt regarding strength and durability should be inspected at more frequent intervals. Repairs to fracture critical members should receive close consideration with respect to inspection frequency.

C10-8.4 Assessment of damage

C10-8.4.1 General

Preliminary assessment of damage shall be made during inspection of damage as described under Section C10-8.3. Final assessment of damage shall involve at least one experienced engineer.

C10-8.4.2 Strength of damaged member

During assessment of damage, a complete evaluation of existing strength shall be made. This analysis should determine stress levels in the damaged member, and these stresses shall be compared to the design stresses. This analysis shall allow for all damage effects such as reduction of section, member distortion etc. Service load stress should always be computed. Overload, ultimate load, and fatigue stresses should be calculated as appropriate. Calculations should consider the effect of stress range and the fatigue category of the member. All preliminary calculations and decisions made during the inspection phase shall be reviewed.

C10-8.4.3 Fracture Critical Members

Fracture critical members shall receive a more rigorous assessment of damage than non-fracture critical members. Selection of repair procedures for fracture critical members shall be more conservative than selecting repair procedures for non-fracture critical members.

CAUTION

In general, crack repairs shall be made with bolted cover plates. If other methods are used, such as welding or flame straightening, elements shall be fully strengthened by adding new bolted cover plates. Enough new material shall be added so that the damaged material can be neglected in computing strength.

C10-8.4.4 Primary members

Primary members can be classified as compression or tension members. Tensile areas of members such as tensile portions of girders in bending are treated as tensile members. Many of the limiting restrictions in this manual apply only to tension members.

CAUTION

Primary members in tension shall be considered to be (classified as) Fracture Critical Members.

To qualify as a compression member, no combination of loading shall produce tension in the portion of the member being repaired. Compression members are not fatigue critical and, therefore, stress range limitations used for tensile members do not apply. Charpy impact toughness requirements apply to tension members only. Most crack repairs and partial replacements in compression areas may be satisfactorily done by welding.

C10-8.4.5 Secondary members

Secondary members are stressed because of deflection of primary members and/or are stressed because of secondary loads such as wind and earthquake. Secondary members that carry compression only shall be assessed and repaired in the same manner as primary compression members. Tension secondary members may be repaired by flame straightening and hot mechanical or cold mechanical straightening. Cracks can be repaired by straightening and welding provided the steel is weldable. No limitation on maximum strain shall be placed on secondary members, provided they can be straightened to allowable alignment.

C10-8.4.6 Straightening of FCMs

Any primary tension member may be straightened but all affected fatigue critical areas are to be plated except those areas where straightening has been achieved without mechanical assistance.

Edge strain (amount of yielding) can be estimated from Table 1:

Maximum versines (2% yielding on edge) (mm)			
Flange width (mm)	300mm chord	600mm chord	1200mm chord
100	4.5	18	72
120	3.8	15	60
140	3.2	13	51
160	2.8	11	45
180	2.5	10	40
200	2.3	9	36
250	1.8	7	29
300	1.5	6	24
350	1.3	5	21

400	1.1	5	18
450	1.0	4	16
500	0.9	4	14

Table 4 - Measurement of edge strain (versine)

Assuming the measured distortion is a circular curve, the edge strain (per cent) is given by

$$S = \frac{400wv}{C^2} \quad \text{where} \quad \begin{array}{l} w = \text{flange width (mm),} \\ v = \text{versine (mm)} \\ C = \text{chord (mm)} \end{array}$$

and the radius of curvature (mm) by

$$R = \frac{C^2}{8v} \quad \text{where} \quad \begin{array}{l} C = \text{chord length (mm)} \\ V = \text{versine (gap) (mm)} \end{array}$$

Sufficient area is to be added to compensate for the damaged section (in the unlikely event that the damaged member/component happens to fracture). As a minimum, 50% additional area is to be added. This minimum addition is based on the simple premise that if the member is initially designed for a working stress of about 0.5 Fy, the straightened member element could be neglected entirely and the maximum stress would not exceed Fy.

C10-8.4.7 Straightening of compression members

Compression members are not generally subject to fatigue failure but it is critical to ensure that buckling does not occur.

C10-8.4.8 Calculation of damage curvature

The assessment of damage to a member and selection of the repair method can best be accomplished from accurate inspection information. A sufficient number of measurements must be made to apply the proposed guidelines. The assessment process should provide information that can be used to select the appropriate repair procedure.

The best way to estimate curvature is by measuring versines of short chords. Straight edges (or spirit levels) 600mm or 1200mm long held against the inside of the curvature are more convenient than using string lines.

C10-8.4.9 Nicks and Gouges

Nicks and gouges shall be carefully described and photographed. Superficial nicks and gouges can be repaired by grinding smooth. More serious damage to weldable steel in compression members and secondary members can be repaired by welding. Other cases can usually be repaired by adding bolted cover plates. Requiring partial replacement due to nicks and gouges is rare.

The distinction between superficial and serious nicks and gouges shall be made by stress calculations. As a guide, superficial nicks or gouges can be taken as those resulting in less than 10% loss of section of the affected element.

C10-8.4.10 Cracks

Crack assessment must be preceded by a detailed inspection to locate the cracks and determine their length and width, including visual inspection supplemented with magnetic particle, or dye penetrant testing. Impact cracks are usually surface connected and ultrasonic testing is not generally necessary. The stress and shock of impact will sometimes cause cracking well away from the area of principal damage.

Look for spalling of paint or scale as an indication that some unusual strain has occurred at such locations and use as a guideline for areas of detailed inspection. Visual examination is not to be

limited to these areas, however, since a crack may occur in areas that were shock loaded but were not strained enough to spall the paint or scale. Visual inspection shall be supplemented with magnetic particle inspection in suspect areas.

Particular attention should be given to the examination of the toes of butt and fillet welds in areas subjected to damage as this is an area where cracks often occur.

Field inspection for cracks is done by magnetic particle, dye penetrant and occasionally ultrasonic inspection.

C10-8.5 Selection of repair methods

C10-8.5.1 General

Repair solutions can be selected from the following range. A combination of repair procedures may result in the best repair solution. Refer to Table 5.

Straightening procedures need to be done with care to prevent overstraightening (i.e. creating bending in opposite direction) and damage from straightening forces and devices. Also distortions due to yielding must not be confused with those due to restraints from other members.

Further information on the specific repair techniques relating to impact damage is contained in the National Co operative Highway Research Program Report 271.

C10-8.5.2 Flame straightening

This repair method does not significantly degrade steel properties, but is not generally effective where yielding has exceeded about 1%. It may be considered for the repair of all bent members with the following exceptions:

CAUTION

- Do not flame straighten fracture critical members unless the flame-straightened area is fully supplemented by bolted cover plates
- Do not attempt to flame straighten excessively wrinkled plates or plates with excessive kinks. It is nearly impossible to flame straighten this type of damage

C10-8.5.3 Hot mechanical straightening

This is a process where heat is applied to all sides of a bent member, and while the member is still hot it is straightened by applying force. Agencies that use this method restrict the maximum temperature to 640oC. The results of this type of straightening are highly dependent on operator skill. Lack of skill (or care) is frequently indicated by waviness of edges (especially the convex side of the damage) and local indentations due to local hot yielding under jacking loads.

It is believed that flame straightening is a superior method and should be used in lieu of hot mechanical straightening for all primary tension members, where practical. Hot mechanical straightening may be used on primary compression members or secondary members provided the operators have the skill to produce results that are free of wrinkles, cracks, bulges, and poor alignment.

C10-8.5.4 Cold mechanical straightening

Cold mechanical straightening is a process where an accidentally bent member is straightened by applying force. No heat is used. It is believed that a bridge member can be cold straightened once without causing significant degradation, provided the plastic strain is limited to 5% nominal strain.

Cold mechanical straightening shall not be applied to member areas that have cracks, nicks, or gouges, or to fracture critical members. Cold mechanical straightening should not be applied to members with low Charpy impact values. It is not recommended that twisted or rotated members be cold straightened.

C10-8.5.5 Welding

Welding may be used for several types of repair, including defect or crack repair, welding replacement segments into place, and adding straightening plates by welding. Poorly executed weld repairs in tensile areas can be very dangerous and in some instances may do more harm than good. Fracture critical members shall not be repaired by welding unless fully strengthened by additional bolted material.

The steels to be repair welded shall be weldable steels.

Do not weld members with low Charpy impact values unless plated in addition.

C10-8.5.6 Bolting

Bolting may be used as a repair method or as a supplement to other repair methods. Replacement of a damaged element with a new piece of steel fastened with fully tensioned high-strength bolts is regarded as the safest method of repair. Replacing damaged riveted elements with bolted material may not be excessively difficult and should be considered.

Fracture critical members shall be repaired by bolting or repaired by other methods and fully strengthened by adding new bolted material.

C10-8.5.7 Partial replacement

In some instances damage will be so serious that partial replacement is necessary. This damage includes excessively wrinkled plates, excessive deformation and bends, tears in member elements, and large cracks.

Partial replacement will normally consist of removing the damaged area and replacement with either a welded insert or a bolted splice insert.

Welded inserts are not recommended for fracture critical members. Partial replacement by bolting and welding is an acceptable method, provided the longitudinal web weld is located in a compression area.

Partial replacements can be used in conjunction with other repair methods, such as flame straightening. For example, a bent member with a crack could be flame straightened and the crack repaired by bolted cover plates.

C10-8.5.8 Complete replacement

Complete replacement of a member is normally the most expensive method of repair.

If a member is excessively damaged throughout its full length, replacement may be the only alternative. Other less difficult methods of repair should be carefully studied prior to selecting complete replacement.

Damage Assessment Factors	Repair Method to Consider								
	Flame Straightening	Hot Mechanical Straightening	Cold Mechanical Straightening	Welding	Flame Straightening	Field Welding supplemented	Bolting	Partial Replacement	Full Replacement
Weldable Steel	✓	✓	✓	✓	✓	✓	✓	✓	✓
Non-Weldable Steel	✓	✓	✓	✗	✓	✗	✓	✓	✓
Low Charpy Impact Values	✓	✗	✗	✗	✓	✗	✓	✓	✓
Adequate Charpy Impact Values	✓	✓	✓	✓	✓	✓	✓	✓	✓
Fracture Critical Member	✗	✗	✗	✗	✓	✓	✓	✓	✓
Primary Tension Member	✓	✓*	✓*	✓	✓	✓	✓	✓	✓
Secondary Members	✓	✓	✓	✓	✓	✓	✓	✓	✓

All Compression Members	✓	✓	✓	✓	✓	✓	✓	✓	✓
Tearing and Excessive Wrinkles	✗	✗	✗	✗	✗	✗	✗	✓	✓
Primary Tension Member Curvature Strain Meets Guidelines	✓	✓*	✓*	✓	✓	✓	✓	✓	✓
Primary Tension Member Curvature Strain Does Not Meet Guidelines	✗	✗	✗	✗	✓	✓	✗	✓	✓
Member Curvature Radius More than	Member will return to correct position when adjacent members or joints are straightened								
Cracks – Weldable Steel	✓	✗	✗	✓	✓	✓	✓	✓	✓
Non Weldable Steel	✗	✗	✗	✗	✓	✓	✓	✓	✓
Superficial Nicks and Gouges	Grind Defect Smooth								
Nicks and Gouges Weldable Steel	✗	✗	✗	✓	✗	✗	✓	✓	✓

Table 5 - Selection of repair method for impact damage

* Flame straightening is recommended

C10-8.5.9 Strength of repair method

Fracture critical members should be repaired by methods that unquestionably restore design strength. These methods may include bolted splices, partial replacement by bolting, and full replacement. All loading capacities, including service load, overload, and ultimate load, should be fully restored, and the service life should be fully regained.

Non-fracture critical members may be repaired by the same methods used for fracture critical members. However, other less costly methods should also be considered and used as appropriate.

C10-8.5.10 Durability of repair

Durability of repair must be given a high priority. All methods of repair should have durability equal to or better than the original member (commensurate with remaining life of the structure as a cost-effective solution). The accessibility of all parts of a repaired structure for inspection, cleaning, and painting shall be accomplished by the proper proportioning of repairs and the design of their details. Closed sections, and pockets or depressions that will retain water, shall be avoided. Pockets shall be provided with effective drain holes or filled with waterproofing material.

C10-9 Management of lead paint on steel structures

C10-9.1 Procedure

Much of the structural steelwork within the NSW rail network is primed with red lead primer. It was standard practice to use red lead primer on new construction until 1968 when it was replaced by inorganic zinc silicate primer and on old construction until 1983 when it was replaced by zinc rich epoxy primer. While red lead primer based systems remain intact, they present no significant health or environmental pollution hazard. However, in most instances these systems require repair or replacement during the design life of the structure, and we are confronted with potential health and environmental pollution hazards associated with lead based paint removal during the course of surface preparation for repainting.

Work involving surface preparation on steel structures with lead based paint shall be carried out in accordance with the requirements of UGLRL CRN's Safety Management System.

The successful management of an industrial structure coated with lead-containing paint requires consideration of the function, design life and service environment of the structure, condition of the existing coating system, coating systems that will be effective in the service environment, hazards associated with lead paint removal to workers, the public and the environment and program cost. These factors and their many intertwined elements need to be judiciously dealt with in a logical and systematic manner for satisfactory painting program selection and implementation to take place. Thus, these guidelines are presented in the form of a step by step procedure to facilitate these considerations when designing a lead-containing paint management project.

The procedure comprises the following steps:

- Determination of the presence of lead paint
- Selection of painting strategy
- Selection of paint removal method and containment system

These steps as described below may require modification to accommodate particular structures, locations or regulations. Further, there may be circumstances where small, isolated patch repairs or repairs to small structures far from the public or a waterway are to be carried out, and it may be unnecessary to follow the procedure except for regulatory requirements concerning worker protection and waste handling.

C10-9.2 Determination of the presence of lead paint

The presence of lead paint may be established by reviewing the historical painting records of the structure if these records are complete and include its entire painting history from original painting through last maintenance painting. If historical records are incomplete, the presence of lead paint may be determined by chemical field testing or laboratory tests of representative paint samples in accordance with AS 1580.501.1 (withdrawn) or atomic absorption spectroscopy.

A paint film is considered to be lead-containing if it has 1.0% or more lead or lead compounds by weight in the dry film. Structures showing paint work with this level of lead should be dealt with in accordance with these guidelines.

C10-9.3 Selection of painting strategy

The following considerations should be taken into account when selecting a cost-effective painting strategy:

- Coating condition
- Service life of the structure
- Service environment of the structure
- Surface coating systems that will be effective in the service environment
- Method of surface preparation required for the coating system to perform effectively and its emission potential.
- The need to keep the structure or facility operational
- Proximity of the work to other facility workers or the public
- Proximity of the work to environmentally sensitive areas

The major consideration in selecting a painting strategy is existing coating system condition. Accurate assessment of this condition is critical to selection of a cost effective painting strategy and should only be carried out by staff with expertise in this field.

The following maintenance painting strategies are available:

C10-9.3.1 No painting

When a lead-containing paint is in good condition, tightly adhering to the substrate, there is no hazard; thus, the option of leaving the coating alone may be appropriate.

If the coating has poor film integrity or shows poor adhesion to the substrate, consideration should be given to potential structural problems due to further corrosion and the possibility of the surrounding area being contaminated by flaking and peeling paint. Again, the option of leaving the coating alone subject to carrying out periodic inspections to monitor coating deterioration may be appropriate.

C10-9.3.2 Overcoating

When a lead-containing paint or system is adhering tightly to the substrate and showing satisfactory film integrity, it may be contained on the structure for an extended period of time by overcoating it with another topcoat or paint system if its film thickness is not so great that the additional weight of the overcoat will weaken or break the bond between the existing system and the substrate.

Prior to overcoating, the existing paint system should be cleaned to remove dust, dirt, grease, oil, loose paint and other contaminants to maximize adhesion. The overcoating material isolates the existing system from the environment thereby eliminating any hazard.

A number of systems are recommended for this application by the Paint Industry, and a few are detailed below:

- Two-pack epoxy sealer with various topcoats
- Two-pack, high build epoxy mastic
- Single pack, moisture cured, polyurethane topcoat pigmented with zinc or aluminium
- Water borne acrylic primer with acrylic topcoat
- Oil modified alkyd topcoat with corrosion inhibiting pigment

When considering this option, one should be particularly cautious where the existing system's film integrity or adhesion to the substrate is suspect and guided by the recommendation of the manufacturer of the overpainting system.

C10-9.3.3 Spot or localised repair

Where a structure is exhibiting localized coating breakdown and steel corrosion products, surface preparation of these areas, priming and application of a finish coat to the localized areas or to the whole of the structure may be cost effective options depending upon the extent of the surface area to be repaired. In this instance, a major risk with overcoating is inaccurate assessment of the condition and adhesion characteristic of the existing lead based paint system, i.e. if either is poor, very early coating failure will occur. When spot (patch) painting, care should be taken to ensure the repair system paint products are compatible with the existing system and provide long term performance in the service environment.

C10-9.3.4 Total coating removal and replacement

Surface preparation of the entire structure, to the extent that all existing coatings are removed and a new protective system is applied, may be a cost effective option depending upon the design life of the structure, despite the high short term costs associated with coating removal, containment and disposal.

C10-9.3.5 Demolition and replacement of the structure

It may be cost-effective to replace the structure rather than removing and replacing the coating. Costs associated with the construction of field containment, the controls over emissions, and worker and environmental protection are applied to a new structure. The risk of environmental, worker, or public contamination is very low, the service life of the structure is optimized and the lead paint coated steel can be smelted and recycled. A disadvantage of this option is the potential disruption while the structure is demolished and replaced.

C10-9.4 Selection of paint removal method

The following factors should be considered when selecting the method of paint removal:

- Painting strategy selected
- Degree of surface preparation required by the maintenance system
- Amount of work to be performed and productivity requirements

- Size, configuration and accessibility of the structure
- Existing service disruption, if any
- Cost

Paint removal methods are categorized into four groups based upon the level and type of paint emissions generated. These categories are:

C10-9.4.1 *Dry abrasive blast cleaning*

Dry abrasive blast cleaning with expendable or recyclable abrasives is in this category as it produces more dust than any other method of paint removal. The use of recyclable abrasives significantly reduces the volume of blasting debris that in most instances must be treated as a hazardous waste. Dry abrasive blast cleaning is the most effective method of surface preparation for paint durability, and it has a high production rate.

C10-9.4.2 *Wet abrasive blast cleaning*

Wet abrasive blast cleaning and water blasting are in this category. It is a dust free method of surface preparation; however, the wet blasting debris is more difficult to handle and transport than dry debris. It requires the use of inhibitors to avoid flash rusting, and it is significantly slower than dry abrasive blast cleaning.

C10-9.4.3 *Power tool cleaning*

Power Tool Cleaning without vacuum attachments and Chemical Removal are in this category. Power Tool Cleaning produces significantly less dust and debris than dry abrasive blast cleaning. It is significantly less effective for paint durability than abrasive blast cleaning, and it has a medium production rate. Power tool cleaning is a dust free operation, and the volume of debris is significantly less than abrasive blast cleaning. It is not a recognized method of surface preparation and pre-treatment of metal surfaces prior to protective coating; thus, further surface preparation is required before painting.

C10-9.4.4 *Hand tool cleaning*

Hand Tool Cleaning and Power Tool Cleaning with vacuum attachments are in this category as dust generation is minimal, and the volume of debris is significantly less than abrasive blast cleaning. Hand tool cleaning is the least effective method of surface preparation for paint durability, and it has a low production rate.

The selected paint removal method has an influence on selection of containment system.

At the completion of this step, if the method of removal selected for the project does not appear to be feasible, the painting strategy chosen in C9-9.3 should be reconsidered.

C10-10 *Welding old steels*

C10-10.1 *Introduction*

The materials used in railway construction over the past 100 years or so have evolved more or less in step with the changes in the metal fabricating industry and it is important to keep in mind that for the first two thirds of this period welding was not widely used and riveting was the usual method of joining. As a consequence of this the weldability of the materials involved was not a prime consideration and the ease, and in some cases the practicability, of welding can vary considerably. This, of course, makes maintenance very much more complex as riveting is no longer commonly used and where welding is not practicable bolting may be the best solution.

In an actual repair situation it is clear that identification of the material involved is the first and most important step.

C10-10.2 Wrought and cast iron

It is appropriate to describe in a short paragraph the differences between wrought and cast iron since, unfortunately, the terminology in common use in Australia does not correspond to the engineering definitions of wrought and cast iron.

Cast iron is a product of a blast furnace with a high carbon content (see Table 6) useful only in compression. Since it has almost no ductility in tension, its use is confined to compression situations, i.e. bridge columns, compression side of beams, and tunnel liners. The cast iron discussed in this note should not be confused with the modern spheroidal graphite iron that has good tensile properties but poor weldability.

The cast iron covered here is the type often described as "Grey Cast Iron".

Wrought iron, as we shall consider it, was produced by "puddling" pig iron on contact with mill scale (iron oxide) and is similar to a low carbon low strength mild steel containing slag inclusions that reduce its properties in the through thickness or "Z" direction. Beams of riveted construction often had cast iron in the compression flange and wrought iron elsewhere. Beams of this type have been seen in the central city tunnels.

Wrought iron is quite easily welded if the provisions described below are observed, but grey cast iron is quite a different proposition and may be either very difficult or impossible to weld.

It is therefore very important to identify the materials involved before any repair work is undertaken.

	Cast iron	Wrought iron	Mild steel	Structural Steel
Carbon	3.5%	0.1%	0.1 to 0.3%	0.18%
Silicon	1.9%	0.2%	0.3%	0.2%
Sulphur	0.1%	0.1%	0.06%	0.02%
Phosphorous	1.0%	0.1%	0.06%	0.2%
Manganese	0.7%	0.4%	0.4%	1.0%
Slag	0	1%	0	0
Yield Stress	-	215 MPa	250 MPa	250 MPa (Specified Min)
Tensile Stress	120 MPa	354 MPa	460 MPa	410 MPa (Specified Min)
Elongation	-	30%	20%	22% (Specified Min)

Table 6 - Iron based structural materials - Typical properties

C10-10.3 Welding wrought iron

Since the carbon and sulphur contents of wrought iron are low, it presents no weldability problems in terms of hot or cold cracking, but the presence of slag stringers aligned with the forging direction presents problems. Firstly, whilst butt welds in the X and Y directions are quite feasible, welds on the surface Z direction are not recommended if significant load is to be transmitted as a lamellar tearing type of fracture is likely to occur.

Welding by the Manual Metal Arc Welding (MMAW) method is usually recommended using Basic Low Hydrogen electrodes. Not that hydrogen is a concern, but the basic flux is better able to accommodate the slag absorbed into the weld pool. Neither preheat nor post heat is required.

Welding procedures and preparations are similar to those used for structural steel but it is recommended that welding speeds be reduced to 50-66% of those used for steel to allow time for slag removal from the weld pool.

Welding procedures and operators should be qualified where possible along the lines of qualification to AS 1554.1. If wrought iron of suitable section is not available, qualification may be done using Grade 250 structural plate.

Where distortion due to welding is undesirable, then the welding procedure may be combined with a peening procedure to eliminate distortion. See Welding Technology Institute of Australia (WTIA) Technical Note 11 "Commentary on the structural steel welding Standard AS 1554" for details of this procedure.

In cases where it is necessary to make a connection to the surface of a wrought iron member then consideration should be given to replacing a length of the wrought iron by structural steel attached by butt welds to wrought iron or by using a bolting technique.

C10-10.4 Mild steel

Mild steel was first produced in the late 1800's as a cheaper substitute for wrought iron that was rather expensive and variable in properties. The differences between wrought iron, mild steel and the current structural steels can be seen in Table 6. The important things to note are the higher carbon content, relatively high sulphur and phosphorous contents and the low manganese content.

Wrought iron is readily forge or blacksmith welded, but mild steel does not lend itself to these welding processes and the introduction of mild steel led to a great increase in riveting and the extinction of forge welding.

Fusion welding of mild steel faces two problems. Firstly, the higher carbon content, which increases the hardenability of the material and requires higher levels of preheat and arc energy to prevent excessive hardening in the HAZ which can be dealt with; but more importantly the tendency towards hot cracking in the HAZ arising from the low manganese to sulphur ratio. (Compare the values in Table 6 for Mild Steel and Structural Steel). Whilst some compensation for this problem is possible by adjusting the welding procedure, the fusion welding of some mild steels is impracticable. An additional factor is that mild steels are frequently segregated, that is, contain bands of high carbon/sulphur content and these areas are likely to cause welding problems.

Whether a particular piece of steel is likely to be difficult to weld depends very much upon the raw materials used in steelmaking and is therefore at the current time very much an unknown. The safest course is to either avoid welding or to extract a small sample of material for chemical analysis and metallographic examination as will be described later.

As with wrought iron and structural steel, both the welder and welding procedure should be qualified prior to commencement of work. Again the qualification procedure described in AS 1554.1 should be followed. Since the mild steels are the most difficult of the three materials being discussed, welder qualification on mild steel should be accepted as qualification for procedures within the limits allowed by AS 1554.1 for wrought iron and structural steel.

C10-10.5 Welding procedures for mild steel

Irrespective of the results of chemical analysis, mild steels should be regarded as being at least Weldability Group Number 5 since the sample removed for analysis and the modern analytical methods test only a small area and may not detect segregated areas.

To minimise the risk of HAZ hot cracking, weld beads should be kept small and if necessary buttering techniques should be used to reduce the residual stress from welding and the chance of cracking. Low hydrogen basic consumables must be used to counteract the deleterious effects of sulphur and phosphorous into the weld metal.

For those reasons, MMAW basic low hydrogen is the only welding process really suitable for welding these materials. Other welding methods have either too large a HAZ and hot cracking risk, or low tolerance to sulphur and phosphorous absorption to yield satisfactory results.

C10-10.6 Structural steel

Structural steel was developed to overcome the shortcomings of mild steel, i.e. cold cracking, hot cracking, brittle fracture and lamellar tearing. Structural steel is intended for welding.

The procedures for welding such steels are fully described in AS 1554.1 and the welder and procedure requirements of this standard should be observed fully. Welding consumables should as far as possible conform to the prequalified consumables listed in Table 4.6.1(A) of AS 1554.1 and in repair situations it will generally be found that adherence to the prequalified preparations of Table E1 will provide the optimum solution.

C10-10.7 Material identification

As discussed above, it is essential before embarking upon any form of welding repair to determine the nature of the materials involved. This can be done from existing records of construction, although care is necessary if previous repairs have been carried out on the bridge; from material identification marks on rolled sections that often give the name of the steelmaker and material standard; and by metallurgical examination of the steels involved.

Removal of a sample for examination should be done for each piece of plate or section involved in the welded repair, preferably in a location where re-welding is not required, and simply grinding smooth the area of material removed is acceptable. Typically notching a V-shaped section from the edge of a plate or section approximately 15 mm wide, half-plate thickness and 20mm long will suffice. Removal is best done by hand using a hacksaw as power grinders often overheat a small sample. An alternative method that is quicker and better is to take a core sample using a trepanning cutter. The sample should be min 15mm diameter. Samples should be sealed in an envelope clearly marked with bridge identification and sample location and sent to a laboratory for metallographic examination and chemical analysis for carbon, manganese, silicon, aluminium, sulphur and phosphorus contents.

Occasionally it may be impossible to remove a sample for the above testing by a simple sawing method. In such cases the use of a "Boat Sample" removal apparatus can be considered, or the use of in situ metallography. In the latter case the metal surface is prepared without metal removal to permit microscopic examination on site or after the removal of a surface replica.

Unfortunately, the current methods of in situ chemical analysis, spectrographic and X-ray diffraction, are not suitable for some of the elements of interest and it may be necessary to remove drillings for chemical analysis. In this case care must, of course, be taken to ensure that the drillings removed are not contaminated with paint, oil or other drilling lubricant. Sample drillings should be sealed in an envelope marked with bridge identification and sample location.

Usually about 30gm of drillings will be required for chemical analysis and this represents a 8 mm dia hole of about 8-10 mm in depth that will provide the necessary sample and will not require welding repair.

C10-10.8 Identification of wrought iron

Identification of wrought iron bridges is very important, so that over estimation of rated capacity and remaining fatigue life does not occur, because they are assumed to be steel. This has been a common mistake. In one case, the same underbridge was assessed on 4 different occasions by 3 different groups and all assumed the underbridge to be steel, when it was actually wrought iron.

Steel structures which have some wrought iron components are particularly difficult to identify. The original drawings cannot be relied upon to indicate those components that are steel and those that are wrought iron.

C10-10.8.1 Office identification of wrought iron

Historical records

Office identification of wrought iron should be based on historical documents recording the construction date of the bridge. Advice may be sought from field personnel, particularly if a dated plaque is on the bridge.

Note: Do not rely on the date on drawings unless you have additional confirmation.

Design details

Design details are another indicator that a bridge may be from the wrought iron era. It must be emphasised that these details can exist in either steel or wrought iron bridges, but they at least indicate further assessment needs to be made. The design details are as follows:

- Flat bar tension members, particularly if they are diagonal tension members.
- Tie rods with turnbuckles for tensioning and pin connections used for wind bracing and/or diagonal tension members.
- Wind bracing or sway bracing which includes:
 - Flat bar members
 - Connections to girders made by forge bending and/or forge welding of bracing members instead of using gusset plates
 - Connection of wind bracing to the centre of web (i.e. mid-depth) and/or away from stiffeners
 - Angles which have chamfered edges on both sides of the toes (steel angles are rounded on the inside face and square on the outside)
 - Expansion bearing of simple plate on plate design instead roller bearings for spans above 20 metres long
 - Rivets with flattened or cone shaped heads instead of round heads

C10-10.8.2 Field identification of wrought iron

Test samples

The most positive method of field identification is to obtain core or other samples for metallurgical assessment by preparing macros or micrographs as detailed above, to determine if laminations exist. For an as-constructed underbridge one sample may be sufficient. For most underbridges over 100 years old there is usually strengthening and/or repairs which have added steel plates and sections. In this case, a number of samples will be required.

Magnetic particle testing of edges

Taking and testing samples is a costly and time consuming process. A simpler method has been field trialled on a number of underbridges of both wrought iron and steel. This method involves filing or grinding an edge, then magnetic particle testing the edge to see if laminations can be seen which indicate wrought iron. Care has to be taken as early steel may have stringer inclusions that look like laminations. Some problems were found with early steels during the field trials, but removing more metal by filing usually gave a more definite result (Figure 14 to Figure 18).



Figure 14 - Poor quality wrought iron can often be identified by defects on the edges of the rolled section. Edges often show the straight outline of laminations and delaminations



Figure 15 - Web edge filed and mpi, showing laminations clearly, positively identifying wrought iron



Figure 16 Edge filed prior to mpi. Laminations just visible



Figure 17 - After application of mpi laminations show clearly, positively identifying wrought iron



Figure 18 - Deck girder, edge filed at top flange. Laminations confirm drawing notation as wrought iron

C10-11 Concrete repair materials

C10-11.1 Introduction

No matter how carefully a repair procedure is carried out, using the wrong material will most likely lead to early repair failure. Therefore, selection of appropriate materials is an absolute requirement for obtaining durable repairs.

The basic criterion for selecting a repair material is that its material properties match the properties of the base concrete and a good bond is achieved and maintained at the repair interface.

C10-11.2 Material properties

The material properties that affect the quality of a repair are:

C10-11.2.1 Dimensional stability

Bond failure between new and old concrete is usually caused by relatively large shrinkage of the new concrete (or mortar) while the old concrete does not shrink further. Therefore, the repair material must be either shrinkage free or else be able to shrink without losing bond.

C10-11.2.2 Coefficient of thermal expansion

When a composite of two materials of widely varying thermal coefficients undergoes a significant temperature change, differences in volume changes can cause failure either at the bond line or within the section of the lower strength. Therefore, when making large or thick patches or when placing an overlay, it is important to use a material with a coefficient of thermal expansion similar to that of the concrete being repaired.

C10-11.2.3 Modulus of elasticity

The modulus of elasticity of a material is a measure of its stiffness. High modulus materials do not deform under load as much as the low modulus materials. Consequently, when materials with widely differing moduli are in contact with each other and subjected to a common load, the lower modulus material would tend to yield or bulge transferring the load to the stronger material that if overloaded may then fracture. For this reason, a wall or section made of relatively flexible (low modulus) material should not be patched with a stiff (high modulus) material.

C10-11.2.4 Permeability

Permeability refers to the capability of a material to transmit liquids or vapours. Good quality concrete is relatively impermeable to liquids but freely transmits vapours. If impermeable materials (such as epoxies) are used for large patches, overlays or coatings, moisture vapour that passes up through the base concrete can be entrapped between the concrete and the topping. Entrapped moisture can cause a failure either at the bond surface or within the weaker of the two sections.

Impermeable materials should also generally be avoided in patching concrete that has been damaged due to corrosion of reinforcing bars as it may accelerate the rate of corrosion.

For the above reasons, whenever possible cementitious materials should be used for repairing concrete structures due to their compatible physical properties with the parent concrete. Cementitious mortars and concretes may be modified by the addition of polymers but are always preferable to resin mortars if circumstances make it practical to use them.

C10-11.3 Types of materials for concrete repairs

C10-11.3.1 General

A large variety of materials are used in the repair of concrete structures. These materials may be used singly or in combination to achieve the best results according to the circumstances of a particular repair job. It is therefore essential to know the characteristics and application requirements of different materials so that an appropriate selection can be made for the repair in hand.

The materials commonly used for concrete repairs are:

- Polymers (synthetic latexes) and polymer modified cement mortars and concretes
- Synthetic resins and resin based material
- Unmodified cement based mortars and concretes
- Steel reinforcement coatings
- Substrate bonding coats
- Acrylic concretes
- Non-shrink hydraulic cement mortars
- Sprayed concrete
- Protective coatings
- Flexible joint sealants

C10-11.3.2 Polymers (synthetic latexes) and polymer modified cement mortars and concretes

Synthetic latexes are made by dispersing polymer particles in water to form a polymer emulsion. When these emulsions are added to Portland cement concrete/mortar, the spheres of polymer coalesce to form a film that coats the aggregate particles and hydrating cement grains and seals off the voids.

The polymer modified cementitious mortars and concretes are high performance repair materials that work monolithically with parent concrete due to their similar physical properties such as modulus of elasticity and coefficient of thermal expansion. Their other good attributes are:

- Increased workability (therefore, lower water-cement ratio),
- Excellent bond to existing concrete and steel in dry, damp or wet conditions,
- Low shrinkage,
- High impermeability to water,

- Higher resistance to chloride ion and carbon dioxide penetration,
- Inherent alkalinity to passivate the steel reinforcement,
- Higher strength
- Increased resistance to freeze-thaw damage and chemical attack
- Higher abrasion resistance

The three basic polymers used as latex modifiers for concrete are:

- Polyvinyl acetates (PVA)
- Acrylics
- Styrene-butadiene rubber (SBR)

PVAs are not recommended for use in wet environments because some types may hydrolyse and break down.

SBR latexes develop a brownish coat after being exposed to sunlight and this may make them unsuitable for patching applications where colour matching is important.

Limitations

The working life of polymer modified mixes is relatively short. Therefore, the quantity of mix for a particular job should be limited according to the placing and finishing time - about 20 minutes. If the mortar or concrete is manipulated after the latex has coalesced, cracking may occur on drying.

Application of polymers is also sensitive to temperature. At low temperatures the polymer spheres will not coalesce to form a durable film around cement and aggregate particles. At high temperatures their working time is too short allowing little time to finish the repair. Manufacturers' instructions in this regard should be carefully followed.

Bonding coat

To obtain a high bond between the latex concrete overlay or mortar patch and the base concrete, a bond coat is brushed or broomed onto the prepared concrete surface. This bond coat can be the mixture used for the overlay or patch, or made by mixing undiluted latex with Portland cement. The surface is first thoroughly wetted with clean water for not less than one hour prior to placement. After removing all free water but with the surface still damp, sufficient mixed material to coat all bonding surfaces is then placed and vigorously broomed to assure maximum contact with the old concrete. The rate of application of bonding material should be limited so that the bond coat does not dry before being covered with repair mortar or concrete.

Curing

The curing procedure of polymer modified concrete is different from normal concrete. Wax or resin found in most curing compounds are incompatible with latex and should not be used without prior evaluation.

The polymer film formed in polymer modified concrete helps to maintain high levels of internal moisture in the concrete. Because of this, prolonged curing is neither necessary nor recommended. To prevent shrinkage cracking before the film has formed, however, all finishing operations must be completed and the surface covered with a single layer of wet burlap as soon as the surface will support it. The curing cover is completed by placing a layer of polyethylene film over the wet burlap. This is left in place for 24 hours after that the burlap and polyethylene are removed and the surface is permitted to dry for 3 to 5 days.

C10-11.3.3 Synthetic resins and resin based materials

Epoxies are frequently used as repair materials because they bond well to almost all materials, cure rapidly, are high in both tensile and compressive strengths, exhibit good chemical resistance,

and they shrink very little during curing. Applications include use in bonding concrete (hardened to hardened, and hardened to fresh) and in patches, overlays and protective coatings.

However, there are significant differences between the physical properties of resins and concrete. In particular, resins have an elastic modulus that is about one-tenth that of the concrete and coefficient of thermal expansion 5 to 8 times higher than that of concrete. The strength of resin based materials in compression is usually higher than the strength of concrete, and in tension it is much higher. These differences result in excessive stresses at the bond interface so that delamination of the epoxy repair is likely to occur either at the interface or just within the concrete substrate. As a result, epoxy mortar repairs are more suitable for thin and small volume repairs.

Further, in contrast to the cementitious materials that re-passivate the steel reinforcing bar, epoxy resin materials do not passivate the bar but rather arrest corrosion only by excluding the oxygen due to their low permeability to moisture and gases. In fact, in marine environments epoxy repair materials are likely to trap chloride spray against reinforcement and introduce in-built potential chloride attack on the steel.

Epoxy resins offer excellent repairs in the following situations:

- Repairs of cracks up to 6 mm width by injection with low viscosity unfilled epoxy resin that has the potential to penetrate and seal cracks down to 0.02 mm width. Crack injection is normally associated with dead cracks that are basically inactive and do not move
(If the cracks are live and continue to move with changing loads or temperatures, they cannot be repaired with resins. In such cases, cracks are treated as expansion joints and sealed with flexible materials)
- Bonding of new concrete to old concrete. Epoxy resin formulations provide excellent adhesives and can give long drying times prior to placing of repair materials. This is particularly useful where complicated formwork has to be assembled
- Bonding of steel, brick, concrete blocks and other materials to existing concrete (i.e., use as adhesive)
- Epoxy resin grouts, mortar and concretes for reinstating deteriorated concrete or patching in thin layer applications without problems of drying shrinkage associated with cementitious repairs
- Resin mixtures can be made fluid enough to flow into places by gravity so that inaccessible places can be filled with them and compaction is unnecessary. This is valuable for packing bearings, machine bases and so on

Epoxy compounds consist of a resin, a curing agent (hardener) and modifiers that make them suitable for specific end uses. Modifiers include accelerators that make the rate of cure depend less on temperature, dilatants that reduce viscosity and improve workability, and fillers such as sand and aggregate that lower cost, decrease shrinkage and reduce the volume change due to thermal expansion.

The resin generally consists of two components that are batched by volume and thoroughly mixed before the incorporation of aggregate. Chemical reactions start as soon as the resin components are combined and the working time will depend on the system, the temperature and the handling procedure.

Accurate batching and proper mixing of the components is crucial for attaining maximum strength and other properties of the epoxy materials. For this reason they should be mixed in whole batches that are obtained pre-proportioned from the supplier.

If formwork is used with epoxy materials or epoxy modified concrete, the form surfaces should be coated with a release agent compatible with the epoxy.

Surfaces of base concrete and steel should be primed with neat resin. Placement and consolidation should be done in layers of limited thickness as recommended by the epoxy formulator.

Considerable skill and experience are needed for the successful application of epoxy resin materials. They have to be applied within a very limited time before they harden and have to be handled cleanly to avoid contamination of both the resin mixture and the people working with them. Therefore, it is advisable to employ specialists to supply them as well as apply them.

C10-11.3.4 *Unmodified cement based mortars and concretes*

To match properties of the base concrete as closely as possible, Portland cement mortar and concrete are frequently the best choices for the repair material. However, if there is a difference in aggregate source, maximum aggregate size or water content, properties will differ. Also, the in-place concrete probably will have undergone considerable drying shrinkage so that differential volume changes between a repair and the in-place concrete will almost certainly occur. The effects of differential volume change can generally be minimised by maximising aggregate size, minimising the water content and by following good curing procedures.

The concrete mix used for repairs must be capable of producing highly impermeable concrete. Additives such as ground granulated blast furnace slag, pulverised fuel ash or microsilica can be used in repair mixes to increase impermeability in the same way as in new concrete. The use of accelerating admixtures may be advantageous but the admixture itself should not contain more than 1 per cent chloride ion by weight and the resulting total concrete should not contain more than 0.1 per cent chloride ion by weight of cement.

The nominal maximum size of the coarse aggregate should be less than of the patch or overlay depth, and not more than 10 mm, bearing in mind that the concrete may have to get into fairly tight locations.

The water-cement ratio should not exceed 0.4 by weight, lower ratios being preferred. The slump of mixes for shallow patches and overlays should not exceed 25 mm. The slump of mixes that are to be consolidated around reinforcing steel by internal vibration should not exceed 75 mm.

Accurately batched and properly mixed concrete is essential to the success of repairs. To avoid variability in site mixed concrete due to the difficulty of accurate proportioning, pre-packaging at the maintenance shop or other suitable location should be considered. If the aggregate cannot be completely dried it must be packaged separately from cement.

A number of proprietary pre-packaged cement based mortars and grouts incorporating special cements, chemical additives and admixtures, specially formulated to exhibit high bond, high strength and non-shrink properties are also available in the market. Prior to using any proprietary material its suitability for a particular job must be verified from the manufacturer's printed literature.

Cement based unmodified materials do not always adhere successfully to old concrete and it is important that the old concrete be kept wet for a period of 12 to 24 hours prior to repair to ensure that the old concrete does not suck away water from the new concrete thus preventing full hydration of the cement at the critical interface. However, prior to placing the new concrete the surface of the parent concrete must be dry and without free water so that the water-cement ratio at the interface is not increased.

The other alternative is to prime the repair areas with Portland cement or latex modified Portland cement grout, or an epoxy system. A bonding agent must be applied for low slump mixes.

The Portland cement grout should consist of a mixture of 1 part cement to $\frac{3}{4}$ or 1 part fine aggregate and sufficient water to make a heavy cream consistency.

Polymer-modified bonding grouts have a short drying time (normally less than 30 minutes) and cannot be used if there is much form fixing to be done before the concrete can be cast.

Epoxy bonding coats have two special advantages. Firstly, they can be formulated to have long open times that makes them suitable for use in hot climates, or when formwork has to be fixed

after the bonding coat has been applied. Secondly, they may provide a more effective barrier than cement grouts against the migration of chlorides.

Note: Although they are water compatible, epoxy bonding coats are applied to a dry concrete surface. However, specially formulated resins are available for application to damp surfaces also.

Curing unmodified cement based mortars and concretes

With low water-cement ratio repair concrete, a continuous water cure is the preferred method for strength development.

Curing compounds may be used, however, they neither furnish desirable external water to low water-cement ratio mixes nor do they provide any cooling effect. They should not be used if additional material is to be later bonded to the surface being cured. Curing compounds should be applied at twice their usual rate to shotcrete and to other rough textured surfaces. It is essential that freshly placed surfaces be kept moist until curing is initiated. A fog spray or a film should be used if there is to be any delay in the application of curing compounds.

C10-11.3.5 Steel reinforcement coatings

The purpose of using reinforcement primers and coatings is to ensure:

- Adequate bond between steel and repair material
- Protection of steel in the repair zone against further corrosion
- Prevention of corrosion progressing under the primer

The following types of coatings can be applied to steel reinforcement:

- Cement slurry.
- Cement slurry modified with polymer or latex emulsion
- Epoxy resin (with or without alkaline admixture)
- Inhibitive primers (such as zinc chromate primer)
- Zinc rich epoxy primers

Alternatively, the cement paste from a well-designed cementitious repair mix may protect the reinforcement better than any separately applied coating.

CAUTION

A coating is not an alternative to removing chloride contamination from the reinforcement nor will it prevent corrosion from being caused by chlorides that are already present on the reinforcement. It is therefore essential to remove the rust and chlorides from corroded steel before applying any primer

Where the repair is cement based, a coating of cement slurry (or the cement paste in the repair mortar or concrete itself) would create a lasting alkaline environment on the surface of the steel and offer a high degree of impermeability to water, carbon dioxide and chloride ions.

Polymer modified cement slurries may dry too quickly to be effective in repairs where forms have to be fixed after the coating is done, but they are suitable where the delay in placing the repair material is short.

Use of epoxy resins, inhibitive primers and zinc rich primers should only be made after consultation with concrete repair experts, as in certain circumstances these coatings may do more harm than good.

Epoxy resin coatings act as an impermeable barrier against external moisture and gases, but if any chlorides are trapped under the coating the corrosion of steel could still continue.

Where corrosion is due to a local flaw in generally sound concrete, the grit blasted bar may be coated with zinc rich paint. Where there is generally poor quality concrete, but the environmental

attack is such that corrosion has occurred in a limited area (i.e. there may be a large number of cathodes compared with the anodic corrosion sites), the anodic zinc coating may be rapidly used up and steel re-attacked. In this case zinc rich paint should not be used. Where there is generally poor concrete and corrosion is very widespread (that is, there are many anodic as well as cathodic sites) zinc rich paint may be used.

C10-11.3.6 Substrate bonding coats

Provided that the surface of the parent concrete has been properly roughened and all loose material removed, it is not essential to have a bonding layer. There is a widespread opinion that it is helpful, and whether one is used depends primarily on the practicality of applying it.

Independent advice should be sought as to the product to use, remembering that some bonding agents intended for use inside buildings actually break down eventually in the presence of moisture. The practical difficulty lies in the requirement for the agent to be applied so that it is tacky at the time of placement of the repair material. The bonding coat will prevent bonding if it is allowed to become dry.

For epoxy materials the bonding coat consists of the same resin as used for the repair material.

For cementitious mortar and concrete, it is possible to use Portland cement grout, latex modified Portland cement grout or an epoxy system according to the practicality of the situation. Detailed description of bonding coats has already been included with relevant repair materials.

C10-11.3.7 Acrylic concretes

Note: Description of this material is given as a matter of interest only. Its use requires training and special skills and repairs should best be entrusted to experts.

Unlike normal Portland cement concrete, acrylic concrete contains no water and no Portland cement. It is made by mixing aggregates with acrylic monomers that polymerise during curing to form hard, tough concrete. Two types of monomers are available for making acrylic concrete: methyl methacrylate (MMA) and high molecular weight methacrylate (HMWM).

Acrylic concrete is expensive, so obviously it cannot be used everywhere. However, because it develops a compressive strength of 35 MPa to 70 MPa in 1 to 2 hours, it has been used to repair pavements, bridge decks, parking decks and warehouse and factory floors that cannot be closed to traffic for several hours or days.

Acrylic concrete has few voids making it dense and impermeable. It resists intrusion of water, chlorides and most other corrosive chemicals. It develops high bond strength with Portland cement concrete. Therefore, it is often used as an overlay, 6mm to 50mm thick, on top of Portland cement concrete. A primer of catalysed acrylic monomer is usually brushed on the base concrete first. Because of its low viscosity, the acrylic penetrates into the pores of Portland cement concrete to produce a mechanical bond with the substrate.

The aggregate for making acrylic concrete should be bone dry, with a maximum moisture content of 0.5 percent. Normally, these products are sold as pre-packaged systems and manufacturer's instructions should be followed for mixing and application.

Both methyl methacrylate (MMA) and high molecular weight methacrylate (HMWM) gain strength in a matter of hours, but they are different in a few important ways. MMA has a low flash point, therefore, it is easily flammable, produces a non-toxic but disagreeable odour, and has a short pot life. HMWM, on the other hand, has a high flash point and its odour is not strong. HMWM is easier and safer to use than MMA but it also costs significantly more.

Because of its low viscosity, low volatility and relatively good bond strength, HMWM has been used without aggregate to weld together inactive cracks in Portland cement concrete. After mixing with a catalyst HMWM is poured onto the concrete surface and distributed with a squeegee. Material must be applied within 15 minutes after mixing. If cracks are blown clean and dry beforehand, the monomer can penetrate the full depth of the cracks. However, monomers should not be used to repair active cracks.

CAUTION

Personnel handling and mixing monomers should use eye-protection and impervious gloves and aprons; respirators with chemical filters should be available for those who wish to use them. Mixing of monomers should be carried out in a shaded, well-ventilated area, free of ignition sources. Storage and handling of all materials must be done in accordance with manufacturer's recommendations.

C10-11.3.8 Sprayed concrete

Note: Sprayed Concrete is a specialised work that requires skilled operators. Only an engineer with the knowledge of, and experience with, the material should decide where and how it should be used.

Sprayed concrete consists of a mixture of cement, aggregate and water (it may also contain fibres and/or other admixtures) forcefully projected onto a surface through a hose and nozzle by means of compressed air.

Sprayed concrete develops excellent bond, is homogeneous and compact and does not sag in wall and overhead applications. It is thus suited to a wide range of coating and lining operations.

There are two different techniques for applying sprayed concrete:

Dry-mix process

In the dry-mix process cement and aggregate are mixed together and metered into a regulated high-pressure hose. The compressed air carries the mixture to a special nozzle equipped with a controlled water spray that dampens the mixture and dispenses it on to the receiving surface. The volume of the water added is controlled by the nozzle operator.

Wet-mix process

In the wet-mix process a mixture of cement and aggregate mixed with water is metered into the delivery hose and conveyed towards the nozzle where compressed air is injected that then projects the mixture into place.

Concrete sprayed by the wet process has lower impact velocity than with the dry process because of inertia. Moreover, the extra workability required for pumping wet concrete dictates the use of higher water content. Strict control is required in the wet-mix process to ensure pumpable concrete at all times, otherwise if the delivery hose is blocked spraying has to be disrupted until the pipes can be cleared.

In the dry-mix process the quality of mix delivered on the surface relies heavily on the operator's competence as the rate at which water is added has to be controlled manually. With a skilled operator the dry-mix process does give better control and adjustment and thereby better quality. The dry-mix process is often the preferred technique.

In addition to suitability, the economics of the use of sprayed concrete must also be considered. For some applications, sprayed concrete may be more economical than conventional in-situ concrete because it needs little or no formwork nor any compaction, and the equipment for placing and mixing is small and portable.

Limitations

Sprayed concrete also has some limitations that should be kept in mind. These are:

- The finished product is largely dependent on the operator skill. Quality control, supervision and testing are difficult
- Correct batching of powdered admixtures in a dry-mix is very difficult. Some admixtures can also be hazardous to handle
- Sprayed concrete must be reinforced with small mesh, small diameter reinforcement (or fibre reinforcement added to the mix) to prevent drying shrinkage

- Dust from the dry-mix process can be objectionable. Protection must be provided for adjacent buildings, materials, trees and gardens
- Curing in sprayed concrete is more critical than in ordinary work because of the small thickness usually applied. If curing compounds are used, they should be applied at twice their usual rate
- Sprayed concrete may have shrinkage and thermal properties that are different to the concrete it is being applied to. To minimise the effect of differential shrinkage and thermal expansion latex admixtures and/or latex based coatings to substrate may be used, but keeping in mind the limited working time available with latex based products
- Sprayed concrete can have relatively high porosity and permeability
- Poor weather conditions such as wind, rain and cold can severely affect the application of sprayed concrete
- The overall thickness of horizontal, downward and vertical work is virtually unlimited (though for practical and economic reasons it is kept to less than 200mm). However, overhead work is generally limited to about 80mm thickness in one day
- Except for thin sections, the cost per cubic metre is generally higher than for in-situ work
- Sprayed concrete is difficult to finish. It is probably best not to trowel or float it

C10-11.3.9 Non-shrink hydraulic cement mortars

A number of proprietary Portland cement based products are available as non-shrink repair materials. Most of these products contain components that cause the mortar or concrete to expand after it has hardened. The expansion is intended to overcome or compensate for the expected drying shrinkage and to maintain a tight bond to the material with which it is in contact. Then, when it undergoes subsequent drying, the loss of moisture simply relieves the compressive stress instead of causing shrinkage.

Non-shrink cement mortars and grouts can be used in the following applications:

- Repairing honeycombing, shrinkage cracks and spalls
- Filling holes left by tie wires
- Making watertight seals around penetrations
- Stopping leaks
- Patching precast concrete
- Providing mortar beds under bearings and base plates
- Anchoring bolts, dowels and rods

It is best to purchase the non-shrink mortar in standard packages rather than in bulk. Proportioning with standard packages is simple: all the user needs to do is mix the package contents with the recommended amount of water and aggregates.

C10-11.3.10 Protective coatings

Surface coatings are used on concrete structures to provide additional protection against ingress of water, water soluble salts and atmospheric gases. In addition, they enhance the aesthetic appearance and help in hiding the patchy appearance of concrete that has been repaired in different places.

It should be noted however that when concrete is already showing signs of deterioration and tests show that enough salt is present at the reinforcement to make it rust, adding a protective coating will not help in reducing such deterioration. In such circumstances it is essential that the concrete be repaired first, and then, if the environment warrants, protected against further deterioration by application of a suitable surface treatment.

For whatever reason the coatings are applied it must be accepted that they will not last as long as a durable concrete surface and re-coating will be needed from time to time.

Basically, there are two types of protective coatings:

- Film forming - relying on adhesion to concrete
- Non-film forming - that penetrate into the concrete surface

Film forming coatings are made from:

- Polyurethane resins
- Epoxy resins
- Coal tar epoxy
- Chlorinated rubber
- Acrylics
- Bituminous materials
- Polymer modified cement

Penetration type coatings are formulated with Silane/Siloxane.

Generally, film forming coatings are highly efficient against ingress of moisture, water soluble salts (chlorides) and gases and vapours (carbon dioxide). However, build-up of water vapour pressure behind them, especially if water can get into the concrete from another face, can cause the coating to blister and peel off unless the adhesion of the film to the concrete is very good. Also, if the film lacks elasticity and fails to bridge across active cracks or subsequently formed shrinkage cracks, pollutants will find easy ingress into the concrete at the site of cracked coating and will eventually cause deterioration in the concrete.

Film forming coatings require a significant amount of surface preparation. The surface of concrete must be free of oils, grease, loose surface layers, dust and surface defects such as blow holes and shrinkage cracks. The surface is to be either sand-blasted or grit/water blasted and steam cleaned and imperfections filled with a suitable levelling or fairing coat prior to the application of the coating system. Most proprietary coating systems consist of a separate primer that improves the adhesion of the coating to the concrete and resists peeling or blistering. Also, pigmented coatings give much better protection and are more durable than unpigmented coatings.

Protective coating systems based on silane/siloxane are penetrating type sealers that impregnate concrete and react with the moisture and silicates present in the cement thus modifying the concrete surface to form a water repellent but vapour permeable barrier. Silane/Siloxane coatings, therefore, prevent contamination against water soluble chlorides but being vapour permeable are not effective against carbonation.

Penetration type coatings require less surface preparation. However, a dry concrete surface is essential for successful application. They generally impregnate the outer 2 to 5mm of the concrete surface and effectively seal blow holes, cracks up to 0.3mm and other minor irregularities.

All proprietary coatings perform differently from each other in regard to ease of application, adhesion to concrete, resistance against ingress of moisture, soluble salts and gases, durability, ability to stretch and bridge cracks and other characteristics. Their selection therefore should be made with care according to the requirements of the repair job in hand.

C10-11.3.11 Flexible joint sealants

Flexible joint sealants are required in repairing live cracks that are subject to movement due to applied loads, shrinkage and temperature. Such cracks cannot be sealed with rigid repair materials, such as epoxies, as failure would occur either in the repair material or a new crack will develop elsewhere in the concrete.

There are three types of sealants in general use – mastics, thermoplastics and elastomers. The properties of each type are given below.

- Mastics
 - Generally viscous liquids of non-drying oils or low melting asphalts
 - Movement capability not exceeding 15%
 - Sealant recess depth/width ratio up to 2:1
 - Extrude at high ambient temperatures
- Thermoplastics
 - Include asphalts, rubber modified asphalts, pitches and coal tar
 - Liquid or semi-viscous when heated
 - Pouring temperatures are usually above 100°C
 - Sealant recess depth/width ratio of 1:1
 - Movement capability up to 25%
 - Susceptible to ultra violet light, lose elasticity
 - Extrude at high ambient temperatures
- Elastomers
 - Polysulphides, epoxy polysulphides, polyurethanes, silicones and acrylics
 - One part or two part materials
 - Superior to other sealants
 - Excellent adhesion to concrete
 - Not susceptible to softening within normal range of ambient temperatures
 - Higher movement capability: up to 50% to 100%
 - Sealant recess depth/width up to 1:2

Choice of a sealant generally depends on the following factors:

- Movement capability of the sealant
- Resistance to environmental exposure (weather, ultra violet radiation, water penetration, chemicals, etc.)
- Adhesive properties, curing rate and paintability
- Service temperature range
- Sealant width/depth ratio (modulus characteristics)
- Trafficability
- Applicability in horizontal and vertical situations

A large number of proprietary sealants are available. The selection for any repair application should be made according to the particular requirements of the job. Manufacturer's printed instructions should be strictly followed.

C10-11.4 Questions to consider before choosing a repair material

Concrete repair materials can be formulated to provide a wide variety of properties. Because the properties affect the performance of the repair, choosing the right product requires careful study. Suppliers of repair materials can help repair contractors choose the right product but they need to

know the anticipated service conditions and the conditions under which the product will be applied. If you have not told the supplier the answers to the following questions, you probably have not provided enough information about the repair conditions for the supplier to help you choose the best repair material.

C10-11.4.1 Application conditions

How thick is the repair section?

In thick sections, heat generated during curing of the repair material may build up and produce unacceptable thermal stresses. Some materials may shrink too much when placed in thick layers and some materials will spall if placed in thin layers. Others can be feather edged. When aggregates are used as an inexpensive filler or extender, the maximum size of aggregate that can be used will be dictated by the minimum thickness of the repair.

Will the substrate be moist?

Some polymer materials will not cure properly in the presence of moisture. Others are moisture insensitive. Heat generated during initial curing of some repair materials may create steam in a moist environment and the steam may cause failure of the repair.

At what temperature will the repair material be placed?

Portland cement hydration ceases at or near freezing temperatures and cement modifier emulsions won't coalesce to form films at temperatures below about 5°C. Other repair materials can be used at temperatures well below freezing, although setting time may be increased. High temperatures will make many repair materials set faster, decreasing the working life or precluding their use entirely.

Will repairs be carried out in poorly ventilated areas or in areas where use of flammable materials is not permitted?

Components of some repair materials are volatile and combustible. Odour can also be a problem.

Is the repair section on a vertical surface or on the underside of a horizontal member?

For unformed wall or soffit patches, repair materials must adhere to the substrate without sagging.

C10-11.4.2 Service conditions

How soon does the repair have to be put into service?

If repairs are subject to early loading, rapid strength gain characteristics are essential.

Will the material be exposed to chemicals such as acids, sulphates, chlorides or strong solvents?

Acids and sulphates will attack Portland cement-based materials, and chlorides may cause corrosion of reinforcing steel. Strong solvents may soften some materials.

Will the repair bear heavy traffic?

A material with good abrasion resistance and good skid resistance may be necessary.

Must the material bond to steel as well as to the concrete substrate?

What are the maximum and minimum temperatures that the material will be subjected to in service?

Because thermal movements can cause stresses in the repair material or at the bond surface, the range of service temperatures must be considered.

Will the finished repair be exposed to vibration?

For applications such as machinery pedestals, vibration can cause distress in brittle repair materials.

Is appearance of the repair important?

If colour matching or duplication of original concrete texture is needed, many repair materials will be unsuitable.

How long must the repair last?

If the repair is only temporary, perhaps a lower cost, less durable, or more easily applied material can be used.

C10-12 Repair options for concrete structures

C10-12.1 *Establish need for repairs*

Before proceeding with the repairs an evaluation should be made to determine the need for repairs. Repairs may be required for any of the following reasons:

C10-12.1.1 *Strength, stiffness and durability*

Damage needs to be repaired if it reduces the strength, stiffness or durability of the structure to an unacceptable level. For example impact damage to beams and columns or loss of reinforcement by corrosion. In such cases calculations should be carried out to determine the stress levels and deformations in the damaged elements to assess the severity of damage and urgency for repairs.

C10-12.1.2 *Appearance*

Repairs may be required to improve the appearance of the concrete surface, for example some types of cracks, minor spalls, scaling, efflorescence, impact damage, etc. These defects may not immediately affect the strength of the structure but if left untreated could lead to further deterioration.

C10-12.1.3 *Functional performance*

Repairs are required if the function of the structure is impaired even if the strength, stiffness or appearance are not significantly affected. Examples include broken treads and handrails on stairs, loss of sealants from expansion joints, dampness due to ponding of water, etc.

C10-12.1.4 *Prevention of further deterioration*

Some repairs need to be undertaken to prevent the existing minor deterioration growing worse if left unattended. For example: evidence of chloride penetration without corrosion may require coating the concrete surface with silane to limit further chloride ingress.

C10-12.2 *Repair options*

Depending on the nature of damage, urgency of repairs and availability of funds and resources, one of the following options may be adopted:

- Do nothing other than carry out regular safety inspections. Wait and see.
- Take action to prevent the deterioration from getting worse.
- Carry out repairs to restore deteriorating parts of the structure to a satisfactory condition.
- Demolish and re-build all or part of the structure.

C10-12.3 *Selection of repair methods*

Having determined the repair option, it then remains to select an appropriate repair method for the job in hand.

Selection of the repair method depends on the location, type and extent of deterioration and the cause thereof. For a particular defect there may be one or more possible repair solutions. Therefore, a comprehensive knowledge of repair methods and procedures is essential. These are given in the Chapter 13 of CRN CM 303.

C10-13 Repair materials for masonry structures

C10-13.1 General

Masonry is constructed from units of brick, stone or concrete joined together with cement or lime mortar.

In masonry repairs the most important material is the mortar. It is essential to understand its function in masonry work and the basic principles that determine the selection of a "strong" or "weak" mortar in repair work.

The other materials used in repairing masonry are:

- Polymers (synthetic latexes) for modifying mortars
- Synthetic resins
- Concrete and grouts
- Sealants

The above materials are described in Section C10-11.3.

Sometimes bricks, stone and concrete blocks are also needed to replace damaged masonry. The basic criterion for their selection is to match them in colour, texture and strength with the existing work.

C10-13.2 Function of mortar

The purpose of mortar in masonry work is twofold:

- To seal the joints between masonry units
- To provide a bed for the units so that the loads are evenly distributed across the joints

The strength and rigidity of the masonry is mainly dependent on the strength of the brick, stone or concrete units rather than the mortar. In fact, a very strong mortar can do more harm than good.

C10-13.3 Problems with strong mortars

Two difficulties arise with strong, rich mortars in masonry:

C10-13.3.1 Cracking

Cracking is not usually attributable to directly applied loads, but is generally caused by differential or thermal movements between the various parts of the structure as a result of thermal or shrinkage movement or foundation settlement.

When a strong mortar is used, fine cracks develop between the mortar and the masonry unit (brick, stone or concrete) that, as well as looking unsightly, may pass right through the masonry units and permit the passage of water.

A weak mortar, on the other hand, permits the masonry work some freedom to absorb movements without obvious cracking. Where cracking does occur, it will tend to be distributed through the joints where it is comparatively easy to repair, rather than through the masonry units themselves.

Thus the resistance of masonry to cracking is improved when weak mortars are used.

C10-13.3.2 Fretting

Fretting occurs when there has been continued evaporation from the wall surfaces. With weak, porous mortars evaporation occurs mainly along the joints. But if a strong, impermeable mortar is used, the evaporation occurs from the masonry blocks and leads to spalling of bricks and fretting of sandstone. It is easier to repair the fretting of weak and porous mortar by repointing than to replace fretted bricks or stone blocks.

It is therefore better to use weak and porous mortar that can be repointed if necessary rather than to use strong mortars and having to re-build the wall.

C10-13.4 Importance of lime in mortars

The addition of hydrated lime to the cement improves the mortar in a number of ways. It makes the mortar "soft", so it is then more able to absorb movements in the masonry and therefore minimises cracking. Lime in the mortar improves its workability - the mortar "comes off the trowel" more easily. It also improves its bond and its capacity for self-healing of cracks.

C10-13.5 Basic principles

In general, the mortar should be slightly weaker and more permeable than the masonry units. For sandstock brick or sandstone, a cement:lime:sand mix (by volume) of 1:2:9 to 0:1:2 is suitable. For normal bricks, 2:1:12 to 1:1:6 can be used.

In all cases, just enough water should be added to attain a workable consistency of the mortar.

Do not use a clay-sand soil unless following a proven local tradition. Do not use crushed stone unless it has been successfully used as building stone. Avoid any sand, clay or crushed stone that may contain salts.

Chapter 11 Certifying structures after maintenance and minor installation

C11-1 Introduction

This section details the requirements for certifying structures as safe for continuing train operations after maintenance and minor installation work.

Structure maintenance certification is the process of determining that a structure is safe for the operation of trains when doing work on the structure that affects or may affect track system integrity.

Persons undertaking certification activities must:

- Understand the requirements of the work and the impact of the activity on the safe and reliable operation of rail traffic over, under or past the structure
- Visually inspect and assess relevant aspects of infrastructure condition or installation work against the authorised design
- Certify the structure with appropriate restrictions if necessary

Certification of any work that affects or may affect the integrity of the track system also requires certification of the track. Refer to CRN Engineering Manual CRN CM 211 “Track Geometry & Stability”.

C11-2 What to Certify

Structure certification is required for work that affects or may affect track system integrity such as:

- Repair structures: bridges, tunnels, retaining walls, platforms, level crossings, buffer stops, track slabs
- Install/replace transoms
- Install minor structures: pipes, culverts, level crossings, buffer stops, ballast retention walls
- Install temporary track support

Certification is not required for routine maintenance of structures such as:

- Clearing vegetation and debris from and around structures/components
- Cleaning drainage weepholes
- Tightening of bolts
- Servicing of bridge bearings
- Patch painting of steel

C11-3 How to Certify

C11-3.1 Planning

Establish what activity you are certifying. It may be a simple activity or a combination of activities.

Consider the impact of the work on the safety and reliability of the infrastructure if the work is not undertaken correctly, either during or after the work:

- Effect of work on structures integrity
- Effect of work on the capacity of the structure to avoid collapse on to the track

C11-3.2 Structures repair work

To assess the integrity of the repair work, consider the following issues:

- Is there evidence that excavations around the structure have weakened the structure support?
- Did the work involve installation of temporary support?
- Was the temporary support constructed to an authorised design?
- Has the temporary support been properly installed?
- Was the repair done to an authorised design or in accordance with the repair manual?
- Do the repair materials meet design requirements?
- Are all fastenings/connections tight & secure?
- Are there obvious deficiencies with the completed repairs?
- Is transom condition and support affected by repairs to superstructure components?
- Does the work method have the potential to create future instability?
- Is the capacity of the structure adequate to avoid collapse of structure?
- Is the structure reinstated to design location? (friction buffer stop).

If the repair has not fully restored the strength and serviceability of the structure, either to the "as new" condition, or to the condition that is required for current or envisaged use ("fit for purpose"), the defect is to remain in the Defect Management System (Infor). A new defect category and a new repair priority are to be allocated.

C11-3.3 Install transoms

To assess the integrity of the transom installation, consider the following issues:

- Do the transoms meet design requirements?
- Were the transoms installed to an authorised design or in accordance with the installation manual?
- Are all fastenings/connections tight & secure?
- Are there obvious deficiencies with the completed installation?
- Are transoms packed to correct line and level?
- Does the work method have the potential to create future instability?

C11-3.4 Install minor structures

To assess the integrity of the installation of minor structures, consider the following issues:

- Is there evidence that excavations around the structure have weakened the structure support?
- Was the installation done to an authorised design or in accordance with the specifications?
- Do the installed components meet design requirements?
- Are all fastenings/connections tight & secure?
- Are there obvious deficiencies with the completed installation?
- Does the work method have the potential to create future instability?
- Is the capacity of the structure adequate to avoid collapse of structure?

C11-3.5 Install temporary track support

To assess the integrity of the temporary track support installation, consider the following issues:

- Is there evidence that excavations around the structure have weakened the structure support?



- Was the temporary support constructed to an authorised design?
- Has the temporary support been properly installed?
- Does the work method have the potential to create future instability?
- Is the capacity of the structure adequate to avoid collapse of structure?

Chapter 12 Certifying structures after examination

C12-1 Introduction

This section details the requirements for assessing and certifying structures as safe for continuing train operations after structures examination.

The regular assessment of structures achieves the following objectives:

- To assess the current condition of a structure
- To determine the need for an engineering rating of the load carrying capacity of structures
- To certify structures safe for the continued operation of rail, road and pedestrian traffic with appropriate restrictions if necessary

C12-2 What to Assess

Structures assessment is required for all structures examined under the structures examination system.

C12-3 When to Assess

Structures are assessed after:

- Detailed examinations
- Special examinations
- Underwater examinations
- Monthly examinations of broad flange beams over roads

For cyclic examinations, the assessment shall be carried out by the Structures Structures Superintendent within one month of the receipt of the examination report.

For impact damage, the assessment shall be carried out within the assessment timeframes specified for the standard defect categories (refer to CRN CM 302).

C12-4 How to Assess

C12-4.1 General

The assessment can be based on the review of the examination report.

If the information in the report is insufficient to permit assessment, then a site inspection is to be carried out.

The assessment will confirm the defect categories, repair priorities, and paint indices where applicable.

C12-4.2 After detailed examination

To assess whether structure integrity has been adversely affected, consider the following issues:

- What defects exist?
- What is the size of the defects?
- What is the maximum defect category as described in CRN CM 302? A, B, C, D or E?
- Are the defect categories correct?
- Are any of the defects Category A, B or C?
- If A or B, seek engineering advice from the Principal Track and Civil Engineer. Based on the recommendations of an authorised design engineer, certify the structure subject to any restrictions recommended by the engineer. Restrictions may be speed or load related.

- If C, seek engineering advice from the Civil Maintenance Engineer (DE). Based on the recommendations of the DE, certify the structure subject to any restrictions recommended by the DE. Restrictions may be speed or load related. The DE may seek advice from the Principal Track and Civil Engineer.
- If D,
 - What components have defects?
 - Identify the structurally critical components?
 - Do any structurally critical components have defects?
 - Do structurally critical components have multiple defects?
 - Is the load carrying capacity of the structurally critical component likely to be reduced as a result of the defects? In bending? In shear?
 - Reduced by how much? A small amount? A large amount?
 - Is the member in tension or compression?
 - What are the structural interactions between this member and other members?
 - What is the crack growth rate?
 - What is the degradation rate of the defect?
 - What is the load rating of the bridge?
 - What is the minimum allowable load rating applicable to structures on that corridor?
 - Effect of defects on rating of structure?
 - Assess any repair/refurbishment work carried out since the last examination. Is work to standard?
 - What is the material type: steel/ reinforced concrete/ pre-stressed concrete/ masonry?
 - Steel: primary load carrying members and secondary member?
 - Condition of prestressing tendons?
 - Cover to reinforcement? Spalling?
 - If necessary seek engineering advice from the DE or the Principal Engineer Track & Civil
 - Certify the structure subject to any speed or load related restrictions
- If E, certify the structure
- If no defects, certify the structure

Also refer to Service Schedule CSS 234 "Structures Assessment" in CRN Engineering Manual CRN CM 101 "Structures Service Schedules".

C12-4.3 After special examination

To assess whether structure integrity has been adversely affected, consider the following issues:

- What damage exists?
- Does the damage create defects?

If yes, follow the process in C12-4.2.

If no, certify the structure.

C12-5 After underwater examination

Follow the same process as for detailed examination.

C12-6 After monthly broad flange beam examination

Follow the same process as for detailed examination.

For new damage, follow the same process as for special examination.

C12-7 How to Certify

Allocate defect categories, repair priorities and paint indices.

Certify the structure after the assessment by signing the examination form.

Ensure that the final defect categories, repair priorities and paint indices are entered into the Defect Management System (Infor).