



Bridge design

Part 8: Rehabilitation and strengthening of existing bridges



This Australian Standard® was prepared by Committee BD-090, Bridge Design. It was approved on behalf of the Council of Standards Australia on 15 March 2017. This Standard was published on 31 March 2017.

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 - Bureau of Steel Manufacturers of Australia
 - Cement and Concrete Association of New Zealand
 - Cement Concrete & Aggregates Australia—Cement
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-

This Standard was issued in draft form for comment as DR AS 5100.8:2016.

Standards Australia wishes to acknowledge the participation of the expert individuals that contributed to the development of this Standard through their representation on the Committee and through the public comment period.

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Australian Standard[®]

Bridge design

Part 8: Rehabilitation and strengthening of existing bridges

First published as AS 5100.8:2017.

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Published by SAI Global Limited under licence from Standards Australia Limited, GPO Box 476, Sydney, NSW 2001, Australia

ISBN 978 1 76035 721 4

PREFACE

This Standard was prepared by the Standards Australia Committee BD-090, Bridge Design, in response to numerous requests from industry, designers and representatives in the field of Bridge Design, especially those in the area of rehabilitation and strengthening of existing bridges.

This Standard is also designated as Austroads publication AP-G51.8-17.

The requirements of the AS(AS/NZS) 5100 series are based on the principles of structural mechanics and knowledge of material properties, for both the conceptual and detailed design, to achieve acceptable probabilities that the bridge or associated structure being designed will not become unfit for use during its design life.

The objectives of the AS(AS/NZS) 5100 series are to provide nationally acceptable requirements for—

- (a) the design of road, rail, pedestrian and cyclist-path bridges;
- (b) the specific application of concrete, steel, timber and composite construction, which embody principles that may be applied to other materials in association with relevant Standards; and

The objective of this Part (AS 5100.8) is to provide requirements for the assessment of the load capacity, and for the strengthening and rehabilitation of existing bridges.

In line with Standards Australia policy, the words ‘shall’ and ‘may’ are used consistently throughout this Standard to indicate respectively, a mandatory provision and an acceptable or permissible alternative.

Statements expressed in mandatory terms in Notes to tables are deemed to be requirements of this Standard.

The terms ‘normative’ and ‘informative’ have been used in this Standard to define the application of the appendix to which they apply. A ‘normative’ appendix is an integral part of a Standard, whereas an ‘informative’ appendix is only for information and guidance.

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STANDARDS AUSTRALIA

Australian Standard Bridge design

Part 8: Rehabilitation and strengthening of existing bridges

SECTION 1 SCOPE AND GENERAL

1.1 SCOPE

This Standard sets out minimum requirements and procedures to—

- (a) repair or rehabilitate a structure in order to restore it to its original or intended level of service;
- (b) extend the remaining service life of the structure;
- (c) strengthen a structure (because of structural deterioration) to restore its original capacity;
- (d) strengthen a structure to increase its capacity for live load (such as in response to a proposed increase in vehicle loads), barrier loading, collision loading, earthquake loading and other loading;
- (e) change the function of a structure (such as a road bridge to a pedestrian/cyclist bridge);
- (f) widen an existing structure; or
- (g) any combination of the above.

Structures covered in this Part of the AS(AS/NZS) 5100 series include existing bridges (both superstructure and sub-structure) approach slabs, wing walls and buried culverts.

NOTES:

- 1 Rehabilitation or strengthening may not be feasible for those structures that—
 - (a) have substandard horizontal and vertical clearances;
 - (b) have a poor alignment both on the structure and on the approaches;
 - (c) have extensive deterioration of the substructure, including active scour undermining, pronounced seismic vulnerability, and/or questionable foundations;
 - (d) have numerous deficiencies throughout the superstructure and/or substructure;
 - (e) have deficient in situ strength; or
 - (f) have structural systems that are non-redundant or incorporate poor details that require increased maintenance and inspection.
- 2 All the requirements of this Standard may not be applicable to emergency response situations.

1.2 NORMATIVE REFERENCES

The following are the normative documents referenced in this Standard:

NOTE: Documents referenced for informative purposes are listed in the Bibliography.

AS

- | | |
|-----------|---|
| 1012 | Methods of testing concrete |
| 1012.8.3 | Method 8.3: Methods of making and curing concrete—Mortar and grout specimens |
| 1012.9 | Method 9: Compressive strength tests—Concrete, mortar and grout specimens |
| 1012.14 | Method 14: Method for securing and testing cores from hardened concrete for compressive strength |
| 1012.20.1 | Method 20.1: Determination of chloride and sulfate in hardened concrete and aggregates—Nitric acid extraction method |
| 1012.20.2 | Method 20.2: Determination of water-soluble chloride in aggregates and hardened concrete |
| 1012.21 | Method 21: Determination of water absorption and apparent volume of permeable voids in hardened concrete |
| 1012.24 | Method 24: Determination of the tensile bond strength of concrete—Repairs and strengthening systems |
| 1065 | Non-destructive testing—Ultrasonic testing of carbon and low alloy steel forgings |
| 1171 | Non-destructive testing—Magnetic particle testing of ferromagnetic products, components and structures |
| 1214 | Hot-dip galvanized coatings on threaded fasteners (ISO metric coarse thread series) |
| 1391 | Metallic materials—Tensile testing at ambient temperature |
| 1544 | Methods for impact tests on metals |
| 1544.2 | Part 2: Charpy V-notch |
| 1597 | Precast reinforced concrete box culverts |
| 1597.2 | Part 2: Large culverts (exceeding 1200 mm span and up to and including 4200 mm span and 4200 mm height) |
| 1627 | Metal finishing—Preparation and pretreatment of surfaces |
| 1627.4 | Part 4: Abrasive blast cleaning of steel |
| 1710 | Non-destructive testing—Ultrasonic testing of carbon and low alloy steel plate and universal sections—Test methods and quality classification |
| 1816 | Metallic materials—Brinell hardness test |
| 1816.1 | Method 1: Test method (ISO 6506-1:2005, MOD) |
| 1817 | Metallic materials—Vickers hardness test |
| 1817.1 | Method 1: Test method (ISO 6507-1:1997, MOD) |
| 2028 | Methods for the measurement of the depth of hardening in flame and induction hardened steel products |
| 2062 | Non-destructive testing—Penetrant testing of products and components |
| 2177 | Non-destructive testing—Radiography of welded butt joints in metal |
| 2205 | Methods for destructive testing of welds in metal |
| 2205.5.1 | Method 5.1: Macro metallographic test for cross-section examination |
| 2207 | Non-destructive testing—Ultrasonic testing of fusion welded joints in carbon and low alloy steel |

AS	
2331	Methods of test for metallic and related coatings
2331.1.4	Method 1.4: Local thickness tests—Magnetic induction and eddy current methods
2350	Methods of testing portland, blended and masonry cements
2350.13	Part 13: Determination of drying shrinkage of cement mortars
2452	Non-destructive testing—Determination of thickness
2452.3	Part 3: Use of ultrasonic testing
2574	Non-destructive testing—Ultrasonic testing of steel castings and classification of quality
2832	Cathodic protection of metals
2832.2	Part 2: Compact buried structures
2832.3	Part 3: Fixed immersed structures
2832.5	Part 5: Steel in concrete structures
3507	Non-destructive testing
3507.2	Part 2: Radiographic determination of quality of ferrous castings
3799	Liquid membrane-forming curing compounds for concrete
3894	Site testing of protective coatings
3894.3	Method 3: Determination of dry film thickness
3894.9	Method 9: Determination of adhesion
3978	Non-destructive testing—Visual inspection of metal products and components
4361	Guide to lead paint management
4361.1	Part 1: Industrial applications
4544	Non-destructive testing—Eddy current testing for the detection of surface flaws—Ferromagnetic and non-ferromagnetic metallic products
5100	Bridge design
5100.1	Part 1: Scope and general principles
5100.2	Part 2: Design loads
5100.3	Part 3: Foundations and soil supporting structures
5100.4	Part 4: Bearings and deck joints
5100.5	Part 5: Concrete
5100.7	Part 7: Bridge assessment
5100.9	Part 9: Timber bridges
SA TS	
101	Design of post-installed and cast-in fastenings for use in concrete
AS/NZS	
1050	Methods for the analysis of iron and steel (series)
1252	High-strength steel fastener assemblies for structural engineering—Bolts, nuts and washers (series)
2041	Buried corrugated metal structures
2041.1	Part 1: Design methods
1554	Structural steel welding
1554.1	Part 1: Welding of steel structures
1554.3	Part 3: Welding of reinforcing steel
1554.4	Part 4: Welding of high strength quenched and tempered steel
1554.5	Part 5: Welding of steel structures subject to high levels of fatigue loading

AS/NZS

- 2566 Buried flexible pipelines
- 3678 Structural steel—Hot-rolled plates, floorplates and slabs
- 3679 Structural steel (series)
- 3725 Design for installation of buried concrete pipes
- 4058 Precast concrete pipes (pressure and non-pressure)
- 5100 Bridge design
- 5100.6 Part 6: Steel and composite construction

ASTM

- A247 Standard Test Method for Evaluating the Microstructure of Graphite in Iron Castings
- A956 Standard Test Method for Leeb Hardness Testing of Steel Products
- C295 Standard Guide for Petrographic Examination of Aggregates for Concrete
- C597 Standard Test Method for Pulse Velocity Through Concrete
- C805 Standard Test Method for Rebound Number of Hardened Concrete
- C1383 Standard Test Method for Measuring the P-Wave Speed and the Thickness of Concrete Plates Using the Impact-Echo Method
- D4788 Standard Test Method for Detecting Delaminations in Bridge Decks Using Infrared Thermography
- E407 Standard Practice for Microetching Metals and Alloys
- E1139 Standard Practice for Continuous Monitoring of Acoustic Emission from Metal Pressure Boundaries

BS

- 6319 Testing of resin and polymer/cement compositions for use in construction
- 6319-2 Part 2: Method for measurement of compressive strength
- 6319-3 Part 3: Methods for measurement of modulus of elasticity in flexure and flexural strength
- 6319-7 Part 7: Method for measurement of tensile strength

1.3 DEFINITIONS

For the purposes of this Standard, the definitions in AS 5100.1 and those below apply.

1.3.1 Aerial weight

The weight of active fibre used for strengthening, not including fibres used to hold the active fibre together, measured in grams per square metre (g/m^2).

1.3.2 Alkali aggregate reaction

Disruptive chemical reaction within the concrete matrix between the alkali hydroxides (sodium and potassium) and susceptible aggregates (characterized by unstable silica minerals) in the presence of moisture, which produces an expansive alkali silica gel which subsequently exerts excessive tensile stresses that results in cracking of the concrete structure.

1.3.3 Anode

A metal that is placed in or in contact with an electrolyte and connected to the steel reinforcement to apply cathodic protection (CP) to a reinforced concrete structure.

1.3.4 Bridge specific assessment live loading (BSALL)

A specific live load developed by probability methodology for a particular bridge using a combination of traffic surveys, forecast traffic patterns, weigh in motion data, weighbridge static surveys and a related database of vehicle dimensions and axle weights.

1.3.5 Cathodic protection (CP)

The prevention or reduction of corrosion of steel in concrete by the distribution of sufficient current to the metal (i.e. steel reinforcement and tendons), to make it the cathode in a galvanic or electrolytic cell.

1.3.6 Cathodic protection, galvanic (sacrificial)

Protection system consisting of a metal less noble or lower in the galvanic series (different electropotentials), which when connected to steel produces a galvanic cell and corrodes in place of the steel to protect the structure.

1.3.7 Cathodic protection, impressed current

Direct current supplied by an external power source to protect cathodically a structure.

1.3.8 Condition assessment

The process of reviewing records, investigating and evaluating the need for maintenance, repair, rehabilitation or strengthening of a structure by undertaking a condition survey to determine the cause, extent and degree of any distress, deterioration or deficiency, and which may include load capacity calculations.

1.3.9 Condition survey

The process of establishing the physical condition of a structure by utilizing a combination of diagnostic techniques, including visual inspection, non-destructive testing, sampling and laboratory testing.

1.3.10 Corrosion of steel reinforcement

Deterioration of steel reinforcement in concrete when the passivity (pH of about 12) around the steel, which protects it, is lost or breaks down, or when the concrete is cracked or delaminated sufficiently to allow easier access of water and oxygen.

1.3.11 Corrosion of structural steel

The deterioration of steel by chemical or electro-chemical reaction resulting from, but not limited to, exposure to air, moisture, sea spray, industrial fumes and other chemicals and contaminants in the environment in which it is placed.

1.3.12 Crack

A complete or partial separation of concrete into two or more parts with or without a readily visible opening produced by breaking or fracturing.

1.3.13 Culvert

A concrete, corrugated steel, corrugated aluminium and/or other material drainage and/or underpass structure with clear spans (or design diameters) extending from 1800 mm and up to 6000 mm.

1.3.14 Delamination

Separation, splitting, debonding or cracking within a concrete component along a plane generally parallel to a surface, as in the separation or delamination at the reinforcing steel caused by corrosion, or debonding of the fibre-reinforced polymer (FRP) which is usually detected by tapping with a hammer producing a hollow sound or by non-destructive testing.

1.3.15 Design life of repair/strengthening

The period of time for which a repair or strengthening system is required to perform its required function based on the design and specified parameters, assuming continued exposure to given in-service conditions.

1.3.16 Deterioration

Manifestation of defects or failure of a component or associated material (cracking, delamination, spalling, staining, etc.), due to influencing factors of an internal and/or external origin.

NOTE: Factors of an internal origin may be linked to the various constituents and chemical reactions in which they participate (e.g. alkali aggregate reaction, corrosion), whereas factors of an external origin may be controlled by the in-service conditions, the technology and physical properties of the material, construction practices and potential structural effects.

1.3.17 Ductility

The ability of a structure or structural element to undergo large deformation prior to collapse.

1.3.18 Effective thickness

The effective thickness for design of fibre-reinforced polymer (FRP) strengthening calculated by dividing the fibre aerial weight by the fibre density, in the direction of the action being resisted.

1.3.19 Electrolyte

The liquid component in a composite material, such as soil or concrete, in which electric current may flow by ionic charge transfer.

1.3.20 Energizing

The process of the initial application of power to an impressed current cathodic protection system.

1.3.21 Fibre fabric

A material formed from fibres orientated in the required direction in a woven or non-woven form and made available in blankets of various widths, weights, thicknesses, and lengths.

1.3.22 Fibre laminate

Fibre-reinforced polymer composite comprising two or more layers of fibres, bound together in a resin matrix and produced in the form of a sheet or plate of required widths, thicknesses and lengths.

1.3.23 Fibre reinforced polymer (FRP)

Composite material comprised of a resin matrix such as epoxy or other adhesive materials reinforced with high strength fibres such as carbon, aramid or glass, moulded and hardened to form the required shape including fibre fabric and laminates, which can act in composite to enhance the capacity of concrete structures.

1.3.24 Maintenance

The ongoing upkeep of a structure undertaken during its service life, comprising routine, repetitive and preventative tasks and repair actions, aimed at delaying damage or deterioration and retaining the structure in a state in which it can perform its required functions under acceptable conditions of serviceability and safety.

1.3.25 Masonry

The building of structures from individual units laid in and bound together by mortar.

NOTES:

- 1 The common materials of masonry for engineering construction are clay brick and stone.
- 2 Masonry does not include concrete brick and blockwork.

1.3.26 Near surface mounted FRP

FRP that is installed into slots that have been prepared in the cover region of the concrete.

1.3.27 Post-tensioning

Method of prestressing in which internal or external prestressing tendons or bars are tensioned after the concrete has hardened.

1.3.28 Post-tensioning, external

Post-tensioned application in which tensile forces in bars or tendons are maintained through anchorages at each end, external to the hardened concrete.

1.3.29 Protective coating systems

Protective coatings, waterproofing membranes and similar systems that may be included in the repair process to extend the service life of the repairs and slow down further deterioration in the structure.

NOTE: Protective coating systems may offer different degrees of protection to the repair and the overall structure and should be selected on their merit to satisfy specific serviceability requirements, assuming continued exposure to given in-service conditions.

1.3.30 Reference electrode

A monitoring sensor (i.e. Ag/AgCl, Mn/MnO₂, Cu/CuSO₄) which is permanently fixed (embedded) within an electrolyte, such as concrete, and close to the steel reinforcement, and used to measure the electrical potential of that portion of the steel reinforcement.

1.3.31 Rehabilitation

The process of restoring a structure (or part thereof) to its original or required level of service by undertaking repairs or modifications that are remedial in nature, and which are less frequent than maintenance activity (see also Clause 1.3.24).

1.3.32 Repair

The process of restoring a damaged or deteriorated structure or structural component to a serviceable and operating condition by replacing or rectifying faulty materials, components, or elements.

1.3.33 Resin

A resin system based on epoxy materials, comprising a thermosetting polymer that is the reaction of an epoxy resin and an amino hardener.

NOTE: A resin system may be in the form of primer, adhesive, saturating resin and putty filler and used as components of the relevant FRP system (see Paragraph A2.3, Appendix A).

1.3.34 Seasoned timber

Timber in which the average moisture content is nominally between 10% and 15%.

1.3.35 Service condition

The combination of environmental exposure and various load effects that a structure or material may be subjected to during its design life.

1.3.36 Service life of repair/strengthening

The period of time for which a repair or strengthening system is expected to remain in a useful and serviceable state, prior to the onset of distress or deterioration, assuming continued exposure to given in-service conditions.

1.3.37 Service life of existing structure

The remaining useful life of a structure during which it is expected to remain in a serviceable and operational state, based on the current rate of deterioration or distress, assuming continued exposure to given in-service conditions without repairs.

1.3.38 Strengthening

The process of increasing the capacity of a structure or structural components in terms of increased flexural, shear or axial strength or ductility or restoring the capacity of weakened elements of structure to their original design capacity, using either active or passive strengthening systems or both.

1.3.39 Strengthening, active

Systems that engage the structure or structural component in composite action immediately upon installation, through the introduction of an external force, such as the application of anchored reinforcing or post-tensioning systems and the construction of deck overlays, section enlargement and the like.

1.3.40 Strengthening, passive

Systems that engage the structure or a structural component in composite action only when additional loads, beyond those existing at the time of installation, are applied to the structure, including steel plate or fibre-reinforced polymer (FRP) composite systems bonded to structural members.

1.3.41 Surface bonded FRP

FRP that is directly bonded to the outside surface of the concrete.

1.3.42 Transformer/rectifier (T/R)

An electrical power unit that transforms and rectifies a.c. line voltages to the required lower d.c. output voltages; also referred to as a 'd.c. power supply'.

1.3.43 Unseasoned timber

Timber shall be considered to be unseasoned where the least dimension of the member exceeds 100 mm.

1.3.44 Zone

A discrete part of a reinforced concrete structure and/or CP system characterized by different environmental exposure conditions (i.e. submerged, tidal, splash, atmospheric zones) within its overall in-service environment.

1.4 NOTATION

The symbols used in this Standard are listed below.

Unless contrary intention appears, the following applies:

- The symbols used in this Standard shall have the meanings ascribed to them below, with respect to the structure, or member, or condition to which a clause is applied.
- Where non-dimensional ratios are involved, both the numerator and denominator are expressed in identical units.
- The dimensional units for length, force and stress in all expressions or equations are to be taken as millimetres (mm), newtons (N) and megapascals (MPa) respectively.

An asterisk (*) placed after a symbol as a superscript (e.g. M^*) denotes a design action effect due to the design load.

Symbol	Definition
b_f	fibre width
b_w	beam width or plate spacing for continuous slab
C_E	environmental reduction factor
E_{cj}	mean value of the modulus of elasticity of concrete at the appropriate age
E_{fd}	elastic modulus of fibre
f'_{ct}	characteristic tensile strength of concrete; <i>or</i> $0.4\sqrt{f'_c}$
l_t	FRP bond length
$l_{t,max}$	maximum anchorage length
T_a	force that will be developed in the FRP at the point of termination
T_k	force in the FRP in the anchorage zone
$T_{k,max}$	maximum ultimate bond force
t_f	total FRP thickness of all layers
ϕM_{uo}	design bending capacity of an FRP strengthened beam

1.5 DURABILITY CONSIDERATIONS

Repairs, rehabilitation and strengthening shall be designed and constructed to achieve a high level of durability and to be able to withstand the expected wear and deterioration throughout their intended service life, whilst achieving the required serviceability and strength limit state condition.

The design and construction of repairs, rehabilitation and strengthening shall adhere to the required techniques, practices and Standards, and shall consider surface exposure and environment, adequate detailing and accessibility for future inspection and maintenance, material properties and the quality of the works.

The design life of repairs, rehabilitation and strengthening shall be based on a lifecycle approach, and shall take into account repeat applications over the lifecycle, future maintenance and repeat access requirements and take into account the residual life of the existing structure.

1.6 SERVICE ENVIRONMENTS FOR BRIDGE STRUCTURES

Bridges and other associated structures can reside in a variety of service environments, characterized by various degrees of severity of exposure specified in the AS(AS/NZS) 5100 series.

When assessing the condition or managing the repair, rehabilitation or strengthening of a structure consideration shall be given to the various mechanisms and forms of deterioration that may be associated with the prevailing in-service environments, including localized microclimate conditions.

Microclimate conditions shall be examined on a case-by-case basis, including, for example, localized water run-down beneath scuppers or blocked drains, and local collection patterns due to the geometry of structural members and prevailing wind speed and direction.

1.7 MATTERS FOR RESOLUTION BEFORE DESIGN COMMENCES

The matters for resolution listed below shall be confirmed as accepted by the relevant authority or owner of a bridge or associated structure before commencing the design process:

- 1 Life expectancy of the particular rehabilitation or strengthening method and materials (see Clause 2.1).
- 2 Design live loading including the use of BSALL (see Clauses 2.3.1, 2.3.2 and Paragraph E1, Appendix E).
- 3 Procedure to measure distribution of carbonation depths using aqueous phenolphthalein solution (see Clause 3.2.5).
- 4 Suitability of a structure for CP system installation (see Clause 3.3).
- 5 Depth and extent of concrete removal for repair (see Clause 3.5.3.4.2).
- 6 Use of encapsulation paints (see Clause 4.5.1).
- 7 Use of non-metallic materials for bolted anchorages of deck joints [see Clause 8.4.1(i)].
- 8 The need to upgrade or replace traffic barriers (see Clause 9.5).
- 9 Design standard and return intervals for the modification of an existing culvert (see Clause 10.2.3).
- 10 Pull off test results where mean bond strengths are less than 1.5 MPa (see Paragraph A4.3.4).
- 11 Assessment of material performance data and design values for fibre fabric, laminates and adhesive resins (see Paragraph A4.3.5.1, Appendix A).
- 12 Alternative design methods for assessment of the capacity of an FRP strengthened beam.
- 13 Power source for the operation and maintenance of CP systems (see Paragraph B11, Appendix B).

1.8 APPROPRIATE SUPERVISION

The assessment, design and installation of the work items described in this Standard shall be undertaken under the supervision of a professional engineer with appropriate knowledge and experience in the specific field of application. Suitably competent and qualified personnel shall undertake specialist testing and inspection functions and the installation of specialist materials and construction processes.

SECTION 2 DESIGN REQUIREMENTS AND PROCEDURES

2.1 GENERAL

During the planning and design stages of bridge rehabilitation or strengthening, special consideration shall be given to, but not limited to, the following:

- (a) Life expectancy of the particular rehabilitation or strengthening method and materials and their ability to achieve the life of the structure as required by the relevant authority.
- (b) Access.
- (c) Aesthetics.
- (d) Functionality.
- (e) Constructability.
- (f) Structural considerations.
- (g) The difference between 'as built' and 'as designed' information, including modifications made after initial construction.
- (h) Drainage.
- (i) Scour protection.
- (j) Waterway improvements.
- (k) Life cycle costs, including the proposed rehabilitation or strengthening.
- (l) The geometry and condition of bridge approaches.
NOTE: The bridge approaches include the adjoining roadway/railway.
- (m) Heritage and historical aspects.
- (n) Liaison with other agencies and stakeholders, including utility authorities, conservation authorities, councils and private property owners.
- (o) The presence of utilities.
- (p) Provision for further rehabilitation at a later date.
- (q) Traffic conditions, including road, rail, pedestrian, cyclist or waterway traffic.
- (r) Waterproofing.
- (s) Vibration.

2.2 ASSESSMENT OF EXISTING STRUCTURAL CAPACITY

The assessment of existing structural capacity shall be in accordance with AS 5100.7.

2.3 DESIGN

2.3.1 General

The existing bridge will have been designed to the Code or Standard current at the time.

The design for refurbishment, rehabilitation or strengthening for the bridge shall be undertaken to the loads and provisions of the AS(AS/NZS) 5100 series.

NOTE: For timber bridges, including loading, reference should be made to Appendix D.

The relevant authority may direct that a lesser design live loading be adopted where all of the following conditions apply:

- (a) The bridge has been originally designed to a road traffic or rail traffic loading less than that specified in AS 5100.2.
- (b) The current loading provisions of AS 5100.2 are not functionally required after allowing for future vehicle mass increases in the short to medium term.
- (c) It is not reasonably practicable or economically viable, based on the results of a life cycle cost analysis, to strengthen the bridge to current AS 5100.2 road traffic or rail traffic loading.

All other design loads, including earthquake loads and longitudinal traffic loads, shall comply with AS 5100.2 unless otherwise determined by the relevant authority.

Particular attention shall be given to changes in the operation of the bridge that may have occurred since its original design and construction.

Design for rehabilitation or refurbishment of the structure shall not reduce the stability or load capacity of any element of the structure.

2.3.2 Bridge specific assessment live loading (BSALL)

When a bridge is found to be inadequate with respect to the design live loading, an alternative BSALL may be derived individually for that bridge, with the agreement of the relevant authority.

NOTES:

- 1 Where bridge specific assessment live loading (BSALL) is proposed to be adopted, reference should be made to Appendix E.
- 2 For short span bridges (loaded length less than 50 m), it is generally not considered cost effective to develop a BSALL; however, there may be particular situations, for example, when heavy vehicles cannot reach the structure because of width restrictions or tight bends, where it may be appropriate to accept a lower assessment loading for short span bridges on minor roads, when agreed with the relevant authority. As extensive traffic surveys and computer analyses are necessary for such load derivation, the relevant authority may agree to such a procedure only when there is a reasonable possibility of achieving the load capacity. Such cases may arise where the design live loading assessment fails by a small margin and the bridge is located on a road with a small percentage of heavy vehicles.

SECTION 3 CONCRETE STRUCTURES

3.1 GENERAL

This Section sets out procedures for the condition assessment of existing concrete structures and their repair, rehabilitation and/or strengthening.

3.2 CONDITION ASSESSMENT

3.2.1 General

Assessment of the condition of a structure may be initiated by a schedule assessment as required by the applicable asset management system, by visual evidence of distress or deterioration, or by a requirement for the structure to carry increased live loads or other loads.

The assessment of a structure, including its material condition, its structural integrity and whether and how to repair it, shall be performed by a professional engineer.

An initial inspection shall be conducted to confirm the existence of deterioration and defects in concrete, identify any immediate safety problems due to the deterioration, and determine the need for a more detailed site inspection that may be required. If a detailed inspection is required to be undertaken, it shall incorporate a diagnostic survey of both chemical and physical tests to establish the location, cause, extent and degree of deterioration already apparent.

The available information shall be confirmed and supplemented by a program of inspection, diagnostic site survey and testing. The elements of the program shall be selected from those listed in Clauses 3.2.3 to 3.2.15 with due consideration of the type of structure, its local environment, form of distress and/or deterioration and likely repair or rehabilitation approach.

The assessment of the structure shall include—

- (a) the nature, extent, location, severity and causes of damage and/or deterioration, and the ability of the structure to perform its function;
- (b) the likely remaining service life of the structure; and
- (c) work required to make the structure safe before and throughout any rehabilitation or strengthening, and for the required service life.

When a rehabilitation or strengthening option is proposed for a structure, investigations shall consider as appropriate, but may not be limited to, those items stated in Clause 3.2, in order to—

- (i) confirm suitability the rehabilitation or strengthening;
- (ii) determine the extent of the rehabilitation and/or strengthening; and
- (iii) provide design input.

NOTES:

- 1 In addition to these investigations, short or medium term small-scale trials may be performed to assess the effectiveness of proposed repair or protective methods or to determine the likely current demands for cathodic protection system design purposes.

- 2 The type of diagnostic techniques used to assess the condition of a structure may vary, depending on the service environment a structure is exposed to and the likely rehabilitation or strengthening method. For example in the case of a structure exposed to a marine or saline environment where cathodic protection may be a potential rehabilitation measure, the condition assessment may include visual inspection, chloride and carbonation testing, concrete cover, alkali aggregate reactivity, reinforcement continuity, half-cell potential mapping and concrete electrical resistivity. On the other hand, if a structure is exposed to a non-marine environment, chloride testing may not be required unless available records support the undertaking of such testing.

3.2.2 Existing information

Available drawings, specifications, records, including photographic records, and notes shall be reviewed to assess the location, quantity, type (e.g. uncoated or coated, prestressed) and continuity of the reinforcement, and the constituents and quality of the concrete.

This information shall also be used to assist in understanding the design assumptions, intended behaviour of the structure, areas of critical load effects and the load events that create these critical load effects.

3.2.3 Visual inspection

Where the structure is accessible, visual survey data shall be collected to assess the type, causes and extent of defects, and the data shall include any features of the structure and its surrounding environment, which could influence the effectiveness of rehabilitation or strengthening. For areas that have been previously repaired, the repair methods and the materials used shall be investigated.

The cause of any deterioration shall be determined. Where any signs of structural distress are evident, an assessment of both the loadbearing capacity of the structure, and the need for temporary or permanent strengthening, shall be carried out by suitably qualified personnel.

All areas of the structure that may require repair shall be checked for delamination of the concrete cover as stated in Clause 3.2.9. Defects such as cracks, honeycombing or poor construction joints which could permit significant water penetration, and which could in turn impair the effectiveness of the repair or protection, shall be recorded.

Where prestressed or post-tensioned concrete components are the subject of the visual inspection, such inspection shall be performed by competent personnel.

3.2.4 Chloride analysis

The chloride content of the concrete sample shall be determined in accordance with AS 1012.20.2.

NOTE: When selecting the core or drill diameter, the maximum aggregate size should be taken into account. Sampling rates and sampling locations should be varied depending on the manner in which the chloride was introduced into the concrete, internally in the mix or externally during service. Chloride and cement content may be determined by the methods described in relevant Standards; however, it should be noted that the accuracy of results obtained may only be of the order of $\pm 20\%$ or even more.

3.2.5 Carbonation

Distribution of carbonation depths shall be measured on freshly fractured concrete surfaces by spraying with aqueous phenolphthalein solution or other method in accordance with a procedure approved by the relevant authority.

NOTES:

- 1 Sampling points should be included at areas with minimum concrete cover to reinforcing steel as determined by cover meter testing and at areas where the concrete appears porous or honeycombed.
- 2 Available test methods include, but are not limited to, Main Roads Western Australia Test Method WA 620.1—2012.

3.2.6 Potential mapping

Representative areas of both damaged and apparently undamaged concrete shall be surveyed for reinforcement corrosion, using portable reference electrodes complying with the requirements of AS 2832.5. Readings shall be taken on an orthogonal grid at a maximum spacing of 300 mm.

Continuity of the reinforcement within any potential mapping survey area is essential. Before starting the potential mapping, electrical continuity shall be checked in accordance with AS 2832.5.

Readings taken in any areas that have been identified as delaminated shall be treated with caution, because delaminations can produce readings that are inconsistent with the level of corrosion of the reinforcement.

3.2.7 Concrete electrical resistivity

Electrical resistivity surveys shall be carried out on representative areas of concrete to provide information on the ability of the concrete to act as an electrolyte and carry electrical currents necessary to induce corrosion and to also provide information for design purposes where a cathodic protection system may need to be considered.

NOTES:

- 1 Core samples may also be obtained from the structure to evaluate volumetric concrete resistivity.
- 2 The resistivity of concrete is related to the moisture content, variability in chloride concentrations, other contaminants or electrolytes and the quality of the concrete itself. Measurements are very sensitive to seasonal variations, prevailing weather conditions and the time of the year.
- 3 For determining concrete electrical resistivity, refer to ASTM G57.

3.2.8 Concrete cover

Concrete cover and reinforcement size and position measurements shall be carried out in order to indicate whether insufficient cover contributed to corrosion to enable an estimation of time for carbonation and chloride penetration to the level of steel reinforcement. Such measurements shall also be carried out in order to assess comparative current flow through areas of high and low cover, and to identify dense regions of reinforcement that may require high current density, where the application of cathodic protection may be considered as a preferred rehabilitation option.

3.2.9 Delamination

A delamination (drummy) survey shall be undertaken by tapping the concrete surface with the rounded face of a ballpeen hammer (to distinguish between solid and hollow sound) for detection of incipient spalls and delaminations over steel reinforcement.

NOTE: Delaminations should be detected promptly because they indicate that serious rusting has taken place and represent areas of unsound concrete, which may lead to more severe deterioration of the structure.

3.2.10 Crack widths

Cracks in concrete shall be assessed to evaluate their influence on the loadbearing capacity, serviceability and durability of the structure. The assessment shall establish the cause(s) of the cracks, crack width, the moisture condition of the crack and whether a crack is active or inactive.

3.2.11 Concrete core samples

Concrete cores ranging from 75 mm to 100 mm in diameter shall be cut from the in situ concrete structure and used in standardized laboratory tests to assess the quality and to quantify the main physical and chemical characteristics of concrete. These tests shall include, as appropriate—

- (a) visual assessment of integrity;
- (b) compressive strength to AS 1012.14;
- (c) volume of permeable voids (VPV) to AS 1012.21, sorptivity testing or other type of standardized permeability test;
- (d) depth of carbonation (see Clause 3.2.5);
- (e) chloride and sulfate ion content to AS 1012.20.1 and AS 1012.20.2 (see Clause 3.2.4);
- (f) cement content and type;
- (g) water/cementitious material (W/C) ratio;
- (h) concrete density;
- (i) petrographic examination of hardened concrete; and
- (j) aggregate type and content.

Where relevant to the design, the in situ tensile strength shall also be determined to ensure that the concrete has the capacity to allow strengthening to be undertaken.

All cored holes shall be repaired in accordance with Clause 3.5.

NOTES:

- 1 A concrete cover meter should be used to identify accurately the location of steel reinforcement within the concrete such that no steel reinforcement is included within the concrete core sample and no structural damage is done to the structure.
- 2 The tensile strength of in situ concrete may be determined in accordance with AS 1012.24.

3.2.12 Corrosion rate measurement

Corrosion rate measurements may be undertaken using the linear polarization resistance technique to determine directly the corrosion current density and the corrosion rate of the steel reinforcement in a concrete structure.

3.2.13 Alkali aggregate reaction (AAR) assessment

If the concrete of the structure under assessment contains aggregates that may be sensitive to alkalis or has experienced damage potentially due to alkali aggregate reaction, the risk of alkali aggregate reaction shall be investigated.

Concrete core samples (minimum dimensions of 75 mm diameter by 200 mm long) shall be extracted from the in situ concrete components for subsequent AAR examination by a petrographic and SEM/EDX analysis, as follows:

- (a) For petrographic examination, the concrete near the core ends shall be prepared and assessed under a microscope to identify the types of minerals present and any suspect mineral phases in accordance with ASTM C295.
- (b) For SEM/EDX analysis, the concrete of the core ends shall be analysed under a scanning electronic microscope (SEM) equipped with energy dispersive x-ray (EDX) detector to examine the morphology of the concrete micro-structure and chemical composition of the concrete.

NOTES:

- 1 Guidelines on minimizing the risk of damage to concrete structures due to alkali aggregate reactions are given in HB 79.
- 2 The AAR assessment should determine the existing and predicted level of AAR expansion to enable determination of the timing and type of remedial options. Where cathodic protection is to be applied, it should be demonstrated to have no influence on AAR.

3.2.14 Non-destructive testing**3.2.14.1 General**

Non-destructive testing shall be carried out by competent and experienced personnel. Non-destructive testing shall be in accordance with Clauses 3.2.14.2 to 3.2.14.8.

3.2.14.2 Cover meter

The cover meter may be used to determine the location and orientation of reinforcing steel, measure the depth of concrete cover and estimate the size of the reinforcing bar. The cover meter shall be supported with a current calibration certificate and shall be capable of detecting the presence of reinforcement and indicating the depth from the concrete surface to the nearest point of the reinforcement with an accuracy of ± 1.0 mm at a depth of 25 mm.

3.2.14.3 Rebound hammer

The rebound (Schmidt) hammer may be used to locate areas of poor quality concrete and variability in the concrete strength throughout the structure, with an average of a minimum of 10 impacts at a location as specified in ASTM C805, to give an indication of the relative compressive strength of the concrete at that test location.

3.2.14.4 Ground-penetrating radar (GPR)

GPR may be used to provide information on reinforcement location, size and arrangement, other buried steelwork, concrete cover and member thickness, prestressing strands, cable ducts, zones of varying moisture content, and to provide an assessment of delaminations and areas of honeycombing, cracking and voids within the concrete.

3.2.14.5 Ultrasonic pulse velocity (UPV)

The UPV technique may be used to provide a relative assessment of the in situ concrete compressive strength, to provide a comparative assessment of quality and assist the detection of voids, delaminations, and under-compacted and honeycombed areas in concrete. UPV shall be undertaken in accordance with test method ASTM C597. Concrete core calibration may be used to improve effectiveness of in situ strength assessment.

3.2.14.6 Impact echo

The impact echo technique may be used to determine the thickness of members (particularly when only one face is accessible), and the presence of delaminations, large cracking, voids and honeycombing in concrete. Impact echo shall be undertaken in accordance with ASTM C1383.

3.2.14.7 Infrared thermography

The infrared thermography technique may be used to provide an assessment of flaws in concrete (particularly delaminations in bridge decks) and to provide an indication of the location of reinforcing bars, cables, cable ducts and interior cracking. Infrared thermography shall be undertaken in accordance with ASTM D4788.

3.2.14.8 Radiographic methods

The radiographic technique may be used to detect interior flaws such as voids in post tensioning ducts and locating cables and cable ducts, reinforcing bars, cover thickness and density of concrete.

3.2.15 Other methods

Other appropriate methods may be used as part of the diagnostic survey and testing regime, such as—

- (a) a precise survey of the deflected shape of the structure;
- (b) load testing of part or all of the structure;
- (c) measurement of stress change using strain gauges;
- (d) measurement of the dynamic response of the structure;
- (e) breakout areas to correlate against non-destructive test results; and
- (f) trials of repair or protective measures.

3.3 ASSESSMENT FOR CATHODIC PROTECTION (CP)

In addition to the requirements of Clause 3.2 and where required by the relevant authority, the structure shall be assessed to determine the suitability and extent of a CP system. The cathodic protection of reinforced concrete structures shall be undertaken in accordance with Appendix B.

The diagnostic assessment shall take account of the components of the structure and of the existing and anticipated cathodic protection exposure zones, and shall include the following:

- (a) Visual inspection of all defects (such as cracks, honeycombing or poor construction joints) including a comprehensive photographic record.
- (b) Delamination survey of the concrete cover.
- (c) Chloride determination at incremental depths.
- (d) Carbonation depth measurement.
- (e) Concrete cover measurement.
- (f) Alkali aggregate reaction (AAR) assessment, including any long-term residual effects of AAR.
- (g) Steel reinforcement survey for—
 - (i) continuity between elements of the structure within each zone of the CP system;
 - (ii) continuity of reinforcement within elements of the structure; and
 - (iii) continuity of metallic items, other than reinforcement, to the reinforcement itself; and
 - (iv) presence of prestress.
- (h) Potential mapping.
- (i) Determination of concrete electrical resistivity.
- (j) Corrosion rate measurement.
- (k) Determination of compressive strength of concrete.
- (l) Determination of volume of permeable voids (VPV) or sorptivity of concrete.

The location and frequency of sampling and testing shall be as required.

3.4 ASSESSMENT OF FIRE-AFFECTED CONCRETE

3.4.1 General

Fire-affected concrete shall be assessed by initial inspection and, where required, supplemented with additional testing to establish the full extent of damage and the residual quality of the in situ concrete.

NOTE: Some materials such as epoxy formulations may not perform well under fire conditions. These materials may have been used to patch the structure, repair cracks, bedding material and in joints between precast elements.

The initial inspection shall estimate the range of fire temperature and the temperature effects of fire damage on concrete by assessing the damage on surrounding material and by comparison with non-fire-affected concrete, in order to determine the extent of fire damage.

NOTES:

- 1 The initial inspection is generally based on visual observations, although hammer tapping and rebound hammer may be utilized. This may be further supplemented with extraction of concrete cores for subsequent testing for compressive strength, petrographic examination, use of computer software to estimate fire temperatures more accurately and undertaking of structural analysis.
- 2 A fire may cause the structure to expand due to uniform heat gain and deflect due to a heat gain gradient through the structural section. These strains may cause distress in the structure at locations distant from the direct effects of the fire.

3.4.2 Temperature effects and changes in concrete damaged by fire

When assessing fire-affected concrete, consideration shall be given to changes including colour changes of concrete associated with variations in concrete temperature as follows:

- (a) Up to 120°C—No significant change in the properties or colour of concrete.
NOTE: With respect to over-drying of concrete up to 120°C, temperature has negligible effect and causes no damage to the pore system or microstructure of concrete. There is also no change in colour and drying results in only the loss of free moisture within the concrete microstructure.
- (b) From 120°C–250°C—Characterized by localized cracking and dehydration of the cementitious paste with complete loss of free moisture and a reduction in paste volume.
- (c) 250°C–600°C—Significant cracking of both the cementitious paste and aggregates due to expansion. At approximately 300°C the colour of concrete changes to pink. At 400°C decomposition of calcium hydroxide occurs.
- (d) Above 600°C—Complete dehydration of the cementitious paste with considerable shrinkage cracking, honeycombing and generally concrete becoming friable, very porous, can easily be broken down, and colour changes to grey.
- (e) Above 900°C—The colour of concrete changes to buff.
- (f) Above 1200°C—The various components of concrete start to melt.
- (g) Above 1400°C—Concrete melts completely.

3.4.3 Visual inspection

Visual inspection of fire-affected concrete shall determine the following:

- (a) Any observed colour changes (see Clause 3.4.2) and the temperature levels to which the concrete may have been subjected.
- (b) The presence of soot blackening and smoke deposits on concrete components, which may be a direct by-product of an intense fire.

NOTE: Light grey areas are usually associated with exposure to high temperatures.

- (c) The location, extent and severity of fire damage, and if the fire-affected portion of concrete has suffered explosive spalling resulting in total delamination of the cover concrete, blown off in layers of shallow depth or has resulted in an exposed coarse aggregate surface shall be established.
- (d) The likely intensity and proximity of the fire to the damaged concrete and, if possible, the actual source or type of combustible or flammable material that may have fuelled the fire.
- (e) The effects if any of subsequent extinguishing of the fire, including possible thermal impact and pressure caused by water contacting the hot microstructure of the concrete.
- (f) The location and extent of exposure of steel reinforcement (including ligatures) and prestressing tendons.
- (g) The physical condition of the steel reinforcement, including any evidence of buckled or deflected steel reinforcement or unravelled prestressing tendons.
- (h) If prestressing tendons are irreversibly damaged or whether any disruption has taken place, which may adversely affect the bond at the interface between the exposed prestressing tendons and the fire-affected concrete.
- (i) Any evidence of cracking or any other defects, fire-related or otherwise.

3.4.4 Concrete coring

Where required, concrete cores shall be extracted from both fire-affected areas and from sound concrete further away from the damage to—

- (a) enable compressive strength testing and relative comparison between fire-affected and non-affected areas;
- (b) allow petrographic examination of the fire-affected concrete;
- (c) establish visually the depth of fire-affected concrete with respect to both steel reinforcement and prestressing tendons; and
- (d) enable a visual inspection of any internal surfaces of voided superstructure components using suitable lighting through the cored holes.

All cored holes shall be repaired in accordance with Clause 3.5.

3.4.5 Strength assessment

Where required, strength assessment shall be based on concrete core compressive strength testing (including conversion to equivalent cylinder strength based on corrected core strength). Rebound hammer tests shall be undertaken in accordance with Clause 3.2.14.3.

NOTE: Physical appraisal using a hand-held hammer during the inspection of the fire-affected concrete structure may also be applicable.

The estimated compressive strengths at the various locations shall be utilized for structural analysis purposes.

3.4.6 Petrographic examination

Where required, petrographic examination shall be used to delineate the extent of the damaged area and identify any significant micro-cracking developed in the cementitious paste, which may also extend to the coarse aggregate pieces in the fire-affected zone and beyond into adjoining sound concrete (and any material that may exist within these cracks).

NOTE: Scanning Electron Microscope (SEM) may also be used to identify both the material that may fill these cracks and the cause of these cracks.

3.4.7 Structural analysis

Structural analysis shall be undertaken as appropriate to establish the residual capacity in satisfying the prevailing loading conditions in terms of the concrete strength and condition of the steel reinforcement in the fire-affected zone.

3.4.8 Hammer testing

Hand-held hammer testing, on both fire-affected concrete and sound concrete, shall be undertaken to obtain an indication of the relative quality compared to undamaged concrete.

3.5 METHODS OF REPAIR AND REHABILITATION OF CONCRETE STRUCTURES

3.5.1 General

The extent (if any) and method of repair and rehabilitation to the structure shall be determined based on—

- (a) the functional requirements for the structure;
- (b) the required remaining life of the structure;
- (c) the outcomes from the structural assessment as described in Clauses 3.3 and 3.4;
- (d) safety, risk and cost benefit assessments; and
- (e) the asset management strategy for the structure.

Consideration shall be given to the possible detrimental impacts of the proposed repair or rehabilitation such as the creating of corrosion cells adjacent to newly repaired concrete, loss of section strength from concrete breakout or the impact on stakeholders due to bridge closures to undertake the repair work.

NOTE: Guidance for concrete repairs is given in HB 84.

3.5.2 Repair of cracks

3.5.2.1 General

Cracked concrete shall be assessed and the influence of cracks on the loadbearing capacity, serviceability and durability of a concrete structure shall be evaluated prior to undertaking crack repairs.

A crack repair method shall be selected, prior to any repair works taking place, based on an assessment of the cause(s) of the crack, crack width and length, the moisture condition of the crack and whether a crack is active or inactive. The crack repair methodology shall include details of the crack filling material properties, surface preparation methods and method of application, crack location, and local environmental conditions.

3.5.2.2 Crack repair methods

3.5.2.2.1 General

The repair of concrete cracks shall be undertaken by one of the following methods, as required, unless otherwise approved by the relevant authority:

- (a) *Pressure injection* Crack filling under pressure using a selected polymer resin (such as epoxy or polyurethane) or cementitious grout to restore structural soundness and impenetrability of concrete. The procedure shall be either—
 - (i) low pressure injection, using proprietary injection kits, based on firm and sustained hand pressure until no further resin can be accepted in the crack; or
 - (ii) multi-component injection, based on proof of ratio and pressure tests relating to the discharge pressure and ratio of adhesive components.

- (b) *Routing and sealing* Enlarging the crack along its exposed face and filling with a suitable joint sealant.
- (c) *Gravity feed* Filling and sealing of horizontally positioned cracks using low viscosity resins by pouring and spreading onto surface or placing into purposely formed reservoirs.
- (d) *Coating over cracks* Application of coatings with a crack-bridging capability or impregnation ability (such as silanes) for cracks of width of 0.3 mm or less. The method shall be used only for cracks that are compatible with the functional requirements of the structure but are not associated with earth-retaining or water-retaining concrete components.
- (e) *Flexible sealing* Use of suitable flexible sealants to seal cracks in the same manner as flexible joint.
- (f) *Crack stitching* Anchoring of U-shaped metal legs in predrilled holes on both sides of cracks to restore the tensile strength of the crack affected concrete.

3.5.2.2.2 *Materials properties*

The properties of the crack-filling material and the characteristics of the equipment to be used shall be considered when assessing crack repair methods. The crack-filling material shall possess the following properties:

- (a) Resin injection:
 - (i) Viscosity at 20°C—Maximum 300 centipoise (cP).
NOTE: 1 cP = 1 mPa.s.
 - (ii) Compressive strength in accordance with BS 6319-2, minimum at 7 days—60 MPa.
 - (iii) Tensile strength in accordance with BS 6319-7, minimum at 7 days—25 MPa.
 - (iv) Flexural strength in accordance with BS 6319-3, minimum at 7 days—50 MPa.
 - (v) Adhesive strength at the crack edges minimum at 7 days—2.5 MPa or failure in concrete, whichever occurs first.
- (b) Routing or sealing, nipple placement, gravity feed or stitching:
 - (i) Compressive strength in accordance with BS 6319-2, minimum at 7 days—60 MPa.
 - (ii) Tensile strength in accordance with BS 6319-7, minimum at 7 days—25 MPa.
 - (iii) Flexural strength in accordance with BS 6319-3, minimum at 7 days—30 MPa.
- (c) Flexible sealing for live or active cracks. The flexible sealant shall be capable of providing for a minimum expected movement of $\pm 25\%$. Materials shall not be used when the ambient temperature is below 5°C (or is 5°C on a falling thermometer) or is above 35°C.

The crack-filling material shall be compatible with all materials with which it is intended to come into contact.

3.5.2.2.3 *Surface preparation*

Cracks and the concrete substrate located a minimum of 50 mm either side of cracks on all areas of application shall be clean and free from loose materials.

All traces of dirt, laitance, paint, oil, grease and efflorescence shall be removed. Impurities that inhibit wetting and adhesion or penetration by the crack-filling material shall be removed.

The surface and cracks shall be allowed to dry thoroughly or shall be dried out by accelerated means prior to the application of materials.

NOTE: The above requirements do not apply to repair materials that can tolerate dampness.

3.5.2.2.4 *Verification of depth of penetration of crack-filling material*

For injected cracks, 75 mm diameter concrete cores shall be extracted from the crack repair works to verify the depth of penetration of the crack filling material. The core samples shall be extracted at predetermined locations to verify that the crack-filling material has penetrated to the full depth of the crack.

A cover meter shall be used to locate the existing steel reinforcement. The cored holes shall be cleaned and repaired with a suitable shrinkage compensating cementitious repair material applied in accordance with the manufacturer's recommended method of use. The exposed surface of the repaired hole shall be similar in texture and colour to the surrounding concrete.

NOTE: For larger areas, non-destructive methods such as ultrasonic pulse velocity or impact echo or spectral analysis of surface waves (SASW) may also be used to verify penetration of material.

3.5.3 **Repair of concrete**

3.5.3.1 *General*

All operations comprising repair shall be performed in accordance with the requirements of the relevant authority, as appropriate.

Concrete repairs may take many forms from large element replacement to relatively small patch repairs. Large repairs or element replacement repairs shall comprise standard concrete construction methods with due allowance for restraint adjacent to the construction joint, difference in material properties between the new and existing concrete and the visual impact of the repair.

NOTE: Guidance on concrete repair and protection is given in HB 84 and EN 1504 series of Standards.

3.5.3.2 *Types and selection of patch repair methods*

The types of patch repair of concrete structures using cementitious repair materials shall include—

- (a) corrosion-deteriorated concrete repair;
- (b) non-corrosion-deteriorated concrete repair; and
- (c) filling of blowholes and surface imperfections.

The procedure for the repair of concrete shall include, where necessary—

- (i) breaking back to sound and dense concrete to receive repair material;
- (ii) preparation of exposed steel reinforcement and concrete substrate;
- (iii) application of an appropriate steel primer and substrate bonding coat; and
- (iv) rebuilding to the original surface profile and finish.

A cementitious patch repair method shall be selected based on—

- (A) an assessment of the cause(s) and extent of the spalled, deteriorated, damaged or defective concrete;
- (B) the location of the patch repair on the concrete structure or member; and
- (C) the proposed repair material properties, likely patch behaviour and the effect on load capacity and structural safety, serviceability and durability.

A cementitious patch repair of concrete shall be undertaken in accordance with a patch repair method that includes procedures for surface preparation, method of application, curing and surface finish, in order to ensure the longevity of the repair solution.

NOTE: The application of decorative, anti-carbonation or anti-graffiti coatings as part of a concrete repair may be considered.

3.5.3.3 *Material properties*

3.5.3.3.1 *General*

Materials used for reinstatement of concrete shall be single- or multi- component polymer-modified cementitious low shrinkage repair mortars, or shall be part of a complete polymer-modified cementitious repair system. Whole bags of material shall be used. Test certificates, material data sheets and health and safety data sheets shall be available for all materials.

3.5.3.3.2 *Repair material*

The proposed repair material shall—

- (a) be capable of being hand applied in vertical and overhead sections with no slumping;
- (b) achieve strength conforming to the requirements in Table 3.5.3.3.2;
- (c) achieve a drying shrinkage compatible with the parent material tested in accordance with AS 2350.13;
- (d) possess an acid soluble chloride-ion content expressed as the percentage of the total mass of cementitious material of not greater than 0.05%;
- (e) be resistant to alkaline solutions; and
- (f) be compatible with the properties of the parent material.

NOTES:

- 1 Repair mortars typically have a wet density in the order of 1600 kg/m³ to 2100 kg/m³ and a water/powder ratio of 0.13-0.17, where powder may include the binder, filler and aggregate. For vertical and overhead applications, a wet density in the order of 1700 kg/m³ is adopted.
- 2 The coefficient of thermal expansion of the repair material should be compatible with concrete substrate.
- 3 The repair mortar should have sufficient hydraulic abrasion resistance where subject to running water.
- 4 The strength properties of the repair material should generally not exceed the strength properties of the parent material by more than 20%, unless appropriate consideration is given to potential incompatibility of materials.

Steel reinforcement primer and concrete substrate-bonding coat shall be compatible with the repair mortar and be part of the same range of proprietary repair system.

TABLE 3.5.3.3.2
REPAIR MATERIAL MINIMUM STRENGTH REQUIREMENTS

In situ strength of structural element	Repair material minimum strength, MPa		
	At 1 day	At 7 days	At 28 days
Compressive strength			
15 MPa to 30 MPa	5	19	23
Greater than 30 MPa to 50 MPa	10	25	35
Greater than 50 MPa	15	40	60
Flexural strength			
15 MPa to 30 MPa	—	—	4
Greater than 30 MPa to 50 MPa	—	—	6
Greater than 50 MPa	—	—	10
Tensile strength			
15 MPa to 30 MPa	—	—	1.8
Greater than 30 MPa to 50 MPa	—	—	2.8
Greater than 50 MPa	—	—	3.8
Bond or pull-off strength to concrete substrate			
All concrete structure strengths	—	0.75	—

NOTE: The tensile strength of in situ concrete may be determined using test method AS 1012.24.

3.5.3.3.3 *Fairing coat*

Cementitious repair material required to fill blowholes and imperfections on concrete structures shall be—

- (a) a single component polymer-modified material;
- (b) capable of application at 0–3 mm thick and fill blowholes and imperfections flush with the finished concrete surface; and
- (c) capable of application over a large area without being subject to shrinkage cracking.

3.5.3.4 *Concrete removal*

3.5.3.4.1 *Corrosion deteriorated concrete repair*

All delaminated and honeycombed concrete shall be broken out to achieve a clean, physically sound concrete surface. The perimeter of each repair shall be cut to a nominal depth of 15 mm to avoid feathered edges around repair patches. Where honeycombing or otherwise physically unsound concrete extends deeper than the main reinforcement, breaking out shall be terminated not less than 20 mm behind the deepest main bar to ensure good mechanical bonding of reinforcement to the replacement concrete. Concrete shall be removed along the length of visibly corroding steel reinforcement until at least 50 mm of sound metal is exposed at each end of the rusted section.

NOTE: Removal of physically sound concrete from areas deeper than 20 mm behind the main reinforcement may be required to facilitate replacement of corroded reinforcing steel, to improve the mechanical coupling of the concrete to the reinforcing steel or other appropriate reasons.

3.5.3.4.2 *Non-corrosion deteriorated or defective concrete repair*

The depth of removal of concrete for non-corrosion deteriorated or defective concrete repair and the amount of exposure of steel reinforcement prior to commencement of application of repair material shall be as required by the relevant authority.

3.5.3.5 *Reinforcement preparation and substrate bonding coat*

All corrosion products shall be removed from the exposed steel reinforcement. Steel reinforcement shall be cleaned to a bright metal to achieve a surface preparation equivalent to AS 1627.4.

All exposed steel reinforcement shall be coated immediately following preparation and cleaning with a primer that forms part of the proprietary repair system to provide immediate protection against corrosion. The steel primer shall be thoroughly applied to the exposed steel reinforcement.

A substrate-bonding coat that also forms part of the proprietary repair system shall be worked into the concrete substrate to enhance the bond at the repair interface.

3.5.3.6 *Concrete reinstatement*

The concrete to be repaired shall—

- (a) have an exposed aggregate finish; and
- (b) be non-friable and dust free.

Abrasive blasting, high-pressure water jet blasting or similar methods shall be used to prepare the concrete for reinstatement.

Percussive methods shall be used where there are environmental concerns with other methods.

Concrete shall be reinstated using proprietary cementitious materials that comply with Table 3.5.3.3.2.

Prior to application of cementitious repair materials, the concrete substrate shall be prepared by thoroughly soaking with potable water. The surface of the concrete substrate shall be saturated surface dry immediately prior to reinstatement.

Where the existing concrete cover to steel reinforcement is less than the design requirements, the concrete repair shall be profiled to ensure that adequate cover of polymer-modified repair material is provided.

3.5.3.7 *Curing and protection*

The concrete repair material shall be cured and protected from drying out and against the harmful effects of water movement and weather, throughout the curing period.

Curing methods may include water curing, wet hessian, polyethylene sheeting that is adequately sealed, formwork left in place, curing compound or a combination of these.

NOTE: Typically curing and protection is required for a minimum of 7 days after placement.

Curing compounds shall comply with AS 3799 and shall be removed prior to the application of any protective or decorative coatings.

For concrete patch repairs greater than 500 mm × 500 mm in size and for all concrete repairs to chloride-affected concrete structures or members, where curing compounds are used, they shall be covered with heavy duty polyethylene sheeting fastened and sealed at the edges.

3.5.3.8 *Quality control testing*

3.5.3.8.1 *Compressive strength of cementitious repair material*

The mixed cementitious repair material shall be sampled at the required frequency and tested at 7 days and 28 days to confirm compliance with the minimum compressive strength requirements specified in Table 3.5.3.3.2. Test cubes shall be made and cured in accordance with AS 1012.8.3 and tested in accordance with AS 1012.9.

3.5.3.8.2 *Bond strength (pull-off) testing*

Partially cored direct pull-off tests of the fully cured in situ repair material shall be conducted at the required frequency to verify the tensile bond strength between the in situ repair material and the existing concrete substrate, 7 days after the completion of application.

The mean bond strength at 7 days shall be not less than 0.75 MPa, with no individual result less than 0.65 MPa. The mode of failure of the pull-off test shall be with tensile failure within the existing concrete substrate.

3.5.3.8.3 *Testing for drummy areas*

A visual inspection of all concrete repair areas shall be conducted immediately prior to the application of any decorative/anti-carbonation coating over the repair area for delaminations and any defects recorded.

A test for drummy areas shall be conducted using the rounded face of a ballpeen hammer (to distinguish between solid and hollow sound) for detection of delaminations.

3.5.4 *Protective coatings*

3.5.4.1 *General*

Protective coatings shall comply with specified requirements.

NOTE: Protective coatings may include film forming type (i.e. acrylic or water based acrylic, epoxy, polyurethane and polymer-modified cementitious) and pore-lining penetrants (i.e. silane, silane/siloxane and siloxane).

3.5.4.2 *Surface preparation*

The surface preparation shall be undertaken as required to achieve the appropriate adhesion, coating penetration into the concrete and aesthetic requirements. Concrete shall be clean and reasonably dry for the coating to adhere properly and be resistant to peeling or blistering.

Previously painted surfaces shall be clean and sound and all traces of peeling or loose paint shall be removed.

3.5.4.3 *Environmental conditions*

Protective coatings shall only be applied when the environmental conditions are within the following limits:

- (a) Air temperature between 10°C–35°C.
- (b) Relative humidity <85%.
- (c) Dew point at least 3°C above concrete surface temperature.

3.5.4.4 *Surface moisture condition*

Where materials to be used require a dry surface, the substrate areas to be treated shall be surface dry before application commences, unless coatings can be applied under wet conditions for special applications. Artificial drying of surfaces shall not be carried out and concrete surfaces may need to be protected to ensure that they are surface dry before applying a film coating or impregnation.

3.5.4.5 *Moisture content of concrete*

The moisture content of concrete shall be less than 10% to prevent water back-pressure on the coating.

NOTE: Moisture content may be measured by one of several proprietary calibrated moisture meters.

3.5.4.6 Application

Film coatings or impregnations shall be applied by brush, spray or roller as a series of continuous operations in accordance with the material manufacturer's specified techniques. The manufacturer's stated method of application, time limitations of materials and inter-coating time intervals shall be strictly adhered to. The coatings should be applied so as to produce a smooth, even coating free of lumps, ripples, sags, runs, air holes and other defects.

3.5.4.7 Timings for application

The timings for application of coatings on concrete surfaces shall be as follows:

- (a) *Newly constructed cast in situ concrete/passively cured precast concrete* Not earlier than 28 days after the concrete has been placed and provided the concrete is in a surface-dry condition.
NOTE: Passive curing includes use of water, wet hessian, polyethylene plastic or formwork in place or a combination of these.
- (b) *Steam or radiant heat cured concrete* Not earlier than 14 days after concrete has been placed.
- (c) *Repaired concrete* Not earlier than 14 days after completion of the repairs. Where normal concrete is used for the repair, not earlier than 28 days.

3.5.4.8 Quality control testing

The following quality control testing shall be undertaken:

- (a) Testing prior to application of materials:
 - (i) Testing the surface moisture condition of the concrete substrate to satisfy requirements as specified.
 - (ii) Testing the moisture content of the concrete to satisfy requirements as specified.
- (b) Testing after application of materials:
 - (i) Testing the bond strength to demonstrate conformance with the minimum adhesion strength as specified.
 - (ii) Measurement of the wet and dry film thickness or the amount of penetration of the coatings into the concrete to the minimum requirements.

The location and frequency of testing of the coating system or the pore-lining penetrant shall be considered on completion of the application of the final coat in accordance with requirements as specified.

3.5.5 Structures affected by cracking caused by alkali aggregate reactivity (AAR)

AAR affected structures shall be inspected on a regular basis by competent and experienced personnel. Monitoring of cracks shall be undertaken using suitable measuring devices. An assessment shall be made of the existing and predicted level of AAR expansion to enable determination of the timing and type of remedial options.

NOTE: Remedial measures to prevent moisture ingress may include—

- (a) the injection of cracks with flexible epoxy or elastomeric chemical grouts;
- (b) the application of flexible epoxy, elastomeric acrylic or cementitious coatings;
- (c) silane impregnations where appropriate;
- (d) concrete encasement or propping and strutting in severe cases where required; or
- (e) combination of the above.

3.5.6 Repair of fire-affected concrete

A detailed repair of fire-affected concrete shall be developed following investigation and assessment of visual and temperature effects of the fire damage on concrete, consideration of any effects on physical properties, temperature effects on reinforcing steel and prestressing strands and the temperature effects on the concrete/steel interface.

All fire-affected and weakened concrete shall be removed right behind the steel reinforcement and prestressing tendons and repaired with flowable or hand/trowel-applied polymer-modified cementitious materials or spray-applied gunite cementitious repair materials in accordance with the requirements of Clause 3.6.3.

Repair of fire-affected concrete shall include—

- (a) breaking back to sound and dense concrete and exposing as much of the steel reinforcement as possible;
- (b) preparation of steel reinforcement and concrete substrate;
- (c) application of an appropriate steel primer and substrate bonding coat; and
- (d) rebuilding to the original surface profile.

When removing fire-affected concrete behind normal steel reinforcement to provide a good mechanical key, consideration shall be given to not exposing prestressing tendons any further if they are not physically damaged or unravelled to ensure continuation of the residual prestressing effectiveness.

3.5.7 Installation of cathodic protection (CP) of reinforced concrete structures

Cathodic protection (CP) of steel in atmospherically exposed, buried and submerged concrete in existing structures shall be designed, installed and monitored in accordance with the requirements of AS 2832.5 and Appendix B. This shall apply to both impressed current and sacrificial cathodic protection systems for structures that include normal reinforcement and prestressed reinforcement embedded in the concrete.

To confirm suitability, the extent of the CP system shall be determined, design input for the system shall be provided, and a diagnostic investigation shall be undertaken as stated in Clause 3.3, which shall be representative of the various structural components of the structure and the various CP exposure zones.

The detailed design and design calculations, technical specification for installation, installation procedures, commissioning, review of compliance to protection criteria, ongoing monitoring and operation of the cathodic protection system shall be undertaken by experienced and competent personnel.

The CP system shall be designed and installed such that it delivers sufficient current to polarize satisfactorily the reinforcement of the whole of the reinforced concrete components being protected for all zones including within the submerged, tidal, splash and atmospheric zones. Satisfactory polarization shall be demonstrated by achieving the protection criteria in accordance with the requirements of AS 2832.5.

The CP system shall be designed and installed to operate at current densities limited to values that do not—

- (a) cause adverse effects to the steel/concrete bond in plain reinforced concrete structures;
- (b) cause excessive acidification within CP anode backfill;
- (c) cause hydrogen embrittlement in prestressing steel; and
- (d) increase the potential for alkali aggregate reactivity due to the increase in alkalinity at the steel.

NOTE: Guidance on the principles of cathodic protection and its application to steel in concrete is given in AS 2832.5.

3.5.8 Waterproofing of concrete bridge decks

3.5.8.1 General

Where required, waterproofing of concrete bridge decks shall be undertaken to prevent moisture ingress or contamination of the deck slab by other deleterious substances.

3.5.8.2 Membranes

Membranes may include synthetic preformed sheet and liquid applied membranes. Preformed sheet membranes shall cover the entire deck between parapets, with careful detailing to ensure continuity under kerbs, verges and central medians.

Sheet membranes shall be overlapped at the seams and sealed so that water or other substances cannot penetrate. The sealing process shall be carried out by heat or cold solvent welding consistent with type of membrane material used.

Spray-applied solvent-free waterproof membranes shall be single component materials capable of achieving effective bond onto the concrete surface with increased resistance to punctures. Surface preparation, environmental checks, surface moisture condition, application, and quality control testing shall be in accordance with the requirements of Clause 3.6.4.

When a wearing course is applied over waterproofing membranes (including sheet membranes), such membranes shall be capable of withstanding both the laying temperature of bituminous materials (155°C to 180°C) and puncturing from aggregates when the surfacing is rolled.

3.5.9 Sulfate attack repairs

An examination for possible sulfate attack shall be undertaken to determine the extent of any damage. The affected component shall be isolated (if required) by containing and propping, and the deteriorated concrete removed and replaced with fly ash, slag or silica fume concrete in accordance with the requirements of Section 4 of AS 5100.5 or polymer-modified cementitious materials for increased resistance to sulfate attack in accordance with Clause 3.6.3.

The use of polymer-modified cementitious materials shall be reviewed for compatibility for exposure to sulfates, as some of these materials may contain high levels of alkali or aluminate to provide the expansion as part of their required performance.

Surface-applied barrier protective coatings such as epoxies may be applied in accordance with Clause 3.5.4 to protect concrete surfaces or concrete repairs against sulfate attack. Other physical protective barriers and membranes may also be used.

3.6 STRENGTHENING OF CONCRETE STRUCTURES

3.6.1 General

Strengthening of concrete structures shall be undertaken by using various materials and techniques including fibre-reinforced polymer (FRP) composite strengthening, steel plating, external/transverse post tensioning, reinforced concrete deck overlay and reinforced concrete jacketing systems.

Strengthening of a structure shall be based on a structural analysis undertaken to determine the level of overloading or under-design for the in-service load conditions, to ensure that the strengthened structure meets the minimum requirements for serviceability, strength and durability.

The structural analysis shall consider any load redistribution in the strengthened structure for the in-service load conditions to prevent any overstressing of unstrengthened sections.

3.6.2 Fibre-reinforced polymer (FRP) composite strengthening

3.6.2.1 General

FRP composite strengthening shall be undertaken in accordance with the requirements of Appendix A and the following:

- (a) The minimum tensile strength of the concrete to be strengthened shall be 1.5 MPa and its minimum compressive strength not less than 17 MPa.
- (b) FRP composite strengthening systems that are based on a combination of fibre-reinforcing material and adhesive resins shall be designed and installed to provide an adhesive bond onto the concrete surface, which allows for the satisfactory transfer of stresses to and from the fibre materials.
- (c) FRP composite strengthening work shall be undertaken by competent and experienced personnel who are competent in the installation procedures developed by the manufacturer of the particular FRP composite strengthening system.
- (d) Surveillance and inspection during installation of the FRP composite strengthening systems and inspection and monitoring of strengthened structures shall be undertaken by competent and experienced personnel knowledgeable in both the material aspects and the installation of FRP composite strengthening works.

NOTE: Whilst long-term performance is still the subject of ongoing research, FRP systems have been successfully used in structural applications for bridges, since the 1980s. As a result, a design life of 30 years is considered a conservative estimate for FRP composite strengthening systems provided regular inspection, monitoring and maintenance regimes are put in place.

3.6.2.2 Validation of FRP composite systems

The immediate performance of the FRP composite strengthening system may be monitored by the installation of suitable instrumentation on the strengthened components at pre-determined strategic locations.

Instrumentation shall consist of strain gauges or fibre-optic gauges embedded within the FRP, bonded to the structure or FRP surface or on additional band of FRP for monitoring and testing purposes.

NOTE: The instrumentation may be installed following completion of all strengthening and any rehabilitation works on the strengthened components (i.e. including any repair of concrete, epoxy injection of cracks and application of anti-carbonation protective coating).

3.6.2.3 Special inspection and testing

When a routine visual inspection has identified a problem, detailed inspection and testing shall be undertaken by competent and experienced personnel with appropriate experience of fibre composites to adequately assess and interpret the findings.

Delamination/drummy testing shall be undertaken by competent and experienced personnel to identify voids and areas of debonding and establish the integrity of the FRP strengthening.

NOTE: Thermal and acoustic non-destructive methods with proven reliability may be used.

Installation of instrumentation such as strain gauges or fibre-optic gauges embedded within the FRP, bonded to the structure or FRP surface or on additional band of FRP for monitoring and testing purposes shall be considered for FRP composite strengthening works.

NOTE: Such devices can provide information as to the ongoing performance of the FRP system as affected by bending and structural in-service live loads or abnormal loadings.

3.6.3 Strengthening of concrete structures with steel plates

Strengthening of deteriorated or underperforming concrete structures or members with steel plates or steel brackets shall be designed to the required thickness and configuration. Flat steel plates shall be used in combination with epoxy adhesives and, where required, bolted onto the concrete substrate to the minimum requirements. Temporary support systems shall also be designed and installed where required.

NOTE: The steel plates or brackets may be coated for protection.

3.6.4 Other types of strengthening of concrete structures

Other forms of strengthening or increasing the structural loadbearing capacity of concrete structures or individual components for flexure, shear, torsion and axial load may also be designed and installed to minimum requirements, including—

- (a) external/transverse post tensioning (steel rods or tendons), provided end anchorage plates are adequately designed and seated securely;
- (b) reinforced concrete deck overlay, which provides strengthening for additional live loads and overall waterproofing of a structure;
- (c) additional reinforced concrete/sprayed concrete strengthening of beams, columns, crossheads and walls;
- (d) additional members such as beams, columns, cross-bracing and walls installed to support damaged structural members;
- (e) adding or replacing embedded steel reinforcement; and

NOTE: Beam enlargement may require steel reinforcement to be epoxied into the concrete or a steel cage placed around it, followed by formwork and then conventional concrete or suitable cementitious grout placed into the forms. In the case of columns, additional steel cage may be built around the column followed by formwork and then conventional concrete or suitable cementitious grout placed into the forms.

- (f) adding ancillary items such as buffer bearings or tuned mass dampers.

Filling or injecting cracks and voids shall be undertaken in conjunction with all types of strengthening where required.

The design of these strengthening systems shall be based on the requirements of the AS(AS/NZS) 5100 series.

Where deteriorated concrete requires removal and repair, this shall be done in accordance with the requirements of Clause 3.6.2 and Clause 3.6.3.

3.6.5 Post-installed fasteners

Where post-installed chemical and mechanical fasteners (anchors) are used for retrofit or maintenance and repair, they shall meet the following requirements:

- (a) The fastening design shall be in accordance with SA TS 101.
- (b) The fastener shall meet the prequalification requirements of SA TS 101:2015 Appendix B. A fastener with a current European Technical Approval/Assessment (ETA) awarded in accordance with ETAG 001 satisfies the requirements of SA TS 101 Appendix B. For applications involving specific conditions such as cracked concrete, repeated loading, sustained tensile loading or seismic loading, products satisfying these conditions through relevant SA TS 101 Appendix B or ETA Options shall be specified.
- (c) The fastener installer shall be certified by the Australian Engineered Fasteners and Anchors Council, or shall be able to provide evidence of competency acceptable to the relevant authority.

SECTION 4 STEEL STRUCTURES

4.1 GENERAL

This Section sets out procedures for the condition assessment of existing steel structures and their repair, rehabilitation and/or strengthening.

The assessment of a metal structure or component shall include the identification of the material as cast iron, wrought iron or steel. Strength parameters for cast iron and wrought iron may be highly variable and some steel types may be non-weldable. Testing shall be undertaken where required to determine appropriate strength parameters, chemical composition and, where applicable, suitability for welding.

NOTE: For material characteristics of typical old iron and steel, see AS 5100.7.

4.2 CONDITION ASSESSMENT

4.2.1 General

Assessment of the condition of a structure may be initiated by a scheduled assessment as required by the applicable asset management system, visual evidence of distress or deterioration, or by a requirement for the structure to carry increased live or other loads.

The assessment of a structure, including its material condition, its structural integrity and whether and how to repair it, shall be performed by competent personnel.

An initial inspection shall be conducted to confirm the existence of deterioration and defects in steel, identify any immediate safety problems due to the deterioration, and determine the need for a more detailed site inspection that may be required. If a detailed inspection is required to be undertaken, it shall establish the location, cause, extent and degree of deterioration already apparent.

The assessment of the structure shall include—

- (a) the nature, extent, location, severity and causes of damage and/or deterioration, and the ability of the structure to perform its function;
- (b) the likely remaining service life of the structure; and
- (c) work required to make the structure safe before and throughout any rehabilitation or strengthening, and for the required service life.

When a rehabilitation or strengthening option is proposed for a structure, investigations shall consider as appropriate, but may not be limited to, those items stated in this Clause, in order to—

- (i) confirm suitability of the rehabilitation or strengthening;
- (ii) determine the extent of the rehabilitation and/or strengthening; and
- (iii) provide design input.

4.2.2 Existing information

Available drawings, specifications, records, including photographic records, and notes shall be reviewed to assess the type and grade of the structural steel, and associated bolts, rivets and welds.

4.2.3 Causes of deterioration

4.2.3.1 General

The causes of the original deficiency shall be determined, to establish whether the repairs will be susceptible to the same problems.

4.2.3.2 Corrosion

Corrosion is the most prevalent factor affecting steel structures. Corrosion is the deterioration of steel by chemical or electro-chemical reaction resulting from exposure to air, moisture, industrial fumes and other chemicals and contaminants in the environment in which it is placed. Corrosion will only occur if the steel is not protected or if the protective coating wears or breaks off. Corrosion of carbon steel is initially fine grained, but as the process progresses it becomes flaky and delaminates, exposing a pitted surface. The process thus continues with progressive loss of section.

Corrosion shall be prevented by any of the following systems:

- (a) Durable protective barriers such as painting, encasing in concrete or galvanizing to prevent oxygen and moisture reaching the steel.
- (b) Inhibitive primers, which hold off attack on the steel substrate.
- (c) Provision of sacrificial anodes.
- (d) Provision of zinc-rich primers incorporated into a high quality protective coating system or galvanizing.
- (e) Provision of cathodic protection by use of an external current to suppress the anodic reaction.

4.2.3.3 Graphitization

Graphitization is a form of corrosion that may occur in submerged cast iron. Cast iron contains carbon in the form of graphite in its molecular structure. Graphitization occurs when the stable graphite crystals remain in place but the less stable iron is converted into somewhat soluble iron oxide and is leached out. As a result, the cast iron retains its shape and appearance but has no strength due to the loss of the iron. The corrosion process is galvanic. Section loss generally occurs at a rate of 5 to 16 mm per 100 years.

4.2.3.4 Permanent deformations

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

Permanent bending deformation usually occurs in the direction of the applied loads and is usually associated with flexural members.

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

Permanent twisting deformations appear as a rotation of the member about its longitudinal axis and are usually the result of eccentric transverse loads on the member.

Slippage at bolted connections can result in permanent deformation at the connection.

4.2.3.5 Cracking

Cracking comprises a linear fracture of the steel. Cracks in steel are mainly produced due to fatigue loading and can, under certain conditions, lead to brittle fracture. Brittle fracture is a crack completely through the component that usually occurs without prior warning or plastic deformation.

Any crack shall be carefully recorded as to its specific location in the member. The length, width (if possible) and orientation of crack shall also be recorded.

The primary factors leading to fatigue cracking are:

- (a) the number of applied stress cycles, which is a function of the nature of traffic or wind;
- (b) the magnitude of the stress range, which depends on the applied live load; and
- (c) the fatigue strength in the connection detail.

Cracks caused by fatigue usually occur at points of tensile stress concentrations, such as welded attachments or at termination points of welds.

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

Welded connections are more prone to cracking than bolted or riveted connections. Once the cracking occurs in a welded connection, it can extend into other components due to a continuous path provided at the welded connection, and possibly lead to a brittle fracture.

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

As cracks may be concealed by paint, rust, dirt or debris, the suspect surfaces shall be cleaned prior to inspection.

Cracks that are perpendicular or inclined to the direction of force in the member could lead to fracture of the member. Such cracks shall be addressed as a matter of urgency.

4.2.3.6 *Loose connections*

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

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

4.2.4 **Methods of assessment**

4.2.4.1 *General*

The assessment of a structure, including its material condition, its structural integrity and whether and how to repair it, shall be performed by competent and experienced personnel.

4.2.4.2 *Visual inspection*

Where a structure is accessible, visual survey data shall be collected to assess the type, causes and extent of defects. This survey shall include features of the structure and its surrounding which may influence the effectiveness of rehabilitation or strengthening. For areas that have been previously repaired, the repair methods and the materials used shall be investigated.

The cause of any deterioration shall be determined. Where any signs of structural distress are evident, an assessment of both the loadbearing capacity of the structure, and the need for temporary or permanent strengthening shall be carried out by a professional engineer.

For steel bridges, visual inspection shall include the following:

- (a) Observing members under load where possible and noting any excessive movement in members or fastenings.
- (b) Identification of any crack on the member surface (see Clause 4.2.3.4).
- (c) Examination for water build-up, especially in areas where build-up could cause corrosion.
- (d) Examination for notches and associated cracking caused by impact from vehicles or equipment.
- (e) Examination of surface treatment, evidence of surface corrosion and, where necessary, measurement of corrosion loss.

Visual inspection has been found to detect most defects in steel structures. Particular note shall be taken of the following:

- (i) Any distortion and misalignment of structures or individual members.
- (ii) Paint condition, including measurement of residual paint thickness.
- (iii) Water, dirt or other debris lying on steel surfaces that could lead to corrosion.

Measurements of corroded steel shall be taken to determine the thickness of remaining sound material for comparison with the original thickness. All loose rust and corrosion product shall be removed at the point of measurement to allow an accurate reading. Vernier callipers or a micrometer shall be used to obtain accurate measurements to at least an accuracy of ± 0.25 mm. The number and location of measurements required will depend on the defect under consideration. As a minimum, the remaining cross-sectional area of an element (flange, web, stiffener) at the location of the greatest corrosion loss and at the location of greatest stress shall be measured.

Visual inspection of steel components shall be in accordance with AS 3978.

4.2.4.3 Hammer test

Hammer testing (where members are tapped lightly with a hammer) may indicate loose plates and fastenings, extent of corrosion, and effectiveness of corrosion protection. Care shall be taken that hammering does not cause damage to the corrosion protection system.

4.2.4.4 Thickness testing of steel sections

The thickness of steel sections can often be measured directly across an edge or a penetration, using rulers, vernier callipers or micrometers. In cases where access to both surfaces is not available or possible, thickness of steel may be measured using an ultrasonic thickness meter.

Thickness testing of steel sections using an ultrasonic instrument shall be undertaken in accordance with AS 2452.3.

4.2.4.5 Coating condition testing

A dry film thickness gauge may be used to determine the thickness of a coating on steel. The coating may be recently applied or showing signs of deterioration as a result of exposure.

Film thickness testing of protective coating shall be undertaken in accordance to AS 3894.3.

Samples of paint shall be taken to determine the percentage of lead present in the full thickness of the coating.

4.2.4.6 Radiographic testing

Non-destructive radiographic testing shall be undertaken in accordance with AS 2177 or AS 3507.2, as appropriate to the item being tested.

4.2.4.7 *Ultrasonic testing*

Non-destructive ultrasonic testing shall be undertaken in accordance with AS 1065, AS 1710, AS 2207 or AS 2574, as appropriate to the item being tested.

4.2.4.8 *Magnetic particle testing*

Magnetic particle inspection (MPI) is a non-destructive testing (NDT) process for detecting surface and slightly subsurface discontinuities in ferromagnetic materials such as steel. The process puts a magnetic field into the member. Cracks are detectable due to the magnetic particles clustering at the crack openings. This is particularly important in mapping fatigue cracks where the end point must be found.

Non-destructive magnetic particle testing shall be undertaken in accordance with AS 1171.

4.2.4.9 *Acoustic emission*

The application of acoustic emission (AE) to non-destructive testing of materials, typically takes place between 100 kHz and 1 MHz. Unlike conventional ultrasonic testing, AE tools are designed for monitoring acoustic emissions produced within the material during partial failure or stress, rather than actively transmitting waves, then collecting them after they have travelled through the material. This process can be monitored locally.

The acoustic emission test shall be undertaken in accordance with ASTM E1139.

4.2.4.10 *Laboratory testing of steel samples*

The laboratory testing of steel samples comprises chemical analysis, metallographic examination and mechanical testing.

Chemical composition shall be determined in accordance with AS/NZS 1050 series of Standards or other procedures that achieve the same, or better, degree of accuracy.

Microstructure testing shall be undertaken in accordance with ASTM E407 and ASTM A247, and with AS 2205.5.1 for welds.

Tensile testing shall be undertaken in accordance with AS 1391. Impact testing shall be undertaken in accordance with AS 1544.2, at an appropriate temperature.

Hardness testing shall be undertaken in accordance with AS 1816.1 for the Brinell hardness test, or AS 1817.1 for the Vickers hardness test, as appropriate.

Rebound hardness (Leeb) testing shall be undertaken in accordance with ASTM A956.

4.2.4.11 *Dye penetrant testing*

Dye penetrant testing is used to locate surface cracking and defects in steel elements and welds. A penetrant solution is applied to a clean and dry steel surface and allowed time to move into surface cracks. Excess penetrant is then removed and a developer solution is applied. The developer draws out the penetrant to reveal cracks under either ultraviolet or white light depending on the dye being used. Dye penetrant does not identify deep defects and discontinuities.

The dye penetrant test shall be undertaken in accordance with AS 2062.

4.2.4.12 *Eddy current testing*

Eddy current testing uses electromagnetic fields to detect discontinuities near the surface of steel elements and welds.

The eddy current test shall be undertaken in accordance with AS 4544 and AS 2331.1.4.

4.2.4.13 *Adhesion testing*

Adhesion testing of protective coating shall be undertaken in accordance with AS 3894.9.