Design Manual for Roads and Bridges

Highway Structures & Bridges
Inspection & Assessment

CS 462
Repair and management of deteriorated concrete highway structures

(formerly BA 35/90, BA 52/94)

Revision 0

Summary
This document provides requirements and advice for repair or management in service of deteriorated concrete highway structures.

Application by Overseeing Organisations
Any specific requirements for Overseeing Organisations alternative or supplementary to those given in this document are given in National Application Annexes to this document.

Feedback and Enquiries
Users of this document are encouraged to raise any enquiries and/or provide feedback on the content and usage of this document to the dedicated Highways England team. The email address for all enquiries and feedback is: Standards_Enquiries@highwaysengland.co.uk

This is a controlled document.

Contents

Release notes

Version	Date	Details of amendments
0	Mar 2020	CS 462 replaces BA 35/90 and parts of BA 52/94. This full document has been re-written to make it compliant with the new Highways England drafting rules

Foreword

Publishing information

This document is published by Highways England.

The document supersedes BA 35/90 and parts of BA 52/94, which are withdrawn.

Contractual and legal considerations

This document forms part of the works specification. It does not purport to include all the necessary provisions of a contract. Users are responsible for applying all appropriate documents applicable to their contract.

Introduction

Background

The most prevalent cause of deterioration in the condition of concrete highway structures is reinforcement corrosion. This is usually caused by the presence of free chloride ions in the concrete, which break down the passive alkaline layer surrounding the steel reinforcement.

The chloride ions come mainly from road de-icing salt, although for some structures, chloride-based accelerators were used with the original concrete mix. Wind-borne chlorides and sea-water are additional or alternative sources at coastal sites.

A breakdown in the passive alkaline layer may also be caused by carbonation alone and sometimes by a combination of chloride contamination and carbonation.

Products of the steel corrosion reaction are typically expansive when occurring within parts of structures exposed to the atmosphere, and the resulting increase in internal stresses often causes delamination and spalling of the cover concrete.

When parts of a reinforced concrete structure are buried below ground, are submerged or are continuously wet (anaerobic or partially anaerobic conditions), soluble non-expansive corrosion products can be produced in the presence of chloride contamination. Localised corrosion can lead to significant reinforcement section loss without any evidence of concrete cracking or delamination.

Delamination and spalling of concrete can also be caused by freeze/thaw temperature cycles, chemical degradation of the concrete (e.g. sulfate attack or alkali aggregate reaction) and damage by a vehicle impact or by fire.

This document incorporates technical updates due to modern developments in construction techniques for repairing deteriorated concrete since publication of the previous advice note BA 35/90. The document also includes updated requirements following improvements in understanding of the mechanisms of internal deterioration of concrete (IDC), and more recent research into previously rare forms of concrete deterioration (thaumasite sulfate attack).

The document has been written for design of repair schemes complying with the relevant parts of BS EN 1504 [Ref 22.N] and the European Construction Products Regulation 2011/305/EU [Ref 29.I] and also covers management of deteriorated structures in service.

Assumptions made in the preparation of this document

The assumptions made in GG 101 [Ref 14.N] apply to this document.

Mutual Recognition

Where this is a requirement in this document for compliance with any part a "British Standard" or other technical specification, that requirement may be met by compliance with the Mutual Recognition clause in GG 101 [Ref 14.N].

Abbreviations

Abbreviations

Abbreviation	Term
AAR	Alkali aggregate reaction
AIP	Approval in Principle document
ASR	Alkali silica reaction
BBA	British Board of Agrément
CP	Cathodic protection
CSE	Saturated copper/copper sulfate electrode
DEF	Delayed ettringite formation
F_{DC}	Compressible strength degradation factor
F_{DE}	Elastic modulus degradation factor
F_{DT}	Tensile splitting strength degradation factor
GI	General Inspection
IAF	International Accreditation Forum
IDC	Internal deterioration of concrete
IStructE	Institution of Structural Engineers
MLA	Multilateral Agreement
OO	Overseeing Organisation
PI	Principal inspection
SI	Special inspection
SMOR	Structure management options report
SMP	Structure management plan
SRPC	Sulfate-resisting portland cement
SSC	Silver/silver chloride electrode
TAA	Technical Approval Authority
TSA	Thaumasite sulfate attack
UKAS	United Kingdom Accreditation Service
WCS_{dc}	Worst credible strength of deteriorated concrete in compression
WCS_{dt}	Worst credible strength of deteriorated concrete in tension

Symbols

Symbols

Symbol	Definition
kΩ.cm	Unit of measurement of bulk electrical resistivity of a material, that is the material's ability to resist passing current. This is the inverse of conductivity and is measured in Ohm cm, kOhm cm or Ohm m.
MPa	Megapascal. 1 MPa is equal to 1 N/mm^2
mV	Millivolt

Terms and definitions

Terms

Term	Definition
Alkali aggregate reaction	Damaging chemical reactions to aggregates within a mass of concrete which can include alkali silica reaction (ASR), alkali silicate reaction or alkali carbonate reaction. NOTE: ASR is the most common.
Cathodic protection	The process by which metal is forced to become the cathode of an electrochemical cell and is thus protected from corrosion.
Concrete highway structure	A structural element (or group of elements) that supports, protects or carries traffic (vehicular or non-motorised users) over or under the highway network, and is partly or wholly constructed from reinforced, pre-stressed or post-tensioned concrete.
Defective concrete	Delaminated concrete located within the cover zone of a highway structure (can also include an excessive concentration of free chloride ions within intact cover zone concrete).
Deteriorated concrete structure	A concrete highway structure exhibiting significant concrete delamination and spalling caused by reinforcement corrosion. NOTE: Impact damage can also have occurred.
Hammer sounding of concrete	A technique used by a surveyor to detect delamination of concrete covering corroded steel reinforcing bars within reinforced concrete. NOTE: Delamination is caused by expansion of corrosion products which exceed the tensile strength of the concrete.
Interim measures feasibility assessment	An assessment report prepared upon identification of sub-standard structures in accordance with CS 470 [Ref 16.N]. It outlines the level of risk within the structure and the appropriate interim measures required.
Internal deterioration of concrete	Deterioration mechanism (consisting of AAR or DEF) which affects the concrete composition and can affect concrete mechanical properties. NOTE: Concrete degradation factors F_{DC}, F_{DE}, F_{DT} define the reduction in mechanical properties which can be used for load re-assessment.
Non-structural repair	A shallow repair 10 mm or less in depth using concrete or mortar that does not surround the reinforcement, and is not required to contribute to the strength of concrete at a cross section.
Overseeing Organisation	See definition in GG 101 [Ref 14.N].
Structures asset management	A systematic approach to managing highway structures owned by a highway authority, comprising an asset management strategy, maintenance plan and a works programme.

Terms (continued)

Term	Definition
Structure management options report	A report outlining the feasible alternatives for managing a deteriorated structure which can include rectifying some of the defects, rectifying all the defects, monitoring the deterioration for a period of time, replacing the structure or structural elements, or removing the structure from service.
Structure management plan	A document that includes or references all available record information for an individual structure, describes routine maintenance activities and outlines the forward plan for periodic minor or major maintenance, in order to meet the levels of service agreed in the structures asset management strategy.
Structural element	A component part of a reinforced concrete substructure or superstructure. NOTE 1: Structural elements of a superstructure can include deck, parapet, string-course, beams, diaphragms, run-on slab, staircase or ramp. NOTE 2: Structural elements of a substructure can include abutment, wingwall, retaining wall, foundation, pilecap, pier, cross head, column, bearing plinth, curtain wall, panel wall, screen wall, pylon base and roof slab.
Structural repair	A repair that entirely surrounds the reinforcement, uses concrete with a characteristic strength >25MPa and is expected to contribute to the strength of concrete at a cross section.
Technical Approval Authority	The organisation responsible for agreeing technical aspects of managing deteriorated structures and/or repair proposals.
Value management	The process of identification and appraisal of a structure renewal schemes by which safety, impact on the network and whole-life cost factors are considered in order to prioritise maintenance needs.
Worst credible strength (WCS_{dc} and WCS_{dt})	The worst value of the compressive or tensile strength of concrete or reinforcement which the assessor, based on experience and knowledge of the material, realistically considers can be obtained in the deteriorated structural element under consideration.

1. Scope

Aspects covered

1.1 This document shall be referred to for managing and rehabilitating deteriorated concrete highway structures including defects affecting structural integrity.

1.2 The document sets out requirements, advice and additional considerations which shall be used for the design of concrete repairs and ongoing monitoring of concrete deterioration in highway structures.

1.3 The document also includes specific requirements which shall be used for structures suffering from internal deterioration of concrete (IDC) and thaumasite sulfate attack (TSA).

Implementation

1.4 This document shall be implemented forthwith for all schemes involving repair or management of concrete highway structures on the Overseeing Organisations' motorway and all-purpose trunk roads according to the implementation requirements of GG 101 [Ref 14.N].

Use of GG 101

1.5 The requirements contained in GG 101 [Ref 14.N] shall be followed in respect of activities covered by this document.

2. Competency of personnel

2.1 Personnel undertaking assessment of the load carrying capacity of the existing structure shall have the necessary and relevant skills, knowledge and experience in assessment of highway structures.

2.2 Personnel undertaking inspection of the existing structure shall fulfil the requirements given in CS 450 [Ref 13.N].

2.3 Personnel undertaking design and specification of concrete repairs shall fulfil the requirements given in BS EN 1504-9 [Ref 28.I].

2.4 Personnel undertaking design and specification of cathodic protection systems associated with concrete repairs shall fulfil the requirements given in BS EN ISO 15257 [Ref 8.N] and be certified to Level 4 in the reinforced concrete structures application sector.

3. Management of deteriorated concrete structures

3.1 Management of a concrete highway structure shall commence as soon as a structure is commissioned to ensure it remains safe for the public to use without significant impact on long term performance or durability.

NOTE Ingress to the concrete by air, moisture, chlorides and other deleterious substances can occur quickly, but visible signs of concrete deterioration due to reinforcement corrosion can take several years to appear.

3.1.1 Defects and causes of deterioration should be identified and recorded so a plan for structure maintenance or other structure management can be agreed with the Technical Approval Authority (TAA) or the Overseeing Organisation (OO).

NOTE 1 Structure maintenance can include repair as part of reactive essential maintenance, and can also include proactive measures as part of preventative maintenance.

NOTE 2 Measurement of chloride contamination and estimation of the rate of deterioration can enable prediction of maintenance intervention points.

3.2 Deteriorated concrete highway structures shall be managed in accordance with the procedures set out by the OO.

NOTE Guidance on the general principles behind structure management strategies can be found in Section 5 of Management of Highway Structures - Code of Practice MHS CoP [Ref 25.I].

3.2.1 A structure management plan (SMP) may be developed, where required by the OO for a particular deteriorated concrete structure.

3.2.2 A SMP may be required for large, structurally complex or high profile structures.

NOTE A SMP can also be termed a lifecycle plan. More information can be found in the London Bridges Engineering Group Good Practice Guide LoBEG [Ref 24.I].

3.2.3 A SMP should be reviewed and updated periodically to ensure it remains relevant and effective.

3.2.4 Periodic review intervals should be agreed with the TAA or the OO.

3.2.5 A SMP should include details of future requirements for inspection and maintenance of the structure and details of any monitoring of impressed current or galvanic anodes installed.

NOTE 1 Inspection intervals are generally the same as for general inspections (GIs) and principal inspections (PIs), unless cathodic protection (CP) systems have been installed as well.

NOTE 2 Guidance about inspection of CP systems can be found in BS EN 12696 [Ref 6.I] .

4. Structure asset renewal scheme planning

General

4.1 Appraisal and prioritisation of schemes for maintenance or renewal of a deteriorated concrete structure shall follow the Overseeing Organisation's asset renewal planning procedure.

NOTE 1 Schemes where structure stability/safety are an issue can be given higher priority.

NOTE 2 General information about maintenance planning can be found in UKRLG Management of Highway Structures - A Code of Practice MHS CoP [Ref 25.I].

Cost estimation of options

4.2 Cost estimates of all scheme options for maintenance or renewal of a deteriorated structure shall be prepared and recorded on file.

4.2.1 Cost estimates should be prepared in accordance with the requirements given in CD 355 [Ref 38.N], using whole-life costing principles.

NOTE 1 The cost of a scheme can be significantly affected by lane closure constraints, interaction with other network schemes and availability of material and human resources.

NOTE 2 The cost of managing traffic can represent a high proportion of the total scheme cost.

4.2.2 A return period for concrete repairs of between 20 and 25 years should be assumed unless otherwise agreed with the TAA.

NOTE It is not possible to accurately forecast the life of concrete repairs, but the recommended return periods represent the approximate frequency of maintenance interventions for significant concrete repair work.

4.2.3 Cost estimates for all concrete repair and rehabilitation options should be incorporated within the structure management options report (SMOR).

5. Determining structure condition

General

5.1 Where rehabilitation of a deteriorated concrete structure is identified as part of a forward programme of work, the condition of the concrete shall be established.

5.1.1 Work should include a desk study and can also include additional investigation consisting of survey, concrete sampling, testing and structural assessment to determine the current condition.

NOTE Results of the condition investigation are intended to assist the development of:

1) *risk management strategy;*
2) *renewal justification;*
3) *strategy for protection or repair;*
4) *design work.*

Desk study

5.2 A desk study, including a review of all available historical data for the structure or structural elements being considered, shall be undertaken.

NOTE An initial indication of structure condition can be obtained from the desk study.

5.2.1 Limited localised structural assessment may also be undertaken.

5.2.2 Sources of existing data to establish the condition of a concrete structure may include the following where available:

1) as built records/drawings;
2) reports from general/principal/special inspections;
3) assessment reports (calculations/AIP/report);
4) ad-hoc surveys;
5) testing information/reports;
6) monitoring information;
7) previous repair history;
8) details of structure modifications;
9) relevant correspondence;
10) construction records;
11) examples of deterioration in other structures with similar detailing or constituent materials.

Additional investigation of concrete condition

Justification for additional investigation

5.3 Archive survey and testing information for a structure or structural element shall be reviewed before additional investigation is proposed.

5.3.1 Sufficient historical data should be available or be obtained to establish an accurate picture of concrete condition.

NOTE 1 Test information is crucial for assessing the condition of a structure and the likelihood of reinforcement corrosion.

NOTE 2 Where practicable and safe to undertake, an ad-hoc site visit and limited visual inspection can confirm whether additional investigation is necessary.

NOTE 3 Some historical data can still be representative of the current concrete condition, but other information can be too old to be useful.

Scope of additional investigation

5.4 The scope and timing of additional investigation shall be agreed with the TAA before commencing work.

5.4.1 Additional investigation may include the following:

1) survey of superficial defects, spalling, concrete delamination, cracking and water staining;
2) survey by instrument of levels or concrete cover;
3) survey by instrument of electrical potential, moisture content, electrical resistivity;
4) exposure and survey of reinforcement, strands or post-tensioning equipment to check condition;
5) in situ measurement of concrete pH;
6) removal of concrete samples for testing;
7) laboratory testing of concrete samples.

5.4.2 Additional investigation should include reporting of results.

5.4.3 Investigation may be done in stages if the scheme or structure is complex and large, or if subsequent repair works are to be divided into phases.

5.4.4 A pilot investigation to determine the feasibility, time and cost of main works investigation may be undertaken with agreement from the TAA.

5.5 Additional investigation of deteriorated concrete elements to support design of repairs shall be undertaken early in the design process.

NOTE 'Early in the design process' includes during preliminary or detailed design, and before a repair contract commences on site.

5.5.1 Additional investigation may be classified as a special inspection in accordance with CS 450 [Ref 13.N].

NOTE Procedures for structure inspections are given in CS 450 [Ref 13.N] and additional guidance can be found in the Inspection Manual for Highway Structures, Volumes 1 and 2. IMHS V1 [Ref 22.I] IMHS V2 [Ref 23.I].

Additional investigation for top of concrete bridge deck usually covered by surfacing/waterproofing

5.6 Causes of frequently occurring defects in bituminous layers of the road pavement pavement layers (potholes) above bridge decks shall be investigated.

5.6.1 The cause of a defect may be further investigated by excavating a trial pit.

NOTE 1 Defects can be caused by deterioration of the underlying concrete.

NOTE 2 Requirements/advice about localised repairs to deck waterproofing damaged while excavating trial pits can be found in CD 358 [Ref 39.N].

5.6.2 The extent and depth of potholes should be measured and compared with the total depth of surfacing present.

NOTE Defects located only in the upper bituminous layers can be caused by debonding between the surface and binder courses, and hence not be an indicator of deck deterioration.

5.7 The condition of deck concrete and reinforcement shall be investigated when the deck is exposed during a re-waterproofing contract.

NOTE Water containing chlorides, can reach the concrete deck via a deteriorated waterproofing system or a leaking deck joint and cause concrete deterioration.

Additional investigation for concrete below ground

5.8 An investigation of buried substructure parts or elements shall be undertaken where risk of deterioration is assessed as significant.

NOTE 1 *Chlorides in solution can percolate through fill surrounding substructure elements adjacent to trafficked lanes e.g. faces of columns.*

NOTE 2 *Chloride contamination of concrete elements is common down to 1m below ground.*

NOTE 3 *A high concentration of sulfate in groundwater can also cause deterioration of concrete.*

5.8.1 A representative selection of substructure elements should be exposed for investigation, where chloride contamination or sulfate attack are likely.

5.8.2 Measures for ensuring stability of structural elements and adjacent parts of the highway carrying traffic should be planned, designed, approved and installed prior to any excavation commencing.

Additional investigation for critical structural elements

5.9 An investigation of critical structural features e.g. half-joints, deck hinges, shall be undertaken if required for confirming strength or stability during repair work.

NOTE *Guidance on verifying the condition factor being assumed for assessment is given in CS 466 [Ref 26.N] for assessment of concrete half-joints, CS 467 [Ref 25.N] for structural assessment of bridges with deck hinges and 'Hidden Defects in Bridges' CIRIA C764 [Ref 20.I].*

5.10 Investigation of concrete in post-tensioned or pre-stressed concrete members shall be planned to minimise the risk of damaging strands or tendons.

5.10.1 The precautions recommended in CS 465 [Ref 15.N] should be followed prior to and during investigation.

Schedule of investigation work

General

5.11 A schedule of work shall be prepared for additional investigation of concrete condition.

5.11.1 Investigation should focus on areas of a structural element most likely to be at risk from reinforcement corrosion.

NOTE 1 *Vulnerable areas can include concrete subject to water leakage through joints, exposed elements subjected to salt spray from road traffic, or elements below ground where adjacent backfill could potentially contain harmful contaminants (e.g. chlorides, sulfates).*

NOTE 2 *Guidance and recommendations for investigation work can be found in the Concrete Society's 'Diagnosis of deterioration in Concrete Structures - Identification of defects, evaluation and development of remedial action' CS TR54 [Ref 14.I] and the Concrete Bridge Development Group's 'Guide to Testing and Monitoring the Durability of Concrete in Structures' CS TG2 [Ref 19.I].*

5.11.2 Structure elements or parts of them potentially exposed to chlorides from road de-icing salts may extend to the following:

1) piers, columns, crossheads and abutments within eight metres of the edge of the carriageway;
2) piers, columns, crossheads and abutments under a deck joint, but with no effective drainage provision;
3) bearing shelves, bearing plinths, ballast walls and deck ends with a deck movement joint above them;
4) concrete parapets, parapet plinths (all faces), string courses, fascias and concrete deck surfaces not protected by deck waterproofing;
5) deck beams, soffits and cantilevers directly over and within eight metres of the edge of the carriageway;
6) parts of wingwalls within eight metres of the edge of the carriageway;
7) parts of retaining walls within eight metres of the edge of the carriageway.

Survey of existing condition

5.12 All areas of visually exposed concrete shall be surveyed.

5.12.1 Hidden/covered concrete should also be surveyed in circumstances described for additional investigation for concrete below ground and for additional investigation for critical structural elements (se clauses 5.8 and 5.9).

5.12.2 Some areas may be omitted from the survey if previous historical information is judged to be still relevant for the current investigation.

5.12.3 Surveys should include a suite of observations about the concrete surface condition and detection of concrete defects to the depth of the outermost reinforcement.

5.12.4 Observations may include superficial defects, concrete delamination (by concrete sounding/hammer test), condition of old repairs, structural cracking and evidence of water leakage.

NOTE 1 Guide to testing and monitoring the durability of concrete structures CS TG2 [Ref 19.I] provides advice about what defects can be included in the survey.

NOTE 2 Guidance and recommendations for the survey can be found in the Inspection Manual for Highway Structures IMHS V1 [Ref 22.I] and CS 450 [Ref 13.N].

5.12.5 Measurement of spot heights and reinforcement cover survey may also be undertaken where required.

5.12.6 An exposed deck area should be inspected for surface defects, and a concrete sounding (delamination) survey undertaken.

NOTE A level survey of a deck surface previously damaged by a road planer can provide useful information for structural assessment and subsequent reconstruction of the original profile.

5.13 Requirements for survey shall be identified and input to a schedule of work for condition investigation.

Instrument survey of reinforced concrete

5.14 Instrument surveys shall be carried out to assess the future risk of concrete deterioration by corrosion of reinforcement.

5.14.1 Survey may include measurements by proprietary instrument of electrical corrosion potential, electrical resistivity or moisture content.

5.15 During planning of an electrical potential survey, structure records shall be examined to determine whether any concrete members have been impregnated, surface coated or whether previous repair patches contain galvanic anodes.

NOTE Electrical potential can be affected by the presence of hydrophobic pore lining impregnant within the concrete, surface coatings or galvanic anodes cast within the concrete.

5.16 Measurements of electrical potential shall be taken within identified test areas.

5.16.1 Electrical potential readings may be taken over a wider area of a structural element, to explore extensive zones of high corrosion potential, or to support a design for restoring protection to vulnerable reinforcement.

NOTE See CD 370 [Ref 6.N] for further details of how vulnerable reinforcement can be cathodically protected.

5.16.2 Electrical potential measurements required over a wider area should be undertaken at a larger grid spacing.

5.16.3 Electrical potential of reinforcement within concrete half-joints may be measured within small diameter bore holes drilled close to critical reinforcing bars.

5.16.4 The bore should be created no closer to existing bars than the nominal cover for reinforcement.

5.16.5 Where appropriate, permanent connections to the reinforcement may be installed to facilitate future electrical potential survey as described in CD 373 [Ref 12.N].

5.16.6 Concrete resistivity testing to further confirm the risk of corrosion activity may be undertaken in accordance with CS TR60 [Ref 16.I] in appropriate circumstances.

5.16.7 Where cathodic protection is a feasible option for the rehabilitation of a structure, resistivity testing of old repairs should be undertaken.

NOTE Resistivity testing of previously repaired areas is only necessary where a resin-based product was used.

5.16.8 Where the electrical resistivity of concrete is required, moisture content of the concrete should also be measured.

5.17 Requirements for instrument survey shall be identified and input to the schedule of work.

Sampling and testing of concrete

5.18 Sampling and testing of concrete shall be carried out within identified testing areas to support the assessment of corrosion risk.

5.18.1 Samples for testing chloride concentration and cement content should be evenly distributed within testing areas.

5.18.2 Samples for measuring the concentration of chloride should be taken at intervals of depth down to the main structural reinforcement.

5.18.3 Samples may be removed from deeper than the reinforcement where a chloride-based accelerating admixture was used in the original concrete mix.

5.18.4 Samples from deck top concrete to test for chloride and cement content should be removed within traffic lanes or over the whole deck.

5.18.5 Frequency of sampling should be between one and two samples per lane per span, or between one sample per $80m^2$ and one sample per $100m^2$.

5.18.6 Sampling frequency for structures more than 4 spans may be reduced after consultation with the TAA.

5.18.7 Samples for chloride and cement determination should be located where failure of the waterproofing membrane has occurred.

NOTE Further guidance can be found in "Determination of the chloride and cement contents of hardened concrete" BRE IP 21/86 [Ref 13.I], and Concrete Society reports CS TR32 [Ref 3.I] and CS TR60 [Ref 16.I].

5.18.8 Test holes may be taken in the concrete to detect the depth of carbonation penetration.

NOTE Carbonation is usually not an issue with higher strength concretes (>40MPa) used for highway structures, but can affect lower strength concrete or concrete with significantly reduced concrete cover.

5.18.9 Carbonation testing of an exposed deck surface may be omitted if the concrete has been adequately protected by waterproofing since construction.

NOTE Requirements for sampling and testing of structures suspected of suffering from IDC can be found in Section 8.

5.19 Requirements for sampling and testing of concrete shall be identified and input to the schedule of work.

5.19.1 The schedule for sampling and testing should also include reinstatement of sample holes.

Testing areas

5.20 Areas for undertaking targeted testing shall be identified for structure elements and input to the schedule or be shown on investigation drawings.

5.20.1 The location of a test area may be adjusted to a position where there appears to be the greatest risk of reinforcement corrosion.

NOTE *Areas at greatest risk include splash zones and areas below deck joints etc.*

5.20.2 Areas or components at lower risk of reinforcement corrosion can be included as a benchmark or control, but the number of these areas should be limited to 20% of the total.

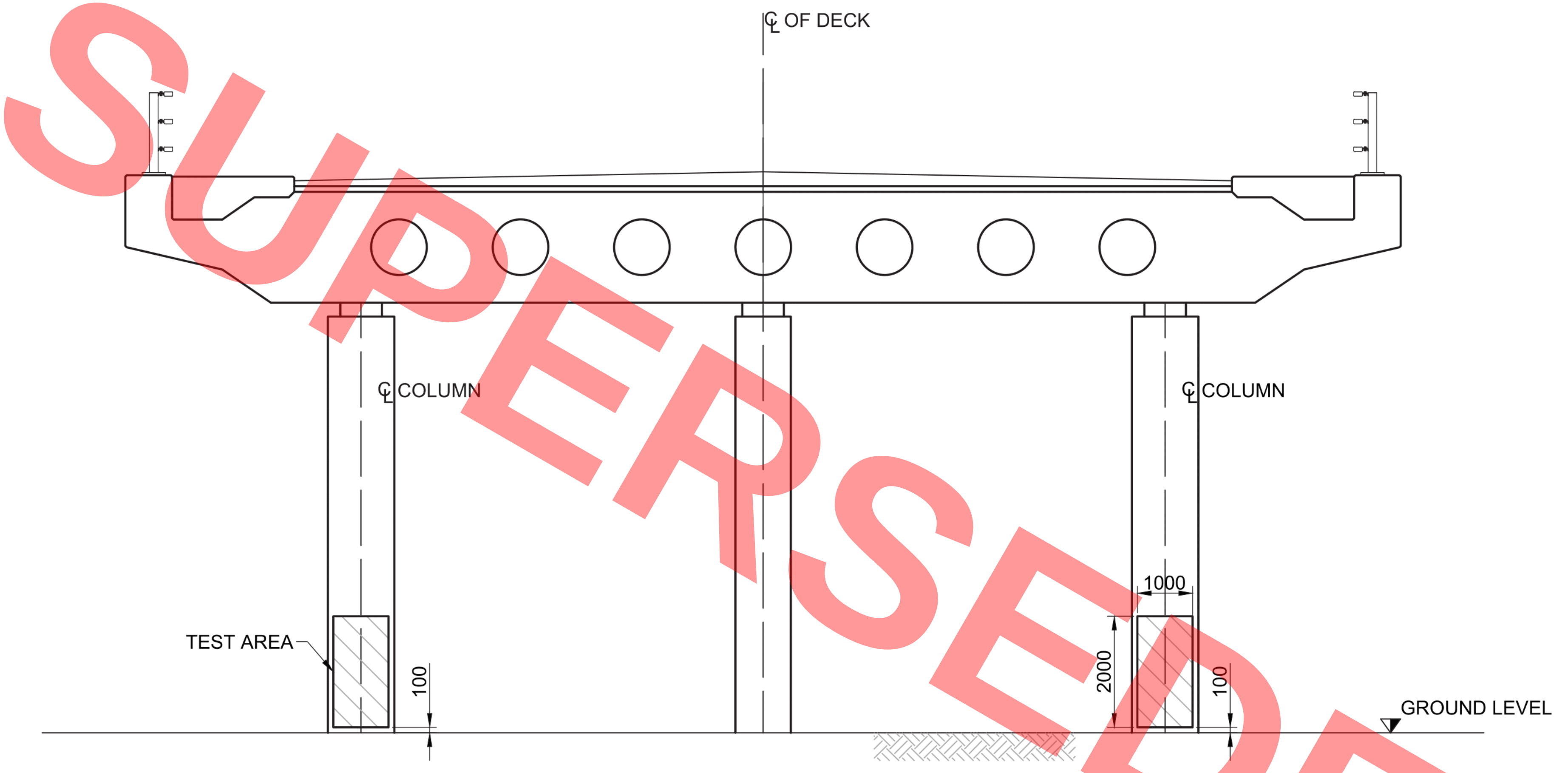
5.20.3 The size of test areas should generally be 2m x 1m, but the dimensions can be altered to suit the size and shape of the element being investigated and level of associated risk.

5.20.4 The location of test areas or test points should be accurately referenced to the structure.

Bridge substructures

5.21 Site-specific survey, sampling and testing of reinforced concrete piers, abutments, columns and crossheads subjected to spray from traffic at the lower level shall be undertaken in test areas located typically as shown in Figures 5.21a to 5.21c.

Figure 5.21a Location of test areas for reinforced piers and columns subject to salt traffic spray, bridges <26m wide



PIERS AND COLUMNS

Figure 5.21b Location of test areas for reinforced piers and columns subject to salt traffic spray, bridges >26m wide

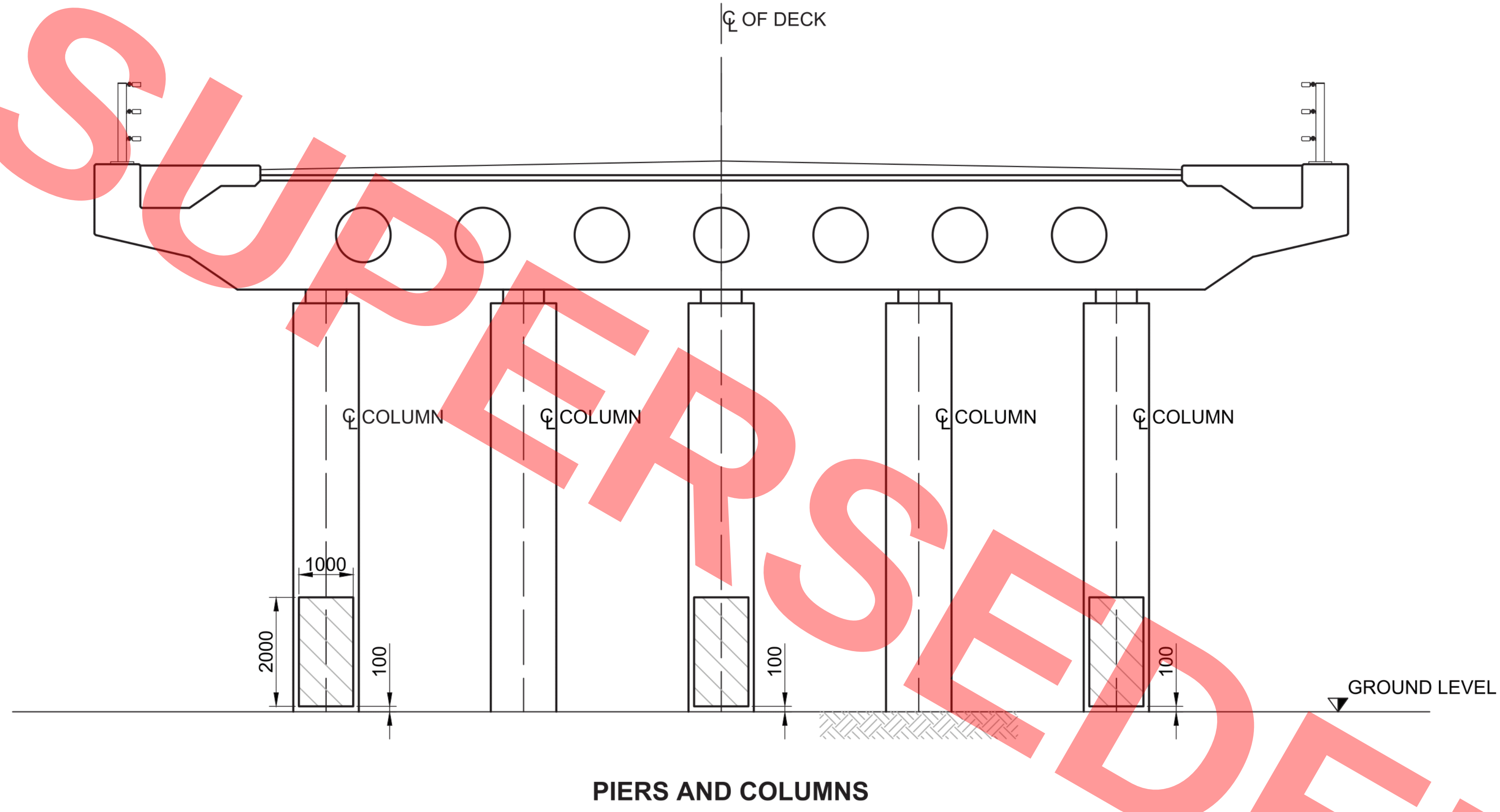
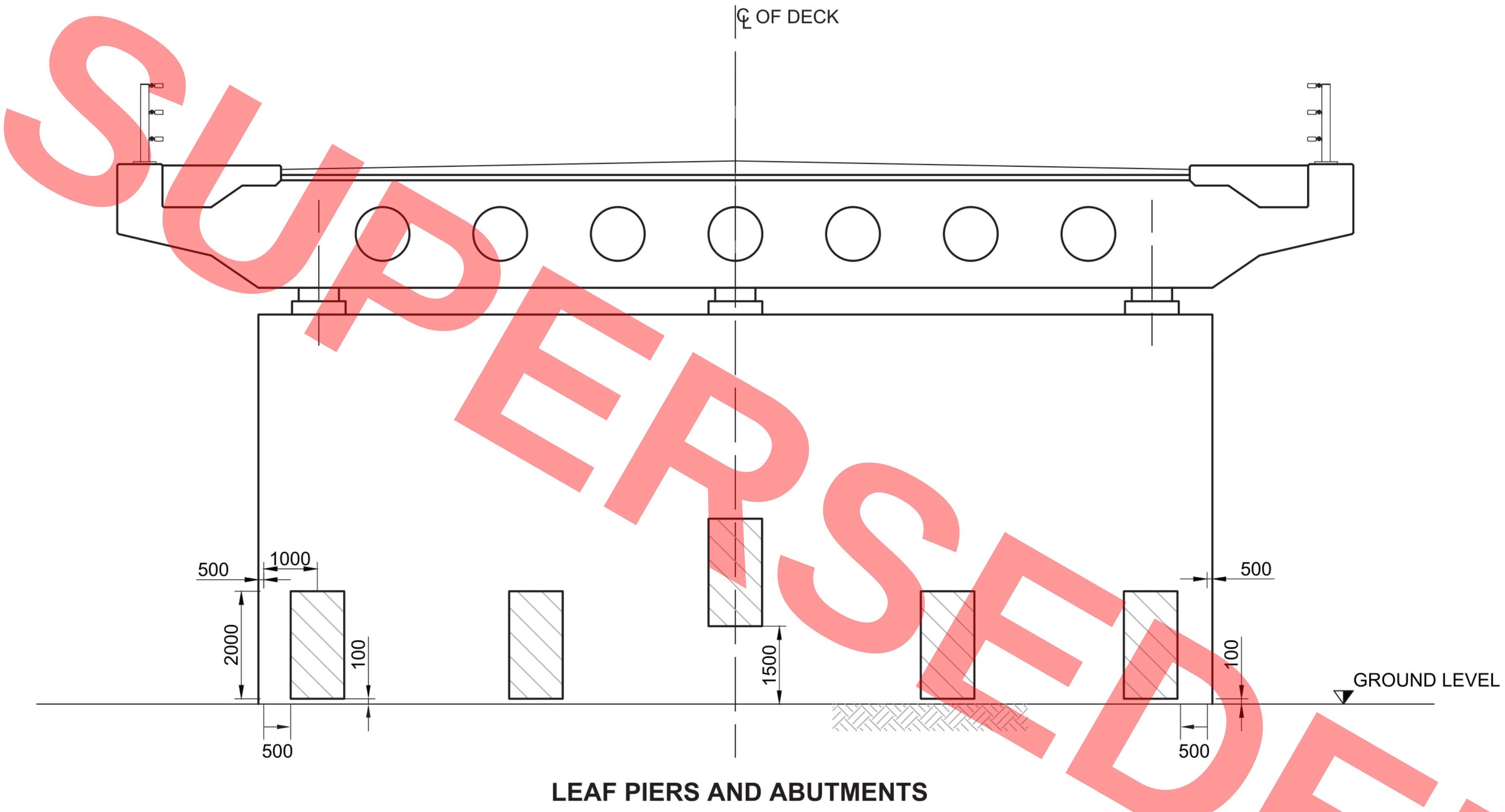


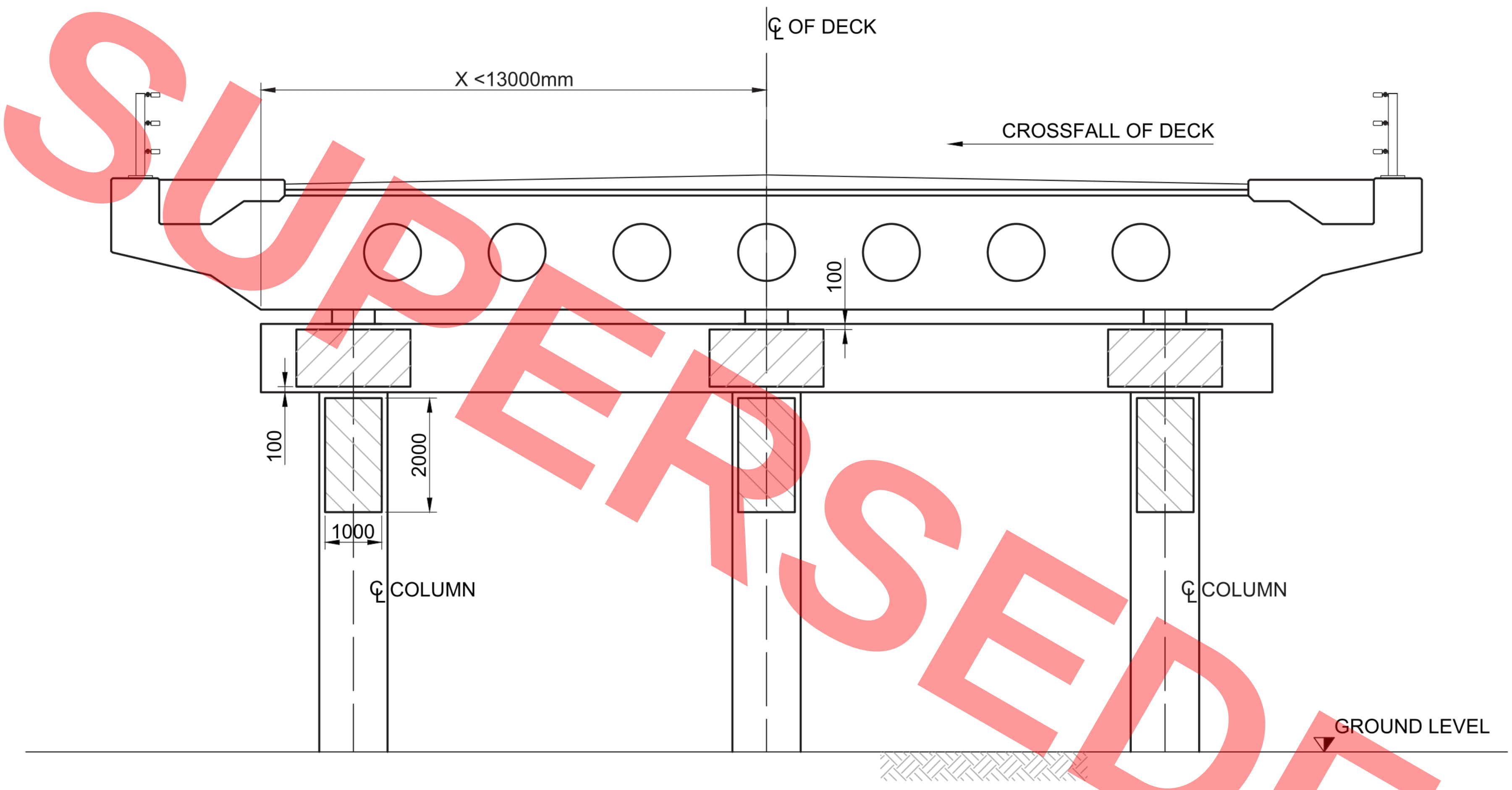
Figure 5.21c Location of test areas for reinforced leaf piers and abutments subject to salt traffic spray



Notes
1. Distance between centreline of test areas is approximately 7m
2. Minimum of 3 test areas; one area to be located on centeline of deck
3. Diagram is not to scale

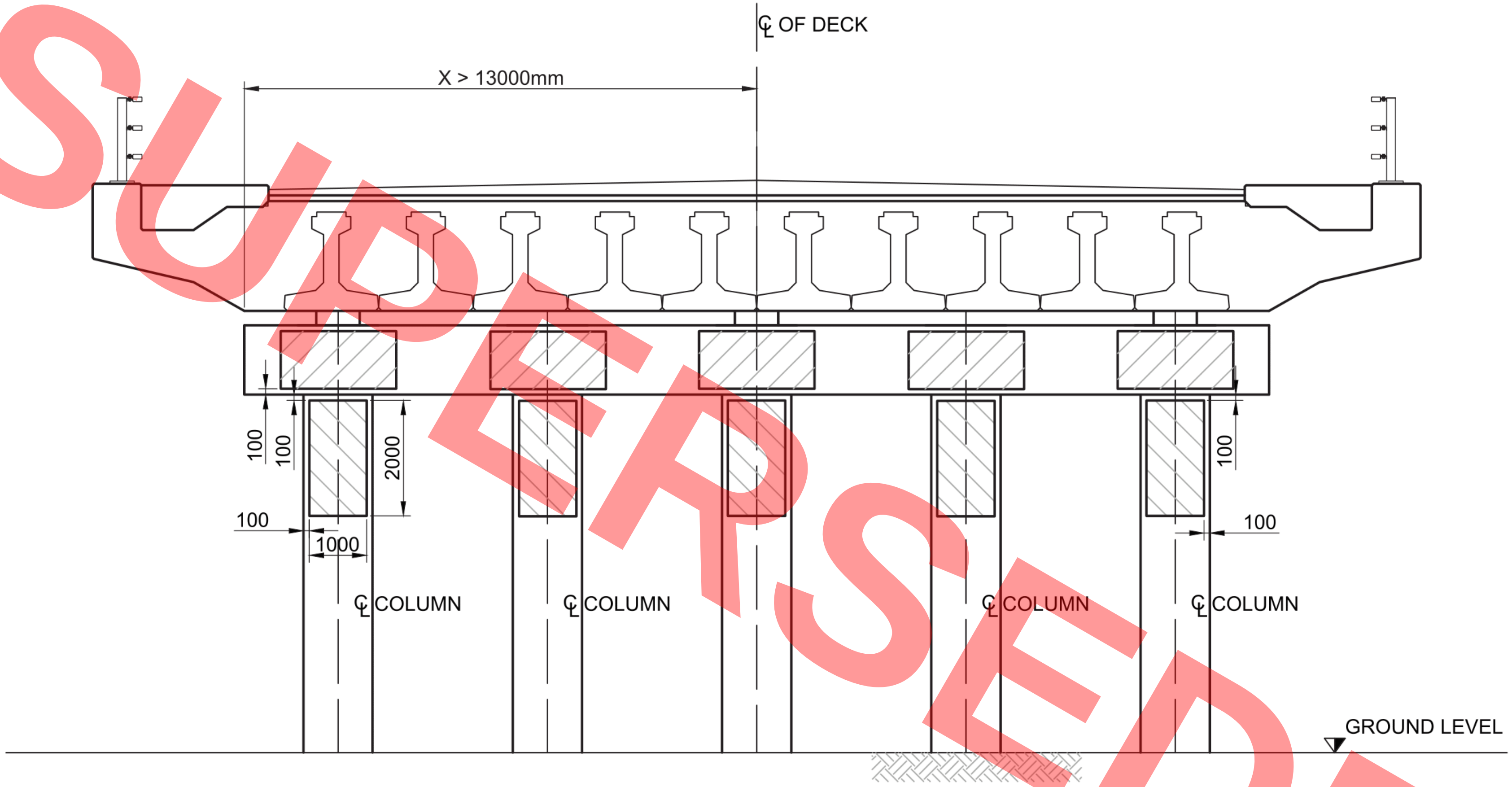
5.22 Site-specific survey, sampling and testing of reinforced concrete substructure elements subjected to water leakage through a deck joint above shall be undertaken in test areas located typically as shown in Figures 5.22a to 5.22c.

Figure 5.22a Location of test areas for reinforced piers, columns and cross heads with a deck joint above, bridges <26m wide



PIERS COLUMNS AND CROSSHEADS

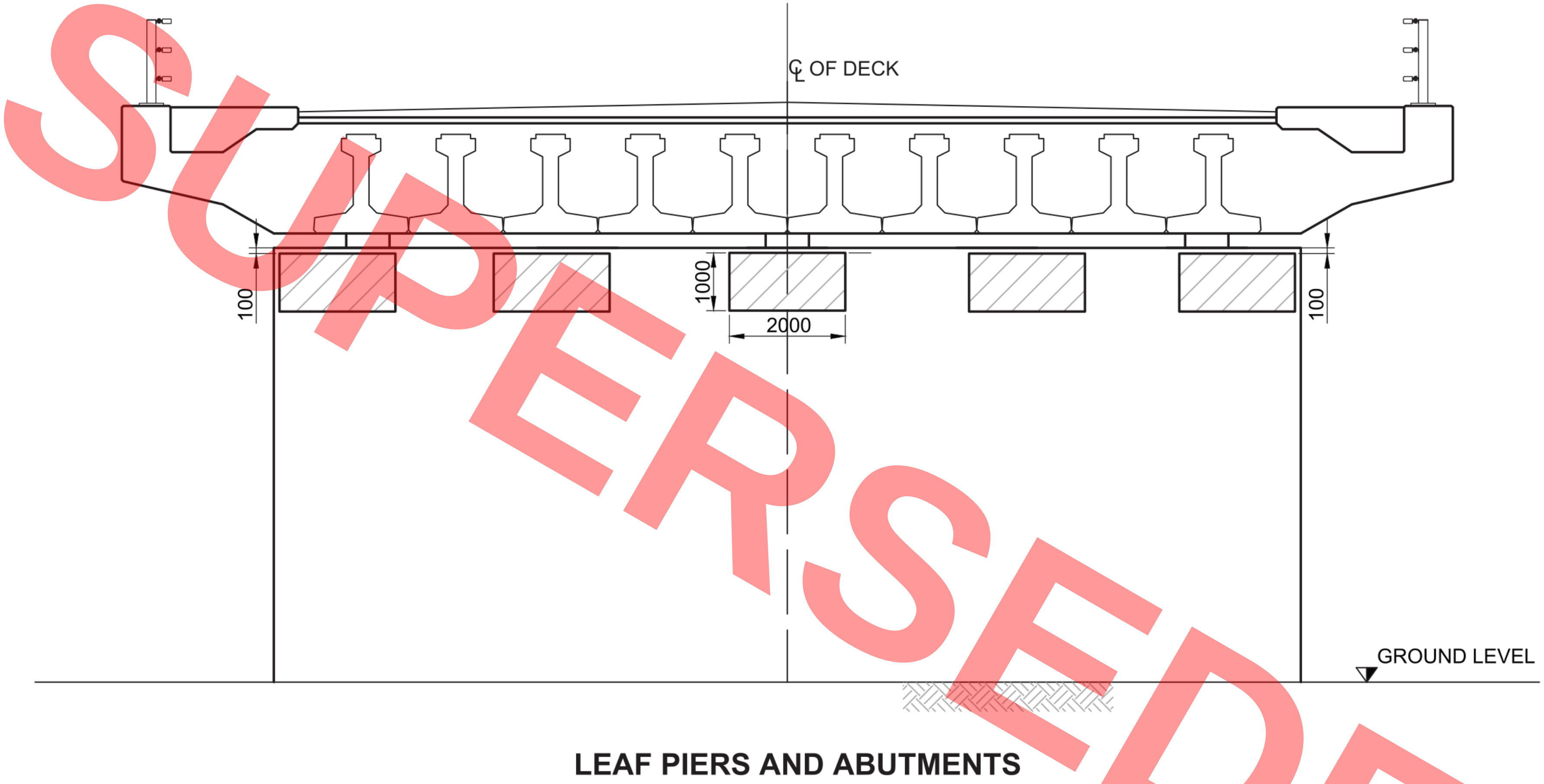
Figure 5.22b Location of test areas for reinforced piers, columns and crossheads with a deck joint above, bridges >26m wide



PIERS, COLUMNS AND CROSSHEAD

Notes
1. Distance between centreline of test areas is approximately 7m
2. One test area on crosshead to be located at centreline of deck
3. Diagram is not to scale

Figure 5.22c Location of test areas for reinforced leaf piers and abutments with a deck joint above

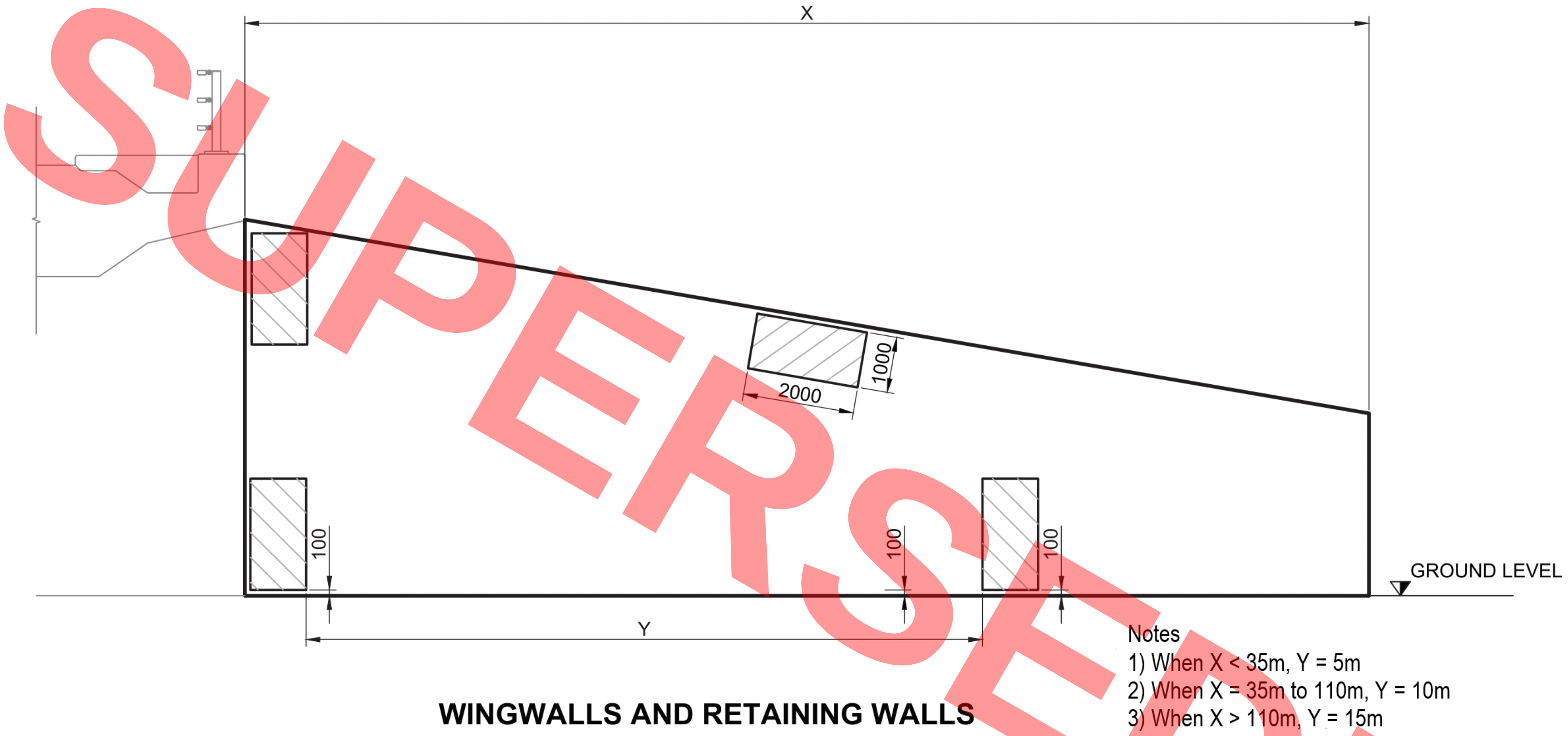


Notes
1. Distance between centreline of test areas is approximately 7m
2. Minimum of 3 test areas; one area located on centreline of deck
3. Diagram is not to scale

5.22.1 Where substructure members are located below deck joints and are also subjected to salt spray from traffic at a lower level, the test areas shown in Figures 5.21a to 5.21c and 5.22a to 5.22c should be combined.

5.23 Site-specific survey, sampling and testing of reinforced concrete wingwalls and retaining walls shall be undertaken in test areas located typically as shown in Figure 5.23.1.

Figure 5.23 Location of test areas to wingwalls or retaining walls subject to traffic spray or leakage from deck joint



5.23.1 The test area should be repeated at distances of either 5m, 10m or 15m along a horizontal line, depending upon the exposure zone and length of the structure.

Bridge superstructures - elements not covered by waterproofing/surfacing

5.24 Site-specific survey, sampling and testing of reinforced concrete deck soffits, cantilevers, diaphragms and reinforced concrete near half-joints and/or hinge joints shall be undertaken in test areas located typically as shown in Figures 5.24a to 5.24c.

Figure 5.24a Location of test areas to under bridge deck soffits subject to traffic spray. Dual 3 lane carriageway deck shown - Dual 2 lane carriageway is similar

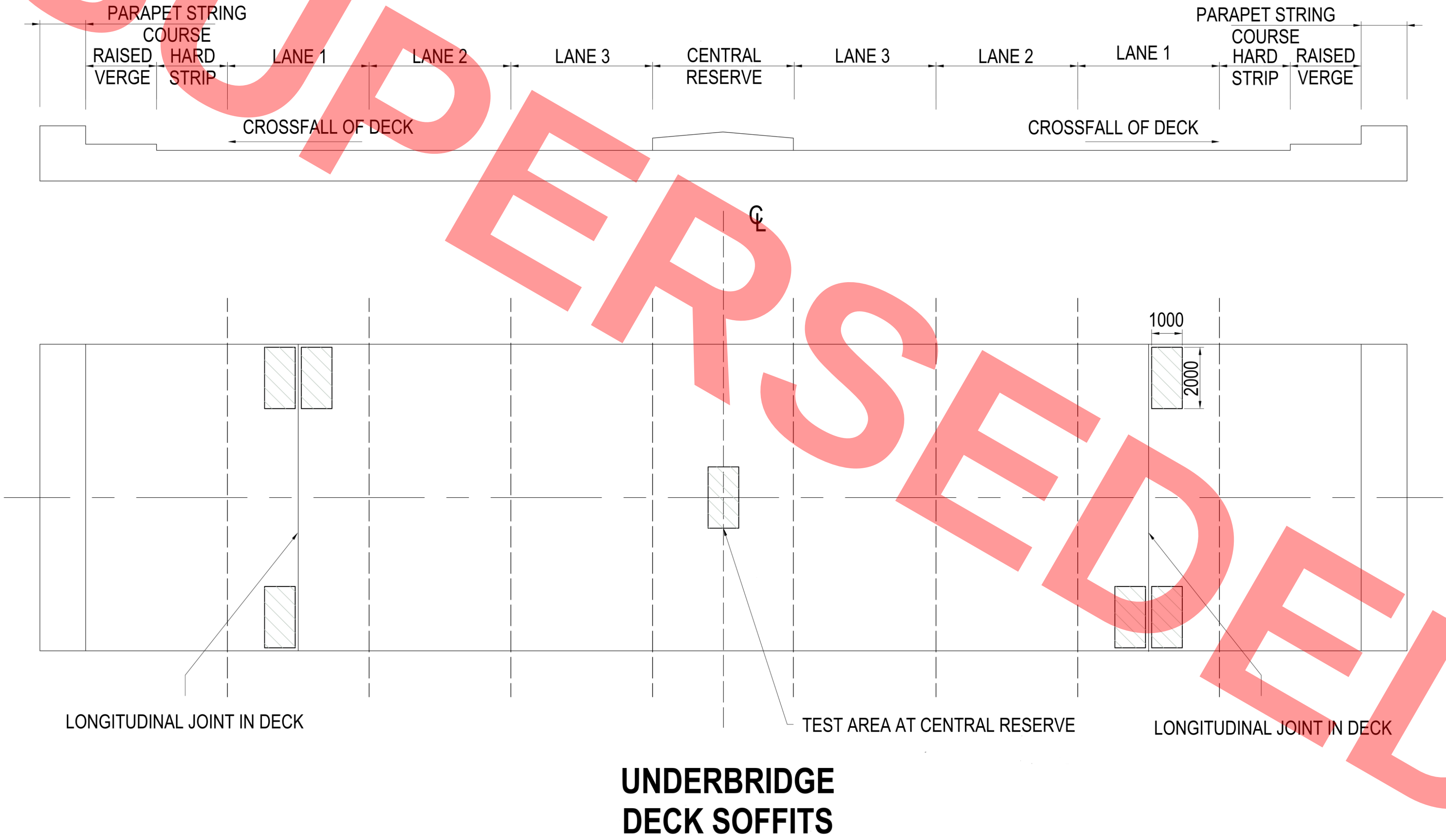
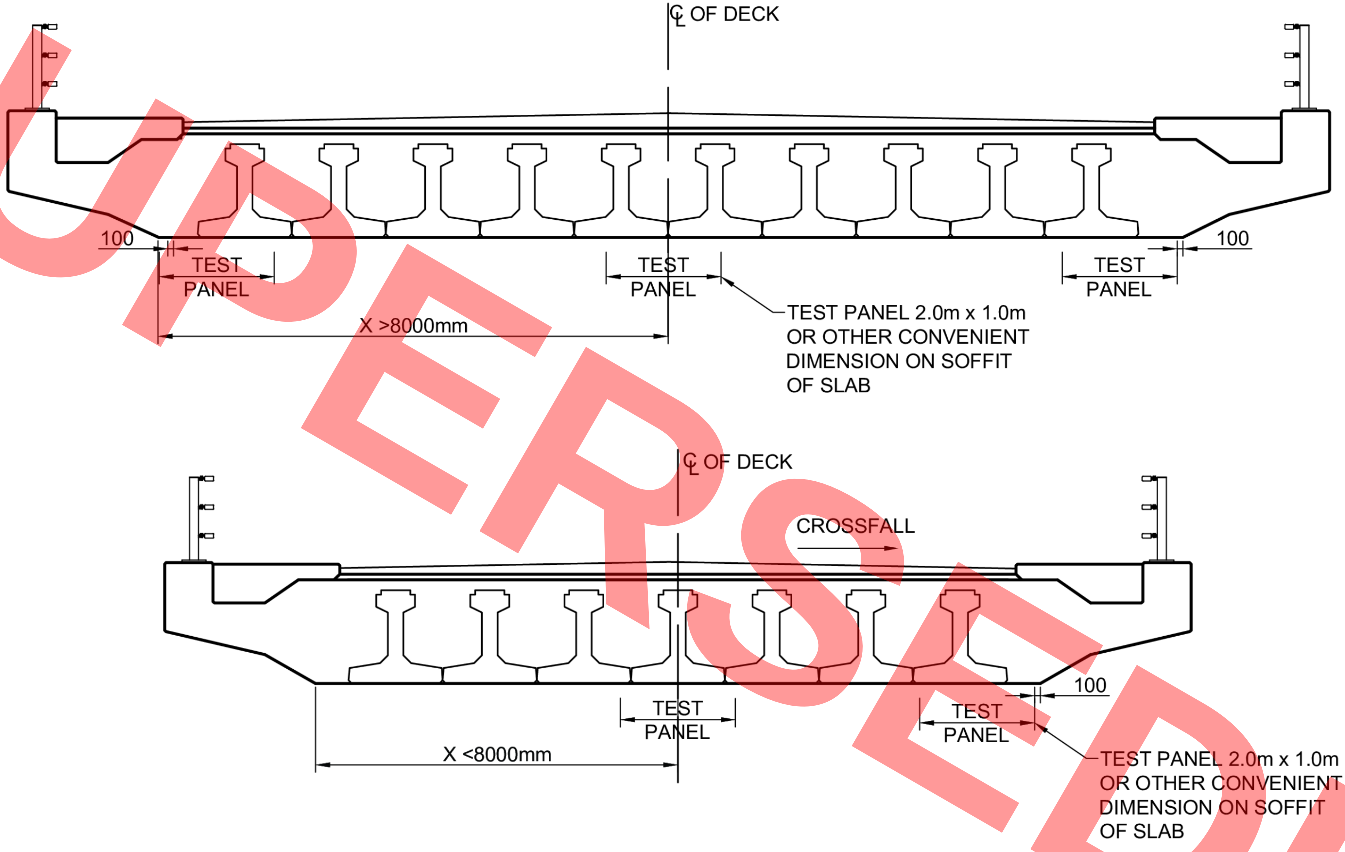
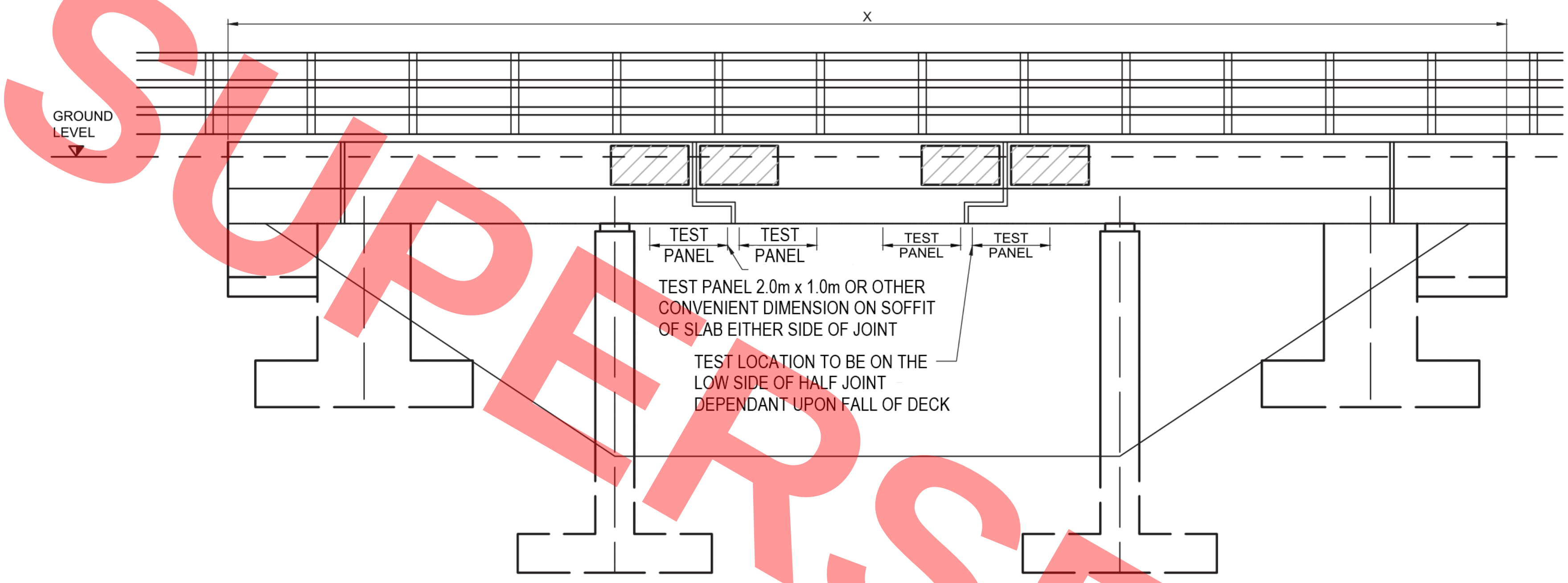


Figure 5.24b Location of test areas of deck soffits subject to traffic spray



DECK SOFFITS

Figure 5.24c Location of test areas to deck containing half-joints



ELEVATION WITH HALF-JOINTS

5.24.1 For parts of reinforced concrete deck soffits subjected to deck joint leakage, the test areas should be on both sides of the joint as shown in Figure 5.24c, and at the leading and trailing edge of the deck as shown in Figures 5.24a and 5.24b.

5.24.2 Test areas should be located on the fascia of the deck slab, deck soffit, cantilever soffit or the edge beam downstand whichever is assessed to be more appropriate.

5.24.3 Where the visible condition and apparent exposure of multiple beams is broadly similar, testing of their condition may be undertaken on one or more beams which are typical of the rest.

5.25 Site specific survey, sampling and testing of reinforced concrete parapets and parapet plinths shall be undertaken in test areas located typically as shown in Figures 5.25a and 5.25b.

Figure 5.25a Locations of test areas to overbridge parapets and parapet plinths subject to traffic spray

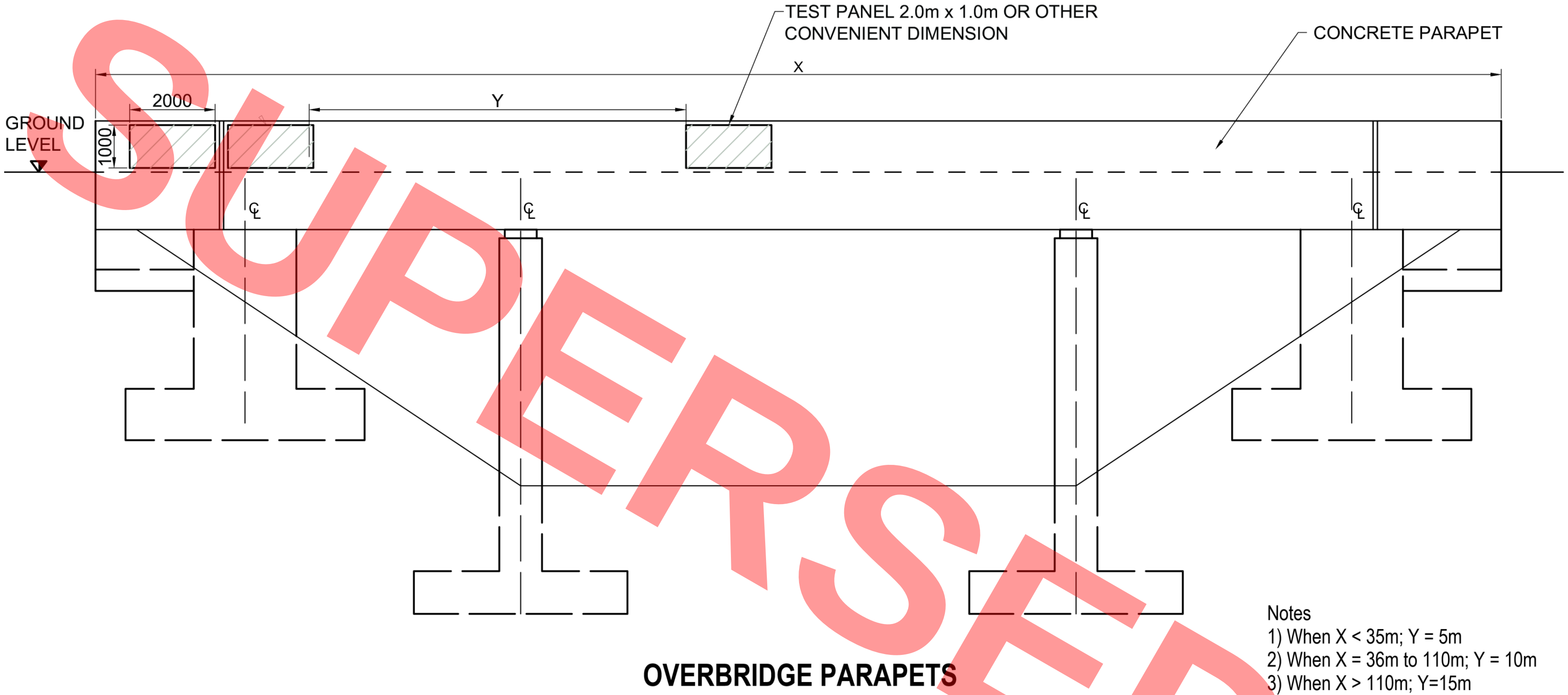
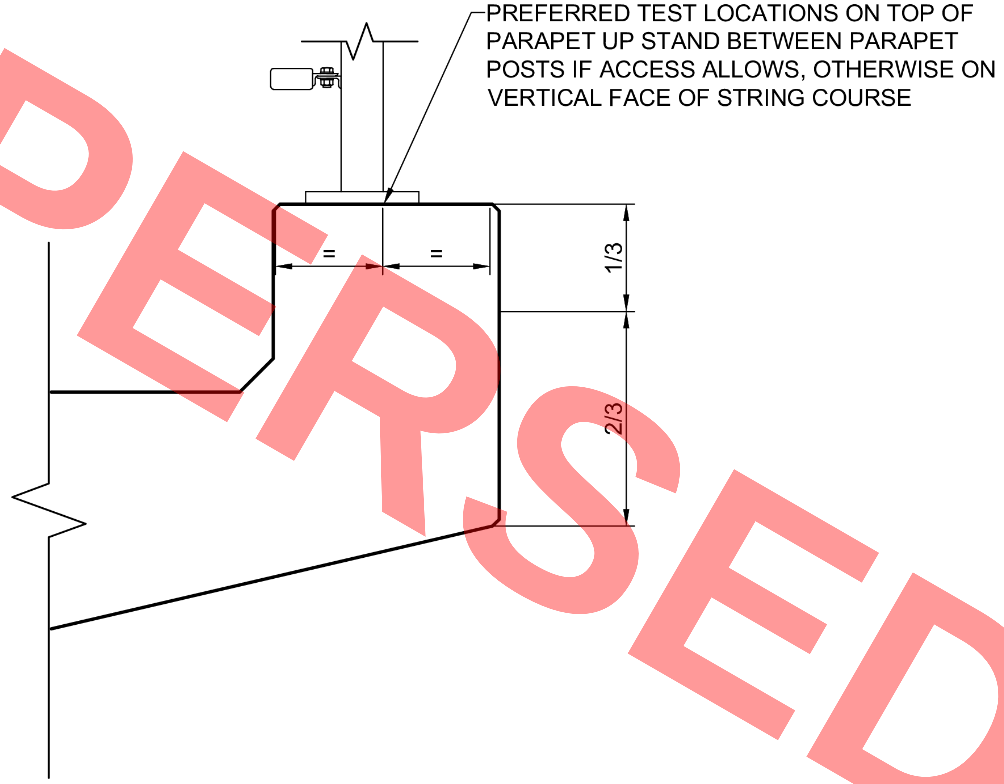


Figure 5.25b Location of test areas to parapet stringcourse beams subject to traffic spray



PARAPET STRING COURSES

5.25.1 Test areas should be located on the face of concrete parapets adjacent to traffic.

5.25.2 Test areas should be approximately 2m x 1m or other convenient dimensions, located approximately 100mm above the bottom edge of the concrete parapet and repeated at a suitable interval of 5m, 10m or 15m according to parapet length (see Figure 5.25a).

5.25.3 For reinforced concrete parapet plinth or string course subjected to traffic spray the test area should be approximately 1m long and repeated at a suitable interval of 5m, 10m or 15m according to length of plinth (see Figure 5.25b).

5.25.4 Testing may be undertaken on top of the plinth if access allows, otherwise towards the top of the exposed vertical face of the upstand.

Bridge superstructures - elements covered by waterproofing/surfacing

5.26 The test area shall comprise the whole of the exposed deck or just trafficked lanes.

5.26.1 For structures with more than 4 spans, a representative number of spans may be used for the survey and testing areas, following agreement with the TAA.

Site sampling

5.27 Sampling shall be carried out by a competent testing organisation accredited for the work required.

5.27.1 The testing organisation should be UKAS Accredited, accredited by an equivalent European Accreditation body which is party to a Multilateral Agreement (MLA) with UKAS, or any equivalent International Accreditation Forum (IAF) MLA signatory with a scope which includes the relevant standard(s).

5.28 Samples shall be removed from the concrete at a minimum of two locations within each test area and be taken to a laboratory for testing.

5.28.1 Samples for measuring chloride concentration and cement content should be taken from locations near to the most negative electrical potentials measured.

NOTE The most negative potential readings tend to be located close together.

5.28.2 Sampling for measuring chloride and cement content should undertaken using the procedures described in "Determination of the chloride and cement contents of hardened concrete" BRE IP 21/86 [Ref 13.I] and in accordance with the recommendations of CS TR60 [Ref 16.I] and CS TR32 [Ref 3.I].

NOTE The method described in BRE IP 21/86 [Ref 13.I] provides a simple method of estimating chloride concentration, but the result is not as accurate as laboratory testing.

5.28.3 The diameter of sample holes should be small and a function of aggregate size.

5.28.4 Prior to drilling holes, the position of existing reinforcement should be detected and marked on the concrete to minimise the risk of damaging bars.

NOTE A cover meter is often used to detect the position of existing reinforcement.

5.29 Removal of cored samples for determining compressive strength shall be carried out using the method in BS EN 12504-1 [Ref 32.N].

5.29.1 Sampling holes should be reinstated promptly following completion of the investigation.

Site testing

General

5.30 Tests shall be carried out by a competent testing organisation accredited for the testing work required.

5.30.1 Tests may be undertaken by instrument or by chemical reagent.

5.30.2 The testing organisation should be UKAS Accredited, accredited by an equivalent European Accreditation body which is party to a MLA with UKAS, or any equivalent IAF MLA signatory with a scope which includes the relevant standard(s).

Measurement of electrical corrosion potential by half-cell survey

5.31 An electrical potential survey using half-cell test equipment shall be carried out at node points defined by a grid marked on the concrete within the test areas.

5.31.1 The half-cell survey method described in 'Method for Half-Cell Potentials of Uncoated Reinforcing Steel in Concrete' ASTM C876 [Ref 27.I] should be used for measuring corrosion potential.

NOTE 1 Where measuring the potential of steel with respect to the electrode, the convention is for potentials to be negative.

NOTE 2 Portable silver/silver chloride reference electrode (SSC [0.5M KCl], Ag/AgCl/0.5M KCl) are commonly used in the UK for half-cell potential surveys.

NOTE 3 The ASTM C876 [Ref 27.I] infers that a saturated copper/copper sulfate electrode (CSE) is the normal reference electrode used for measuring corrosion potential using half-cell survey. The advice reflects practice in the United States, but use of a CSE electrode is not recommended in the UK.

NOTE 4 Significant errors can arise from the use of CSE electrodes and their use is discouraged.

5.31.2 A 0.3m x 0.3m grid spacing should generally be used for taking readings of electrical potential.

5.31.3 More potential readings at a closer grid spacing may be taken if the potential gradient is greater than 100mV between readings taken at 0.3m x 0.3m spacing.

5.31.4 The survey grid spacing may be reduced near to vulnerable areas like transverse joints or half-joints.

5.31.5 A 2m x 2m grid spacing should be used for the top of a concrete deck exposed during re-waterproofing contract.

5.31.6 Procedures should be in accordance with the Transport Research Laboratory Application Guide AG9 , "The Half-Cell Potential Method of Locating Corroding Reinforcement in Concrete Structures" TRL AG9 [Ref 42.I] and in accordance with the recommendations of CS TR60 [Ref 16.I] and CS TG2 [Ref 19.I].

5.31.7 The condition of the concrete substrate should be examined before undertaking the survey to decide if it is suitable for the half-cell survey method.

5.31.8 Where there are many areas of damp concrete, consistency in results may be achieved by pre-wetting the whole surface.

NOTE 1 Areas of damp concrete can affect corrosion potential measurements, particularly on thin deck slabs.

NOTE 2 Potential readings of a wetted concrete surface can be slightly more negative than for dry concrete.

5.32 Electrical continuity of the reinforcement shall be checked and confirmed prior to carrying out the corrosion potential survey.

NOTE Electrical continuity of reinforcement can be locally interrupted.

5.32.1 Where no electrical continuity exists between bars in a test area, a temporary connection may be established to enable corrosion potential measurements to be made.

NOTE Readings obtained can be affected by the changes in electrochemistry.

5.32.2 The location of survey areas may need to be moved if it is not possible to obtain half-cell survey readings.

5.32.3 Where deep holes are required for measuring corrosion potential near to critical bars, concrete cover should be measured frequently during drilling to ensure bars are not damaged.

NOTE 1 A 'deep' hole extends beyond the first layer of reinforcement.

NOTE 2 Deep holes can be required where investigating corrosion potential of half-joint reinforcement.

5.33 Where corrosion potential readings indicate there is a high risk of reinforcement corrosion, but concrete is intact with no detectable delamination, the actual condition of the reinforcement shall be investigated by exploratory breakout.

NOTE *Anaerobic corrosion of deck reinforcement can occur beneath waterproofing in the presence of moisture and chloride contamination even if there is no or little delamination of concrete.*

Testing of carbonation penetration

5.34 Carbonation depth shall be tested at locations where concrete dust samples have been collected in accordance with BS EN 14630 [Ref 24.N], using the recommendations of CS TR60 [Ref 16.I].

NOTE 1 *The depth of carbonation is usually determined by spraying the freshly fractured surface of concrete with phenolphthalein solution.*

NOTE 2 *The depth of carbonation can be determined by petrographic examination on polished samples obtained from cored specimens.*

Measurement of concrete cover to reinforcement

5.35 An electromagnetic cover meter shall be used to detect reinforcement bars positioned parallel and perpendicular to the concrete surfaces, using the method described in BS 1881-204 [Ref 34.N].

NOTE *Measured depth of cover to reinforcement can be verified by carefully breaking-out a small pocket of concrete e.g. 100x100mm at each test panel.*

Measurement of electrical resistivity

5.36 Measuring electrical resistivity of concrete shall be in accordance with the procedure given in CS TR60 [Ref 16.I].

5.36.1 Portable equipment based on the Wenner four-probe technique may be used or an alternative approach using a square wave AC current to accommodate the effects of a poor surface contact.

NOTE *Resistivity is usually related to moisture content of the concrete, but concrete density and strength are also factors affecting readings.*

Measurement of moisture content

5.37 Moisture content of concrete shall be measured using a suitable proprietary instrument.

5.37.1 Portable measuring equipment may be used.

Other non-destructive testing

5.38 Non-destructive testing shall comply with the requirements of CS 464 [Ref 19.N].

Laboratory testing

5.39 The tests described in the following sub-sections shall be carried out in a laboratory by a competent testing organisation accredited for the testing work required.

5.39.1 The testing organisation should be UKAS Accredited, accredited by an equivalent European Accreditation body which is party to a MLA with UKAS, or any equivalent IAF MLA signatory with a scope which includes the relevant standard(s).

Testing of chloride concentration

5.40 Chloride content of concrete shall be determined on dust samples collected from the test areas.

5.41 Analysis of dust samples to determine chloride content shall be carried out in accordance with BS 1881-124 [Ref 31.N].

5.41.1 The test procedure should concentrate on aspects which can improve accuracy of results as discussed in "Analysis of hardened concrete. A guide to tests, procedures and interpretation of results." CS TR32 [Ref 3.I].

Testing of cement content

5.42 Cement content of concrete shall be determined on dust samples collected from the test areas.

5.43 Analysis for dust samples to determine cement content shall be carried out in accordance with BS 1881-124 [Ref 31.N].

5.43.1 The test procedure should concentrate on aspects which can improve accuracy of results as discussed in "Analysis of hardened concrete. A guide to tests, procedures and interpretation of results." CS TR32 [Ref 3.I].

NOTE The test method in BS 1881-124 [Ref 31.N] only provides reliable results for concretes containing CEM I cement.

Testing of compressive strength

5.44 Testing to determine the in-situ compressive strength of existing concrete shall be carried out in accordance with BS EN 12390-3 [Ref 35.N], BS 6089 [Ref 4.N] and BS EN 13791 [Ref 3.N].

NOTE More information on establishing compressive strength can be found in CS TG2 [Ref 19.I].

5.44.1 Estimation of compressive strength may be carried out using indirect methods such as ultrasonic pulse velocity, rebound hammer and pull-out tests.

Testing of water content and water/cement ratio

5.45 Analysis for original water content shall be carried out in accordance with BS 1881-124 [Ref 31.N].

5.45.1 Estimation of original water to cement ratio should be determined on dust samples collected from the test panels.

NOTE A potential for inaccuracy in calculating results is discussed in CS TR32 [Ref 3.I].

5.45.2 Water content result may be corroborated by examination under a microscope of a polished specimen containing cement particles still unhydrated.

Petrographical testing

5.46 Petrographical testing shall only be used to:

1) identify the extent and/or strength reduction of concrete exposed to temporary high temperature (e.g. following a fire);
2) positively identify the presence of forms of IDC and to confirm sulfate content of concrete; and,
3) confirm cement content where it is crucial the result is accurate.

NOTE See Assessment design and repair of fire-damaged concrete structures, CS TR68 [Ref 4.I].

5.47 Concrete samples shall be prepared for petrographic examination in accordance with BS 1881-211 [Ref 33.N].

NOTE Advice on petrographic examination is provided in the Concrete Society's 'Concrete petrography', CS TR71 [Ref 7.I], the Applied Petrography Group's 'A code of practice for the petrographic examination of concrete' SR2 [Ref 1.I], and ASTM's 'Standard Practice for Petrographic Examination of Hardened Concrete' ASTM C856 [Ref 34.I].

Other examination and testing

5.48 Cored specimens shall be examined for reinforcement, delamination, compaction, presence of voids, honeycombing and cracks and defects reported.

5.49 Density of concrete from cored specimens shall be determined in accordance with BS EN 12390-7 [Ref 36.N].

5.50 Analysis of concrete samples for sulfate content shall be carried out in accordance with BS 1881-124 [Ref 31.N].

Concrete investigation report

5.51 A concrete investigation report shall be prepared where survey and testing has been carried out on a concrete structure.

5.51.1 The contents of the investigation report should be based on the recommendations for Recording and Reporting Test Results in 'Part E. Investigation and Testing' of the Inspection Manual for Highways Structures, Volume 1' IMHS V1 [Ref 22.I].

5.52 Electrical potential readings shall be presented with reference to a CSE reference electrode.

5.52.1 Conversion of readings from the half-cell electrode used to copper/copper sulfate equivalent values may be necessary.

NOTE For conversion of SSC [0.5M KCL] to CSE values, the measured value with respect to SSC [0.5M KCL] is 50mV more negative; i.e. -200mV with respect to SSC [0.5M KCl] converts to -250mV CSE.

5.53 The values of electrical potential by half-cell survey shall be presented in the form of colour-coded equi-potential contour plots.

5.53.1 Contours should be presented at 50mV intervals superimposed on scale plans of the structural elements with numerical values of the readings alongside the contours.

5.53.2 Individual values of electrical potential may also be presented in the report.

5.53.3 Results of chloride analysis may be presented in various ways e.g. profile of chloride concentration against depth, or alternatively a list of chloride concentrations.

5.53.4 The report should identify any locations where readings could not be obtained e.g. half-cell potentials.

Structural assessment

5.54 Where structure deterioration is of particular concern, CS 451 [Ref 27.N] shall be used to determine the need for a structural review and assessment of the whole structure or a structural element.

5.54.1 Where deterioration is extensive, structural integrity or the structure load carrying capacity can be compromised, and the TAA should be informed.

NOTE Small scale deterioration consisting of spalling and delamination is sometimes not a particular concern, but its significance can depend on the original detailing of the structure and its assessed utilisation.

5.54.2 The scope of structural assessment should be agreed with the TAA before commencing work.

5.55 Structural assessment of the deteriorated structure or structural elements shall be carried out in accordance with CS 454 [Ref 2.N] and CS 455 [Ref 37.N].

5.55.1 The assessment should take account of reduction in both reinforcement (corrosion) and concrete sections (during repair) for the current condition, based on testing and inspection results.

5.56 The structural capacity of a deteriorated element shall be checked by assessment at all proposed stages of repair to determine whether it complies with CS 454 [Ref 2.N].

5.56.1 Where a structural element does not comply, weakened elements may require temporary support until the deteriorated element or structure has been strengthened.

5.56.2 The scope of assessment should include critical details like half-joints and deck hinges where they are affected by deterioration.

5.57 Assessed stability of deteriorated structural elements and the structure as a whole shall be recorded as part of a design stage safety risk assessment during planning or design of a repair scheme.

6. Managing a deteriorated concrete structure in service

6.1 Options for mitigating safety risks shall be identified and compared, where a deteriorated structure complying with CS 454 [Ref 2.N] is to be managed in service.

6.1.1 Management in service should be agreed with the TAA.

6.1.2 Proposals for making safe and monitoring defects should be presented in a SMOR.

6.1.3 Options for making safe may include:

1) knocking off loose concrete;
2) coating exposed bars;
3) netting;
4) temporary safety barriers or temporary propping pending later completion of formal concrete repairs.

6.2 Where the load capacity of a deteriorated structure does not comply with CS 454 [Ref 2.N] or the deteriorated structure is otherwise deemed to be substandard, the structure shall be managed in accordance with CS 470 [Ref 16.N].

6.2.1 Where the deteriorated structure is classified as sub-standard, the SMOR should include an interim measures feasibility assessment and possible safety measures to manage and mitigate the hazards.

6.2.2 Short term options for making safe may include:

1) knocking off loose concrete;
2) coating exposed bars;
3) netting;
4) temporary safety barriers;
5) temporary propping;
6) lane closure;
7) downgrading the function or complete closure pending later completion of formal concrete repairs.

6.2.3 Options for the medium to long term may include repair, strengthening or replacement of the structure.

6.3 Where an immediate and unacceptable risk to public safety has been identified, the TAA shall be notified without delay in accordance with CS 470 [Ref 16.N].

6.4 Structures affected by IDC and TSA or severe chemical attack shall be managed in accordance with the requirements in Section 8 or Section 9 of this document.

Monitoring

6.5 Where a structure is deemed to be substandard due to concrete deterioration, the reason for, aim and frequency of monitoring and the trigger levels prompting further intervention shall be stated within the CS 470 [Ref 16.N] interim measures feasibility assessment.

NOTE Guidance about monitoring techniques can be found in the appendices to CS 470 [Ref 16.N].

6.5.1 Monitoring may include instrumentation to determine real time behaviour of the structure.

6.5.2 Monitoring as part of a special inspection to record condition of the element under review may be undertaken more frequently than the general and principal inspection intervals.

NOTE Periodic measurement of length and width of structural cracking, can indicate if movement is still happening or has ceased.

6.5.3 Within the monitoring regime, the following data should also be recorded about the structure where applicable:

1) environmental conditions in which the structure is located;

2) structural form or type;
3) constituent materials and chemical composition;
4) previous repairs; and,
5) level of structural utilisation.

NOTE Data gathered from monitoring can indicate the rate of deterioration.

7. Protect and repair strategy

Diagnosis of the cause of defects

7.1 A schedule of concrete defects shall be prepared and recorded for all deteriorated structural elements.

7.1.1 The reported defects should include a judgement about the underlying cause with supporting evidence.

NOTE 1 Common causes of deterioration in reinforced concrete structures are tabulated in Figure 1 of BS EN 1504-9 [Ref 28.I] and are discussed in 'Diagnosis of deterioration in Concrete Structures - Identification of defects, evaluation and development of remedial action' CS TR54 [Ref 14.I].

NOTE 2 Defects in concrete structures can arise from inadequate design, specification, supervision, execution, materials or management.

NOTE 3 Defects can also occur in service during exposure to environmental and atmospheric conditions causing superficial deterioration of concrete or reinforcement corrosion, and due to physical loading, accidental or intentional damage and stray electric currents.

NOTE 4 Defects can also result from increased internal stresses caused by deterioration mechanisms such as alkali- silica reaction (ASR), delayed ettringite formation (DEF) (see further guidance in Section 8 on IDC) and forces due to restraint of loading.

7.1.2 The schedule of concrete defects should be completed prior to planning and designing a repair solution.

7.1.3 An existing schedule of defects on file for a structure should be revised to include new data obtained during inspection or testing work.

Assessment of corrosion risk

7.2 The risk of corrosion to reinforcement within intact concrete shall be assessed where there is sufficient relevant information.

7.2.1 Where there is insufficient information to undertake an accurate risk assessment, additional testing should be undertaken.

7.2.2 The risk assessment should include a suite of corrosion probability indicators including the following:

1) total free chloride ion concentration in concrete;
2) moisture content of concrete;
3) depth of carbonation of concrete;
4) corrosion potential of reinforcement; and,
5) other relevant indicators for concrete or reinforcement.

7.2.3 The recommendations of the Concrete Society's CS TR60 [Ref 16.I], and the Structural Concrete Alliance's 'Corrosion Testing of Concrete Structures' Technical Note 27 [Ref 12.I] should be followed when interpreting the results of testing.

NOTE Advice on the interpretation of test results can be found in many reference documents including CS TG2 [Ref 19.I], IMHS V1 [Ref 22.I] and IMHS V2 [Ref 23.I], CS TR54 [Ref 14.I] and 'Repair of Concrete in Highway Bridges – A Practical Guide' TRL AG43 [Ref 30.I].

7.2.4 The results of electrical potential mapping together with measurements of associated chloride ion concentration should be used to assess the risk of reinforcement corrosion.

7.2.5 Depth of carbonation may also be used in conjunction with chloride ion concentration and electrical potential to assess the risk of corrosion.

7.2.6 The assessed risk of corrosion should be validated by comparing it against the surveyed condition of the structure.

Corrosion probability indicator - electrical potential mapping

7.3 Values of current and absolute rate of potential change, shall be used to interpret results from half-cell electrical potential testing.

NOTE A local increase in moisture content of the concrete can distort the pattern of electrical potential readings.

7.3.1 Corrosion probability should be assessed with respect to values of electrical potential which have been converted to a CSE reference electrode.

NOTE The relationship between the electrical potential readings obtained by using different electrodes and the standard CSE, Cu/CuSO4 electrode can be found in CS TR60 [Ref 16.I] and CS TR73 [Ref 5.I].

7.3.2 Guidance in ASTM C876 [Ref 27.I] may be used for interpreting results of the electrical potential survey using half-cell testing.

NOTE 1 Listed values are with respect to a CSE base electrode (Cu/CuSO4).

NOTE 2 Potentials numerically less than -200mV with reference to a CSE electrode indicate a low risk of corrosion .

NOTE 3 More advice on interpretation of test results can be found in other reference documents including CS TG2 [Ref 19.I], the Inspection Manual for Highway Structures IMHS V1 [Ref 22.I], IMHS V2 [Ref 23.I], CS TR54 [Ref 14.I] and TRL AG43 [Ref 30.I].

NOTE 4 Where galvanic anodes are embedded in the concrete, or if the concrete is impregnated or coated, there can be a shift in electrical potentials measured by the half-cell survey method.

Corrosion probability indicator - chloride contamination

7.4 The total chloride ion concentration at the depth of the outermost layer of reinforcement shall be used for assessing the risk of corrosion.

7.4.1 Chloride content of concrete with respect to the mass of cement should be used for assessment.

7.5 0.4% chloride ion concentration by mass of cement shall be adopted as the limit below which the risk of reinforcement corrosion is acceptably low.

7.5.1 Where concrete carbonation has penetrated beyond 50% of the nominal cover to the outermost reinforcement, the lower level of chloride ion concentration may be reduced below 0.4%.

7.6 1.0% chloride ion concentration by mass of cement supported by evidence of reinforcement corrosion and concrete delamination shall be adopted as an upper limit above which the risk of reinforcement corrosion is unacceptably high.

7.7 A range of testing data shall be used to determine the acceptability of reinforcement corrosion risk when chloride ion concentrations are between 0.4% and 1.0%.

NOTE More guidance can be found in BS 8500-1 [Ref 8.I] and BS EN 1504-9 [Ref 28.I] and TRL RR78 [Ref 10.I].

Corrosion probability indicator - electrical resistivity

7.8 Interpretation of electrical resistivity measurements shall be in accordance with CS TR60 [Ref 16.I].

NOTE 1 ASTM C1760 [Ref 35.I], AASHTO TP95 [Ref 33.I] and RILEM TC-154 [Ref 38.I] can provide additional information about interpreting electrical resistivity readings.

NOTE 2 Interpretation of measurements of electrical resistivity can be misleading, unless considered in conjunction with a measure of concrete moisture content. The quality/integrity of concrete can also influence the values obtained.

Corrosion probability indicator - carbonation depth

7.9 Where depth of carbonation has penetrated 75% of the concrete cover, the risk of reinforcement corrosion shall be deemed to be high.

Mitigating the risk of reinforcement corrosion

7.10 When the assimilation of corrosion probability indicators suggests the risk of reinforcement corrosion is unacceptably high, the risk shall be mitigated.

7.11 The following are examples where risk of reinforcement corrosion is unacceptably high and shall be used as a guide to promoting a maintenance intervention:

1) 1.0% total chloride concentration by mass of cement supported by evidence of reinforcement corrosion and concrete delamination;
2) total chloride concentration by mass of cement exceeds 0.4%, half-cell potential measurements are more negative than -350mV relative to CSE, and other factors indicate a high risk of corrosion.

NOTE Guidance about the levels of corrosion risk are described in BRE DG 434 [Ref 9.I], BRE DG 444: Part 2 [Ref 11.I] and ASTM C876 [Ref 27.I].

7.11.1 Where total chloride concentration by mass of cement is generally between 0.4% and 1.0%, but half-cell potential readings are generally more positive than -350mV, the risk may be classified as 'medium'.

7.11.2 Maintenance intervention may be deferred if risk and consequence of corrosion is assessed as medium to low, and is justified by evaluation of test results.

7.11.3 Where maintenance intervention is deferred, the deteriorated structure should be monitored at appropriate intervals.

7.12 Repair intervention shall not be undertaken when the risk of reinforcement corrosion is 'acceptably low'.

7.13 A brief summary of the corrosion risk assessment shall be added to the structure file record.

Developing a repair scheme

General

7.14 When a maintenance intervention has been accepted into a major scheme or maintenance works programme, a methodology for repair and protection shall be developed.

NOTE 1 Remaining service life of the structure can be relevant when selecting an appropriate repair methodology.

NOTE 2 Service life is usually assumed to be 120 years from date of construction.

7.14.1 Generic options and factors for repair which are listed in BS EN 1504-9 [Ref 20.N] should be used to develop the repair methodology.

BS EN 1504 Part 9 repair principles and methods

7.15 Table 1 of BS EN 1504-9 [Ref 20.N] shall be used to identify suitable principles and methods for repairing the defective concrete.

7.15.1 Principles and methods for protection and repair of deteriorated concrete structures should be identified and recorded in a schedule for each deteriorated structural element.

7.15.2 Principles and methods permitted or preferred by the OO should be used.

NOTE Methods most likely to be relevant relate to repair principles 3, 4, 5, 7 and 10 from Table 1 of BS EN 1504-9 [Ref 28.I].

7.15.3 More than one BS EN 1504-9 [Ref 28.I] method of protection and/or repair may be selected for a repair works to provide the contractor with a choice.

NOTE Different protection methods can be mutually incompatible.

Dealing with causes of concrete deterioration

7.16 The adequacy and effectiveness of existing structure drainage, deck waterproofing and deck joints shall be reviewed.

NOTE The durability of repairs and continuing durability of the structure can depend on effective exclusion or removal of water.

7.16.1 Surface water should be able to quickly exit the structure through any drainage paths provided.

7.16.2 Existing drainage facilities may require unblocking, cleaning, improvement or renewal.

NOTE Specialised drainage systems have been developed for use in confined areas beneath deck joints.

7.16.3 It may be necessary to correct any deficiencies in deck waterproofing and deck joints by repair or replacement and further guidance can be found in CD 358 [Ref 39.N] and CD 357 [Ref 5.N].

7.17 Significant damage to a deck top surface shall be repaired by reinstating to original levels and profile.

NOTE Deck concrete can be accidentally damaged by a road planer during replacement of defective waterproofing systems.

7.17.1 Repair of the damaged deck may be waived if the following are adequately demonstrated to the TAA:

1) there is no reduction below the required live load capacity of the deck;
2) the reduction in cover to reinforcement is tolerable; and,
3) the proposed waterproofing system can be successfully applied to the damaged surface in accordance with CD 358 [Ref 39.N]

Repair options

General

7.18 Where the risk of reinforcement corrosion is deemed to be unacceptably high, the following repair or protection options, in addition to replacement of delaminated and spalled concrete, shall be appraised and the proposed approach agreed with the TAA, following the value management and whole life cost analysis procedure required by the OO:

1) remove and replace any defective intact concrete containing excessive chloride contamination;
2) incorporate galvanic anodes inside the perimeter of concrete repair patches to protect reinforcement within intact chloride-contaminated concrete adjacent to repair;
3) use cathodic protection system to protect reinforcement within intact but chloride-contaminated concrete in accordance with CD 370 [Ref 6.N];
4) no further treatment to intact but chloride-contaminated concrete, but monitor for future deterioration.

7.18.1 Options in addition to replacement of delaminated or spalled concrete may be proposed either singly, or as permitted combination 1) and 2).

NOTE Galvanic anodes used for option 2) can protect reinforcement within intact chloride-contaminated concrete for a distance up to 0.3m away from the repair area by controlling the incipient anode effect.

7.18.2 Where in-situ survey and laboratory test results indicate extensive and severe deterioration of a structural element has occurred, an economic comparison may be made between the costs of repair and replacing the element concerned.

Structure management options report

7.19 Where significant deterioration of a structure has been identified, and if required by the TAA, a SMOR shall be prepared.

NOTE *Preparation of an SMOR only applies where there is more than one BS EN 1504-9 [Ref 28.I] option for repairing or managing the structure.*

7.19.1 A SMOR should include at least the following:

1) description of structure;
2) structure history including details of any previous repair works;
3) future requirements or use for the structure;
4) summary of inspection, survey and testing results;
5) summary of latest structural assessment of the structure or structural elements;
6) summary of actions taken to mitigate risk associated with deterioration;
7) options considered and relevant factors from BS EN 1504-9 [Ref 28.I];
8) health and safety including preliminary risk assessment and identification of potential hazards;
9) summary of any technical approval requirements;
10) estimated cost of each option;
11) impact of structure management or repairs on road network;
12) recommended management strategy for the structure;
13) any other factors or information.

NOTE 1 *Estimated cost of each option prepared using whole life costing techniques (see Section 4 of this document and CD 355 [Ref 38.N]).*

NOTE 2 *Other factors or information can include a corrosion report (if available).*

7.19.2 Where it is proposed to monitor concrete deterioration rather than promoting a repair scheme, a predicted rate of deterioration, risk of structural instability, and recommended intervention date should be reported in the SMOR.

7.20 The draft SMOR shall be submitted to the TAA for comment and agreement of the option required.

Preliminary design of repairs

7.21 The minimum extent of concrete repairs shall comprise the areas where cover concrete is delaminated and areas where concrete has spalled.

NOTE *Repair of deteriorated concrete and reinforcement can prove difficult where access is restricted. Guidance is given in CS 466 [Ref 26.N] for concrete half-joints and CS 467 [Ref 25.N] for deck hinges.*

7.21.1 Areas of intact but chloride-contaminated concrete where the risk of reinforcement corrosion is high should be treated by either of the following methods:

1) removal of the intact but chloride-contaminated concrete which is assessed as having a high risk of corrosion;
2) protect reinforcement within intact but chloride-contaminated by using galvanic anodes fixed to reinforcement within the repair area.

NOTE *Galvanic anodes attached to reinforcement within a repaired area can protect reinforcement within adjacent concrete up to 0.3m away from the edge of the repair.*

7.21.2 As an alternative, impressed current cathodic protection (ICCP) may be used to protect reinforcement within intact concrete contaminated by chlorides.

NOTE *ICCP can be appropriate where chloride contamination of intact concrete is more extensive.*

7.21.3 Defects in a structure assessed with a high reserve of strength may be repaired to allay third party concerns about delaminated concrete potentially causing harm to the public.

NOTE *Unrepaired defects can sometimes look worse than they are.*

7.21.4 Temporary holding repairs may be undertaken to reduce deterioration, pending design and construction of a full repair scheme at a later date.

NOTE *Durability of holding repairs can be significantly less than a permanent repair.*

Galvanic anodes within repairs to control incipient anode effect

7.22 The risk of corrosion to reinforcement within intact but chloride-contaminated concrete adjacent to delaminated areas shall be assessed and conclusions recorded.

NOTE *Accelerated reinforcement corrosion within chloride contaminated concrete adjacent to a new repair is known as the incipient anode effect.*

7.23 Options for mitigating a high risk of corrosion shall be appraised and conclusions recorded.

7.23.1 The extent and concentration of chloride contamination within intact concrete adjacent to delaminated areas should be measured before proposing galvanic anodes within repairs.

NOTE *Galvanic anodes placed within repair patches can control the incipient anode effect.*

7.24 Galvanic anodes shall not be used in the following situations:

1) where the chloride contamination of intact concrete surrounding proposed repairs is less than 0.4% by mass of cement;
2) where an impressed current cathodic protection system overlaying or underlaying the repair is proposed;
3) within repair patches associated with mechanical, impact or fire damage, unless the adjacent intact concrete is chloride contaminated.

7.25 Design of galvanic anodes within a concrete repair shall be undertaken by an Engineer with the qualifications and experience required by BS EN ISO 15257 [Ref 8.N] .

NOTE *An array of galvanic anodes provided inside the perimeter of repairs is unlikely to meet the requirements of cathodic protection as defined by BS EN ISO 12696 [Ref 7.N], but they can prevent initiation of corrosion to reinforcement immediately adjacent to the repair patches.*

7.25.1 Galvanic anodes fixed to reinforcement should be designed to prevent initiation of corrosion on reinforcement located within intact but contaminated concrete up to 0.30m away from the edge of the repair patch for at least 10 years.

NOTE 1 *Galvanic anodes are electrically connected to existing reinforcement within the repair patch to control the initiation of corrosion in reinforcement located in adjacent chloride-contaminated but intact concrete.*

NOTE 2 *The life of galvanic anodes installed within repairs depends on the resistivity of the parent concrete and new repair concrete. In lower resistivity concrete, anodes provide more current and better protection, but the anode life is reduced.*

7.25.2 A longer design life up to 15 years may be specified with the agreement of the TAA.

7.25.3 The performance of an array of galvanic anodes fixed to reinforcement within a repair patch should not require monitoring by permanent reference electrode.

NOTE 1 *The current supplied by the anodes varies according to demand from the surrounding concrete, but cannot be increased if later measured to insufficient. Lack of concrete delamination for 10 years or longer is definitive confirmation of adequate performance.*

NOTE 2 *Potential for adequate performance of anodes can be inferred by carrying out a corrosion potential (half-cell) survey outside the repair patches before removing defective concrete, and again following repairs including galvanic anodes. Evidence of initial protection is a positive shift in electrical potential.*

Other cathodic protection systems

7.26 Other cathodic protection systems proposed in conjunction with concrete repairs shall be designed in accordance with CD 370 [Ref 6.N].

7.26.1 Where a cathodic protection system overlaying repairs is proposed, existing concrete containing conductive fibres and existing resin-based concrete with a resistivity greater than 100 k Ω.cm should be replaced TRL AG43 [Ref 30.I].

Substandard concrete cover

7.27 Where the existing concrete cover to reinforcement is substandard, a corrosion risk assessment shall be undertaken to decide whether remedial action to restore cover is appropriate.

NOTE Design standards for concrete cover to reinforcement have been updated many times with a gradual increase in recommended cover. See Historical approaches to the design of concrete buildings and structures CS TR70 [Ref 21.I] for more information.

7.27.1 Additional protection to reinforcement may be required where existing concrete cover is less than 20mm thick.

NOTE 1 Where low cover is encountered on a deck beneath waterproofing, the waterproofing can provide additional corrosion protection.

NOTE 2 Where a cathodic protection system consisting of a surface-mounted anode within a cementitious overlay is proposed, sub-standard concrete cover would not need to be increased unless it is less than 15mm.

7.28 Concrete cover shall be restored with a cementitious repair material to at least the minimum requirements recommended by BS 8500-2 [Ref 9.N] for an intended working life of 50 years and an allowance for deviation of zero.

Concrete injection

7.29 The cause of significant cracking in concrete shall be investigated before planning repair options.

NOTE 1 Cracking can indicate structural integrity is adversely affected and/or durability has been reduced.

NOTE 2 Construction of the structure can predate the publication of crack control codes.

NOTE 3 Provision of nominal secondary reinforcement was sometimes insufficient to prevent early thermal cracking.

NOTE 4 More information about causes of cracking can be found in CS TR44 [Ref 43.I].

7.29.1 The investigation should comprise a full survey of cracks (length, width and depth) and where appropriate long term monitoring of crack movement.

NOTE Light grit blasting of the concrete surface prior to inspecting cracks can make them more visible and reveal other smaller defects.

7.30 Cracking caused by concrete delamination following reinforcement corrosion shall be rectified using a conventional concrete repair.

7.30.1 Delaminated concrete has already partially debonded from the base concrete, and concrete injection techniques should not be used for repair.

7.31 Treatment of concrete cracking using concrete injection techniques shall follow the recommendations in Annex A of BS EN 1504-9 [Ref 28.I] and in 'Repair of concrete structures with reference to BS EN 1504' CS TR69 [Ref 31.I].

7.31.1 Concrete injection techniques may be used for treatment of cracks in existing concrete when the cracks are wider than 0.3mm.

NOTE Fine cracks, typically less than 0.3mm, which are not considered to contribute to the durability of concrete and corrosion of reinforcement in accordance with 'The relevance of cracking in concrete to corrosion of reinforcement' CS TR44 [Ref 43.I], can be monitored and sufficient time allowed for self-healing prior to concrete injection being carried out.

7.31.2 Repair of cracked concrete may be achieved by structurally stitching the sides of crack together using a force transmitting material, type F complying with BS EN 1504-5 [Ref 21.N]. or creating a formal movement joint and filling it with ductile material, type D

NOTE *There is a risk of causing or extending cracking because of the applied fluid pressures during injection. Vacuum-assisted injection techniques can lessen the risk.*

7.31.3 Where cracks are widely spaced and approximately parallel, new formal movement joints may be created and filled with ductile material, type D complying with BS EN 1504-5 [Ref 21.N].

7.32 Proposals for sealing cracks which leak water using swelling fitted filling products Type S complying with BS EN 1504-5 [Ref 21.N], and other techniques not covered by BS EN 1504-5 [Ref 21.N] shall be subject to the technical approval of the TAA.

Surface protection systems

7.33 The use of surface protection systems in conjunction with concrete repairs shall be in accordance with CD 369 [Ref 28.N].

7.34 Approval for other surface protection systems in accordance with Principles 1 or 2 of Table 1 in BS EN 1504-9 [Ref 28.I] and used in conjunction with concrete repairs shall be obtained from the TAA or OO before design commences.

7.34.1 Where testing indicates concrete carbonation has penetrated to within 10mm of the reinforcement, and some spalling has occurred, anti-carbonation coatings may be used with concrete repairs if agreed with the TAA.

Technical approval

7.35 Technical approval of the repair proposals shall comply with CG 300 [Ref 29.N].

7.36 Where relaxations to the CG 300 [Ref 29.N] category are proposed, reasons shall be presented to and agreed with the TAA before detailed design commences.

Maintaining structural safety during repair work

7.37 The capacity of structural elements, and global stability of the structure for the temporary conditions shall be assessed during the design process.

7.37.1 The rate and sequence of concrete removal should be planned to ensure structural integrity and continuity of load transference are maintained in the structure and structural elements.

7.37.2 Where deterioration is severe, temporary propping of structural elements may be necessary while repairs are carried out.

NOTE *The risk to structural stability can be more serious where compressive stress applied to a structural member in service is close to the allowable limit. e.g. small diameter slender columns.*

7.38 Safety risks associated with construction of repairs shall be identified and reviewed during the design process.

NOTE 1 *Risks can include structural collapse, debris, construction activities associated with maintenance and repair or from implementing traffic management while carrying out repair works.*

NOTE 2 *Those affected can include users of the road network, and those undertaking the work, including supervisors, inspectors and visitors.*

7.39 Removal of defective concrete in structural elements identified as high risk e.g. pre-stressed or post-tensioned elements or half-joints, shall be assessed for each of the construction stages during the repair works.

NOTE 1 *The strength of pre-stressed or post-tensioned concrete elements can be significantly weakened by removal of defective concrete as the majority of concrete is in compression.*

NOTE 2 *The strength of half-joints can be adversely affected by either deterioration or the method of repair undertaken, e.g. repairs close to strand anchorages can initiate loss of pre-stress, or significant loss of half-joint strength through debonding or cropping of critical bars within half-joints such as longitudinal soffit bars and diagonal bars.*

7.39.1 Defective concrete in pre-stressed or post-tensioned elements should generally be removed by high pressure water jetting to avoid mechanical damage.

NOTE *Removal of concrete using mechanical breakers can cause vibration in the remaining concrete and result in micro-cracking. Reinforcement bond can be reduced by micro cracking.*

Materials, products and systems for repairing concrete

7.40 Proprietary concrete repair products intended for repairing concrete shall comply with BS EN 1504 [Ref 22.N].

NOTE 1 *Proprietary repair products are manufactured in accordance with relevant parts of the standard e.g. Parts 3, 5, 6 and 7 and are certified by CE marking.*

NOTE 2 *A proprietary cement-based repair concrete or mortar is often used for restoring small to medium sized areas to a depth of up to 150mm.*

7.40.1 Designed or designated repair concrete complying with MCHW Series 1700 [Ref 18.N] may be used instead of proprietary products, where large volume replacement of concrete is necessary.

NOTE *Large volume repairs can include replacement of concrete in parapet supporting elements or a full depth of concrete deck embedding a large transverse mechanical movement joint.*

7.41 New steel reinforcing bars shall be weldable carbon steel complying with MCHW Series 1700 [Ref 18.N] and have a similar or higher grade of steel to existing bars.

NOTE *Existing bars are generally made of mild or high tensile carbon steel.*

7.41.1 New stainless steel bars complying with MCHW Series 1700 [Ref 18.N] may be used where the risk of bi-metallic corrosion is low or can be mitigated.

NOTE 1 *The difference in electrical potential between carbon steel and stainless steel can result in accelerated corrosion of carbon steel where the existing concrete contains high levels of chloride or low alkalinity.*

NOTE 2 *Proprietary construction products are generally used for anchoring reinforcing bars or dowels and for coating exposed reinforcement.*

7.42 Steel mechanical couplers shall comply with MCHW Series 1700 [Ref 18.N] and be an appropriate diameter and similar material to the existing reinforcement.

7.42.1 Mechanical couplers used in fatigue sensitive locations for splicing new bars onto existing reinforcement should be rated fatigue Class D.

NOTE *Certificates confirming coupler fatigue class rating can be found on the websites of UK CARES CARES [Ref 47.I] or The British Board of Agrément (BBA) British Board of Agrement [Ref 45.I] .*

7.43 Galvanic anodes complying with MCHW Series 5700 [Ref 17.N] shall be used to control the incipient anode effect.

Detailed design of repairs

7.44 Concrete repairs shall be detailed to ensure the repair is and continues to be an integrated part of the repaired structure.

NOTE 1 *Placement of repair concrete around reinforcement restores an alkaline environment, and cleaning the existing reinforcement (e.g. wet or dry grit blasting) removes contaminants so there is no longer a risk of reinforcement corrosion.*

NOTE 2 *Structural repairs are secured in place mechanically by encompassing the reinforcement, and by adhesion of the repair material onto a roughened substrate.*

7.45 Repairs shall be structural when located above public highways and other areas used by the public.

NOTE *A well executed structural repair can minimise the risk of premature failure due to debonding.*

7.46 The required depth of breakout behind reinforcing bars shall be clearly indicated in contract documentation.

NOTE *Specifying a dimension of breakout behind bars allows the contractor to choose a repair product with an appropriate aggregate size.*

7.46.1 Concrete removal should be detailed to extend at least 25mm behind the outermost reinforcement, so the repair achieves a good mechanical key.

7.46.2 The depth of removal behind the outer main reinforcement may be reduced to an absolute minimum of 15mm where there is more than one level of reinforcement, provided the removal depth is not less than 2.5 times the maximum aggregate size of replacement concrete.

NOTE *Information and guidance about techniques of removing defective concrete (e.g. high pressure water jetting, mechanical breakout) can be found on the website of the Structural Concrete Alliance SCA Website [Ref 46.I].*

7.46.3 Where impracticable for a repair to encompass the reinforcement, partially-structural repairs may be used with agreement from the TAA by bonding small diameter stainless dowels into the existing concrete.

NOTE 1 *The method can be suitable for minimising removal of concrete for repair of pre-stressed or post-tensioned concrete beams.*

NOTE 2 *Durability of superficial structural repairs can be affected by freeze thaw cycles unless additional reinforcement bond is provided e.g. stainless steel dowels with mesh reinforcement.*

NOTE 3 *A series of unconnected steel dowels can be incompatible with a CP system if used.*

7.47 Concrete repairs shall be detailed to promote durability.

7.47.1 Elastic strain characteristics of new repairs should be compatible with the existing concrete.

NOTE 1 *It is important the elastic modulus of a repair is similar to the estimated short-term elastic modulus of the parent concrete to minimise shear stresses at the interface and improve repair durability.*

NOTE 2 *Compatibility of elastic modulus can be particularly relevant for structural members predominantly subjected to compressive forces in service e.g. columns, and locations subject to frequent cycles of traffic loading e.g. decks.*

7.47.2 Where the upper surface of a concrete element is being repaired, the final surface should be sloped to shed water if feasible.

7.47.3 Where repair areas are large, the risk of early thermal cracking should be assessed and mitigated in accordance with 'Early-age thermal crack control in concrete' CIRIA C660 [Ref 15.I].

NOTE *Where only nominal secondary reinforcement was provided in the original design, It can be necessary to provide additional reinforcement to control cracking of the repair concrete.*

7.47.4 Staged construction of large area repairs may be required to avoid shrinkage or thermal cracking.

7.48 Concrete repair patches shall be detailed to achieve a neat appearance.

7.48.1 The perimeter of repair patches should be shaped as a square, rectangular or a rectilinear polygon.

7.49 Reinforcement which is heavily-corroded reinforcement shall be replaced or supplemented.

7.49.1 New reinforcing bars may be spliced or coupled onto existing corroded bars to restore section strength.

NOTE 1 *Lapping and mechanical coupling of new bars is permitted by the MCHW (see MCHW Series 1700 [Ref 18.N].*

NOTE 2 *Welding of new bars is permitted in certain circumstances by the MCHW (see MCHW Series 1700 [Ref 18.N]).*

7.50 Where welding of replacement bars is proposed within the repaired concrete section, a fatigue verification of the welds shall be carried out in accordance with BS EN 1992-1-1 [Ref 10.N].

NOTE 1 Butt welded reinforcement can be acceptable following a fatigue verification.

NOTE 2 Where a reinforced concrete deck is heavily traffic, the fatigue loading of a proposed welded joint in reinforcement can exceed permitted limits.

7.51 The compressive strength of new repairs shall be similar to the designed or tested strength of the existing concrete.

NOTE 1 Strength classes R1 to R4 for concrete repair products are explained in BS EN 1504-3 [Ref 23.N].

NOTE 2 The highest strength category in BS EN 1504-3 [Ref 23.N] is R4 which requires a 28-day characteristic strength of at least 45 MPa.

7.52 The execution class for repair work shall be specified in accordance with BS EN 13670 [Ref 11.N].

NOTE 1 Information and guidance about flowable, hand-applied and sprayed materials and the construction techniques for applying them can be found on the website of the Structural Concrete Alliance SCA Website [Ref 46.I].

NOTE 2 The Notes for Guidance on the Specification for Highway Works MCHW NG [Ref 26.I] provides advice about information required in contract-specific appendices for concrete repair contracts.

7.53 The curing class for repair work shall be specified in accordance with BS EN 13670 [Ref 11.N].

NOTE 1 Proper curing of repaired concrete is very important for development of concrete strength, bond and to avoid early thermal cracking.

NOTE 2 Advice about curing can be found in 'Good Concrete Guide. Concrete practice - guidance on the practical aspects of concreting' CS 164 [Ref 17.I].

Non-structural (minor) repairs

7.54 Non-structural repairs shall only be used where the assessed strength of the concrete section is acceptable when the structural contribution of the repair is ignored.

7.54.1 Non-structural repairs should be limited to areas equal to or less than 0.1m^2 with an average depth of less than 10mm.

7.55 Non-structural repairs shall not be located on soffits or vertical surfaces above highways, public rights of way and other areas which are or can be used by the public or workforce.

NOTE 1 Non-structural repairs are secured in place only by adhesion of the repair material onto a roughened substrate.

NOTE 2 There is a risk of injury to the public or damage to property if the repair falls out.

Records

7.56 A record of repairs undertaken on a structure shall be included in the structure file on the management information system in accordance with CG 302 [Ref 1.N].

8. Internal deterioration of concrete

8.1 Characteristic signs of internal deterioration of concrete (IDC) in a structure shall be investigated.

NOTE 1 IDC in this context means degradation of concrete within the interior of a structural concrete element which is potentially or actually caused by forms of alkali aggregate reaction (AAR) or delayed ettringite formation (DEF).

NOTE 2 Typical signs can include map-pattern cracking, surface deposits and there can be leakage through surface cracks possibly suggesting crack pathways through the concrete core.

8.1.1 Relevant information may be obtained by visual inspection, from archive records of construction suggesting there are susceptible aggregates combined with high alkali in concrete, or by rapid expansion testing in accordance with BS 812-123 [Ref 30.N].

8.2 Where IDC is suspected, a special inspection including testing shall be undertaken to confirm its presence.

NOTE Information and guidance about AAR can be found in Structural effects of alkali-silica reaction Technical guidance on the appraisal of existing structures – July 1992 ISE ASR 1992 [Ref 37.I] and Structural effects of alkali-silica reaction Technical guidance on the appraisal of existing structures – Addendum April 2010. ISE ASR 2010 [Ref 36.I]

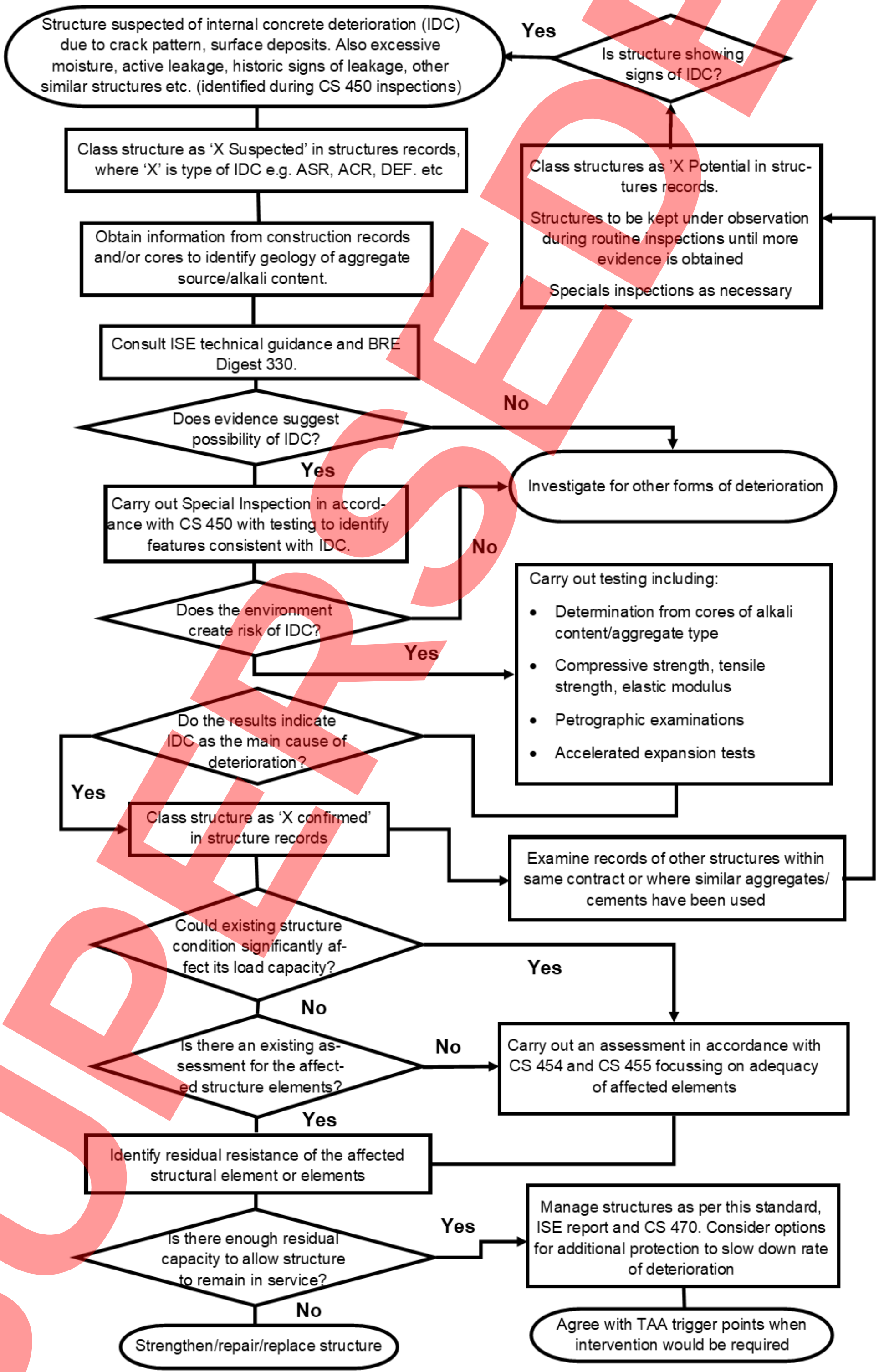
8.2.1 Delamination and excessive cracking located in areas of high bond or shear stress should be noted.

NOTE Delamination of cover concrete is rarely caused by IDC and more frequently due to corrosion of reinforcement.

8.2.2 The diagnosis of IDC should be confirmed as early as possible, to keep deteriorated concrete structures in service.

8.3 The flowchart in Figure 8.3 shall be used as a guide to the steps required for diagnosing IDC and the procedure to be followed after diagnosis.

Figure 8.3 Investigative steps and the interaction with assessment for the deteriorated structure suspected of being affected by IDC



8.4 Where IDC is confirmed, the progression of deterioration shall be monitored and recorded.

8.4.1 Overall structural dimensions and rate of expansion should be measured.

8.4.2 Individual crack widths should also be measured especially at structurally sensitive locations.

8.4.3 The location of existing reinforcement should be surveyed, marked and compared to the pattern of cracking.

8.5 Where IDC cannot be discounted at this stage of investigation, the affected structure shall be categorised as 'suspected' in accordance with 'Alkali–silica Reaction in Concrete' BRE DG 330 [Ref 2.I].

8.6 Concrete from the deteriorated zones shall be sampled and tested to confirm whether IDC is present.

NOTE Cement content, type of aggregates, sulfate, carbonate and alkali content can be useful information for confirming the presence of IDC.

8.6.1 Concrete cores should be removed from affected areas and tested to accurately determine current strength properties.

NOTE 1 Tested values of concrete strength can be variable because of the cracking.

NOTE 2 Testing a small number of samples on a single occasion is not recommended, as results can be ambiguous.

8.6.2 Samples should be removed from affected areas during at least two years for improved accuracy.

NOTE Final confirmation of a reduction in the assessed capacity can therefore be delayed.

8.6.3 Petrographical techniques may be used for examination of concrete cores to confirm the presence of IDC.

8.6.4 Petrographical examination of core samples should be carried out in accordance with BS 1881-211 [Ref 33.N] and 'Concrete petrography' CS TR71 [Ref 7.I].

NOTE The microscopic examination can provide evidence to help estimation of future deterioration as part of a long-term structure management plan.

8.7 Sampling and testing for the presence of IDC and reporting shall be undertaken by a specialist testing laboratory accredited by UKAS for the work required.

8.7.1 IDC should only be declared as the predominant cause of deterioration when other explanations have been ruled out.

8.8 For structures confirmed as suffering from IDC, an expansion index shall be estimated in accordance with IStructE guidelines ISE ASR 1992 [Ref 37.I] and ISE ASR 2010 [Ref 36.I].

NOTE A mild amount of IDC is present where there is an estimated 0.7mm/m or less of free expansion.

8.8.1 Structures with an expansion index rated 'very severe' may require interim measures such as limited strengthening or load-restrictions or complete replacement.

8.9 Structures confirmed with IDC shall be re-inspected and assessed regularly to monitor deterioration as part of a long term programme of structure management.

NOTE 1 Concrete structures built before ASR/AAR was identified and understood and new codes were published can still be at risk of deterioration.

NOTE 2 There is evidence that progress of IDC does not cease after 10 - 15 years (see information in Appendix A).

8.9.1 The frequency of repeating special inspections and assessment should be in accordance with Table A.1 in Appendix A.

NOTE 1 IDC damage and cracking are progressive and material properties can deteriorate with time.

NOTE 2 Structures with mild amounts of IDC, i.e. with an estimated 0.7mm/m or less of free expansion, do not require assessment.

8.10 The current strength of structural elements confirmed with IDC shall be assessed in accordance with CS 455 [Ref 37.N].

8.10.1 The sensitivity of a structure to predicted further deterioration should be included as part of the assessment (see ISE ASR 1992 [Ref 37.I] and ISE ASR 2010 [Ref 36.I]).

8.11 Wherever possible, ground water and other sources of water shall be excluded from contact with parts of structures suffering from confirmed IDC.

8.11.1 Options for preventing water from getting to vulnerable areas including methods which comply with Principles 1, 2 or 4 in Table 1 of BS EN 1504-9 [Ref 28.I] may be proposed for consideration by the TAA.

8.11.2 Options should be carefully chosen to avoid creating hidden defects.

NOTE *Where expansion is severe and large fissures have occurred, the provision of a ventilated cladding can provide a more sheltered environment.*

9. Thaumasite sulfate attack (TSA)

9.1 The risk of TSA occurring on vulnerable structural elements shall be assessed.

NOTE 1 Where a risk assessment report already exists on file, there is no need for further risk assessment unless circumstances have changed.

NOTE 2 Risk assessment was carried out for structures on most UK strategic roads in the 1990s, and a limited programme of further investigation was undertaken on structures located in the south west of England.

NOTE 3 Structural elements below ground can be vulnerable where the chemical constituents of foundation backfill care known to trigger TSA.

NOTE 4 High concentration of sulfate in backfill soil and the availability of water are predominant factors when considering the risk of TSA.

9.1.1 Cores may be taken from parts of substructure elements above ground to determine the aggregate and cement types present in the concrete.

NOTE 1 Sulfates in groundwater react with carbonates in the concrete aggregate and calcium silicate hydrates in the cement to form thaumasite. More information can be found in the Thaumasite Expert Group Report DETR 1999 [Ref 44.I] and BRE SD-1 2005 [Ref 32.I].

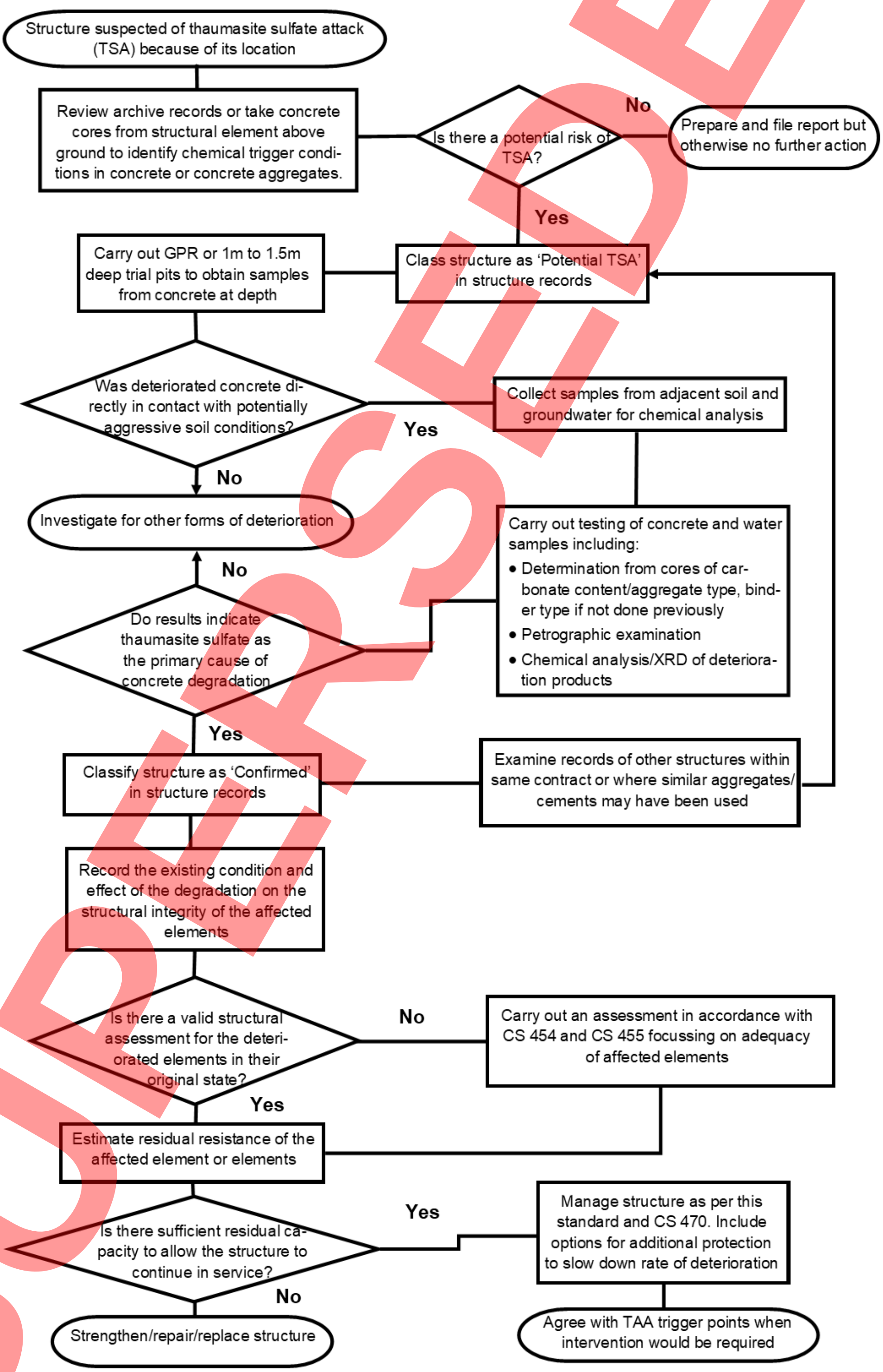
NOTE 2 TSA can occur in structures located in areas where the local geology does not include vulnerable aggregates if transported there and used in concrete mixes.

NOTE 3 Sulfate-resisting Portland cement (SRPC) concretes can also be vulnerable to TSA.

9.1.2 Results and conclusions of the risk assessment should be recorded on file.

9.2 The flowchart in Figure 9.2 shall be used as a guide to the steps necessary to confirm the presence of TSA, assess the effect of damage and commence the process of damage repair.

Figure 9.2 Investigative steps for confirming the presence of TSA-affected structures



9.3 Where a significant risk of TSA is agreed with the TAA, the structure shall be classified as provisionally sub-standard and managed in accordance with CS 470 [Ref 16.N].

9.3.1 Management may include monitoring the structure for signs of distress, and at-risk structural elements below ground can be exposed for inspection, sampling and testing where economical.

NOTE *TSA can cause a reduction in the binding ability of the cement matrix resulting in loss of strength and structural integrity of the concrete.*

9.3.2 The soil surrounding foundations of structures classed at risk may be investigated for evidence of aggressive chemical compounds and the presence of groundwater.

NOTE 1 *Lower Lias clays and made ground containing unburnt or burnt (red) colliery shales are significant sources of sulfate and can be present as a backfill material around bridge foundations.*

NOTE 2 *More information can be found in Thaumasite Expert Group Report: Review after three years experience DCLG Thaumasite [Ref 41.I].*

9.4 Sampling of concrete and testing shall be undertaken to accurately determine properties of concrete within the deteriorated zone and confirm whether TSA is present.

NOTE *The most accurate method of determining thaumasite sulfates by mass of cement is through petrographic examination.*

9.5 The severity of concrete deterioration and risk to structural stability shall be assessed where TSA is confirmed.

9.5.1 The severity of attack should be judged according to the depth of concrete damage (concrete strength or integrity significantly reduced) and potential for further damage due to high concentration of sulfate in the surrounding backfill.

NOTE 1 *Deterioration to a depth of up to 50 mm after 30 years in service has been uncovered in bridge sub-structures, constructed from concrete containing Portland cement and carbonate aggregate and subsequently backfilled using reworked pyritic clay.*

NOTE 2 *Existing reinforcement can suffer from a combination of general corrosion and deep pitting if TSA and chloride ion attack are both present.*

NOTE 3 *The extent and severity of reinforcement corrosion in buried concrete affected by TSA can be low when there is a limited supply of oxygen, lack of carbonation and chlorides are bound within the cementitious matrix of concrete.*

9.6 Parts of concrete structures confirmed as being affected by TSA shall be repaired in accordance with the principles and methods in BS EN 1504-9 [Ref 28.I] and following additional requirements:

1) further investigation to establish the full extent of TSA in the concrete;
2) removal of affected concrete until the remaining concrete is:
   a) strong enough and free from cracking;
   b) free from visible signs of sulfate products;
   c) has sulfate content of 5% SO_4^{2-} or less by mass of cement.

NOTE *Current repair protocols for concrete affected by TSA can be found in the Thaumasite Expert Group Report DETR 1999 [Ref 44.I].*

9.6.1 Water should be excluded from parts of the structure suffering from TSA by limiting groundwater movements, and diverting drains away from affected foundations.

9.6.2 Bituminous coating of repaired and unaffected existing concrete, free-draining backfill and new positive drainage may also be used to exclude water.

NOTE *Bituminous coatings can have a limited life.*

10. Normative references

The following documents, in whole or in part, are normative references for this document and are indispensable for its application. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

Ref 1.N	Highways England. CG 302, 'As-built, operational and maintenance records for highway structures'
Ref 2.N	Highways England. CS 454, 'Assessment of highway bridges and structures'
Ref 3.N	BSI. BS EN 13791, 'Assessment of in-situ compressive strength in structures and pre-cast concrete components'
Ref 4.N	BSI. BS 6089, 'Assessment of in-situ compressive strength in structures and precast concrete components. Complementary guidance to that given in BS EN 13791'
Ref 5.N	Highways England. CD 357, 'Bridge expansion joints'
Ref 6.N	Highways England. CD 370, 'Cathodic protection for use in reinforced concrete structures.'
Ref 7.N	BSI. BS EN ISO 12696, 'Cathodic Protection of Steel in Concrete'
Ref 8.N	BS EN ISO 15257, 'Cathodic protection – Competence levels of cathodic protection persons – Basis for certification scheme'
Ref 9.N	BSI. BS 8500-2, 'Concrete. Complementary British Standard to BS EN 206. Specification for constituent materials and concrete.'
Ref 10.N	BSI. BS EN 1992-1-1, 'Eurocode 2: Design of concrete structures. General rules and rules for buildings'
Ref 11.N	BSI. BS EN 13670, 'Execution of concrete structures'
Ref 12.N	Highways England. CD 373, 'Impregnation of reinforced and prestressed concrete highway structures using hydrophobic pore-lining impregnants'
Ref 13.N	Highways England. CS 450, 'Inspection of highway structures'
Ref 14.N	Highways England. GG 101, 'Introduction to the Design Manual for Roads and Bridges'
Ref 15.N	Highways England. CS 465, 'Management of post-tensioned concrete bridges'
Ref 16.N	Highways England. CS 470, 'Management of sub-standard highway structures'
Ref 17.N	Highways England. MCHW Series 5700, 'Manual of Contract Documents for Highway Works Volume 1 Specification for Highway Works - Series 5700 Concrete Repairs'
Ref 18.N	Highways England. MCHW Series 1700, 'Manual of Contract Documents for Highway Works, Volume 1 Specification for Highway Works - Series 1700 Structural Concrete'
Ref 19.N	Highways England. CS 464, 'Non-destructive testing of highways structures'
Ref 20.N	BSI. BS EN 1504-9, 'Products and systems for the protection and repair of concrete structures - Definitions, requirements, quality control and evaluation of conformity. Part 9: General principles for use of products and systems'
Ref 21.N	BSI. BS EN 1504-5, 'Products and systems for the protection and repair of concrete structures. Definitions, requirements, quality control and evaluation of conformity. Concrete injection'
Ref 22.N	BSI. BS EN 1504, 'Products and systems for the protection and repair of concrete structures. Definitions, requirements, quality control and evaluation of conformity. General principles for use of products and systems '

Ref 23.N	BSI. BS EN 1504-3, 'Products and systems for the protection and repair of concrete structures. Definitions, requirements, quality control and evaluation of conformity. Structural and non-structural repair.'
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Ref 25.N	Highways England. CS 467, 'Risk management and structural assessment of concrete deck hinge structures'
Ref 26.N	Highways England. CS 466, 'Risk management and structural assessment of concrete half-joint deck structures'
Ref 27.N	Highways England. CS 451, 'Structural review and assessment of highway structures'
Ref 28.N	Highways England. CD 369, 'Surface protection for concrete highway structures'
Ref 29.N	Highways England. CG 300, 'Technical approval of highway structures'
Ref 30.N	BSI. BS 812-123, 'Testing aggregates. Method for determination of alkali-silica reactivity. Concrete prism method.'
Ref 31.N	BSI. BS 1881-124, 'Testing Concrete - Part 124: Methods for Analysis of Hardened Concrete '
Ref 32.N	BSI. BS EN 12504-1, 'Testing concrete in structures. Cored specimens. Taking, examining and testing in compression.'
Ref 33.N	BSI. BS 1881-211, 'Testing concrete. Procedure and terminology for the petrographic examination of hardened concrete'
Ref 34.N	BSI. BS 1881-204, 'Testing concrete. Recommendations on the use of electromagnetic covermeters'
Ref 35.N	BSI. BS EN 12390-3, 'Testing hardened concrete. Compressive strength of test specimens'
Ref 36.N	BSI. BS EN 12390-7, 'Testing hardened concrete. Density of hardened concrete.'
Ref 37.N	Highways England. CS 455, 'The assessment of concrete highway bridges and structures'
Ref 38.N	Highways England. CD 355, 'The design for highway structures'
Ref 39.N	Highways England. CD 358, 'Waterproofing and surfacing of concrete bridge decks'

11. Informative references

The following documents are informative references for this document and provide supporting information.

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Ref 3.I	Concrete Society. CS TR32, 'Analysis of hardened concrete: A guide to tests, procedures and interpretation of results'
Ref 4.I	Concrete Society. CS TR68, 'Assessment, design and repair of fire-damaged concrete structures'
Ref 5.I	Concrete Society. CS TR73, 'Cathodic protection of steel in concrete'
Ref 6.I	BSI. BS EN 12696, 'Cathodic protection of steel in concrete'
Ref 7.I	Concrete Society. CS TR71, 'Concrete petrography'
Ref 8.I	BSI. BS 8500-1, 'Concrete. Complementary British Standard to BS EN 206. Method of specifying and guidance for the specifier.'
Ref 9.I	BRE. Hollinshead, K. BRE DG 434, 'Corrosion of reinforcement in concrete: electrochemical monitoring'
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Ref 11.I	BRE. BRE DG 444: Part 2, 'Corrosion of steel in concrete - investigation and assessment'
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Ref 13.I	BRE (Archived). Roberts, MH. BRE IP 21/86, 'Determination of the chloride and cement contents of hardened concrete'
Ref 14.I	Concrete Society. CS TR54, 'Diagnosis of deterioration in Concrete Structures - Identification of defects, evaluation and development of remedial action'
Ref 15.I	Construction Industry Research and Information Association. CIRIA C660, 'Early-age thermal crack control in concrete'
Ref 16.I	Concrete Society. CS TR60, 'Electrochemical Tests for Reinforcement Corrosion'
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Ref 21.I	Concrete Society. CS TR70, 'Historical approaches to the design of concrete buildings and structures'
Ref 22.I	TSO. IMHS V1, 'Inspection Manual for Highway Structures. Volume 1: Reference Manual'

Ref 23.I	TSO. IMHS V2, 'Inspection Manual for Highway Structures. Volume 2: Inspector's Handbook'
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Ref 25.I	Department for Transport. UK Roads Liaison Group. MHS CoP, 'Management of Highway Structures - Code of Practice'
Ref 26.I	Highways England. MCHW NG, 'Manual of Contract Documents for Highway Works Volume 2: Notes for Guidance on the Specification for Highway Works'
Ref 27.I	ASTM. Subcommittee GO1.14. ASTM C876, 'Method for Half-Cell Potentials of Uncoated Reinforcing Steel in Concrete'
Ref 28.I	BSI. BS EN 1504-9, 'Products and systems for the protection and repair of concrete structures. Definitions, requirements, quality control and evaluation of conformity. General principles for use of products and systems'
Ref 29.I	2011/305/EU, 'Regulation (EU) No 305/2011 of the European Parliament and of the Council of 9 March 2011 laying down harmonised conditions for the marketing of construction products and repealing Council Directives 89/106/EEC'
Ref 30.I	TRL. Pearson S & Patel RG. TRL AG43, 'Repair of Concrete in Highway Bridges – A Practical Guide'
Ref 31.I	The Concrete Society. CS TR69, 'Repair of concrete structures with reference to BS EN 1504'
Ref 32.I	BRE. BRE SD-1 2005, 'Special Digest 1:2005, Third edition, Concrete in aggressive ground.'
Ref 33.I	American Association of State Highway and Transportation Officials. AASHTO TP95, 'Standard Method of Test for Surface Resistivity Indication of Concrete's Ability to Resist Chloride Ion Penetration'
Ref 34.I	American Society for Testing and Materials. ASTM C856, 'Standard Practice for Petrographic Examination of Hardened Concrete'
Ref 35.I	American Society for Testing and Materials. ASTM C1760, 'Standard Test Method for Bulk Electrical Conductivity of Hardened Concrete'
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Ref 37.I	The Institution of Structural Engineers. ISE ASR 1992, 'Structural effects of alkali-silica reaction Technical guidance on the appraisal of existing structures – July 1992'
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Ref 39.I	BSI. BS 1881-121, 'Testing concrete. Method for determination of static modulus of elasticity in compression'
Ref 40.I	BSI. BS EN 12390-6, 'Testing hardened concrete. Tensile splitting strength of test specimens'
Ref 41.I	DCLG. DCLG Thaumasite, 'Thaumasite Expert Group Report: Review after three years experience'

Ref 42.I	Transport Research Laboratory. TRL AG9, 'The Half-Cell Potential Method of Locating Corroding Reinforcement in Concrete Structures'
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Ref 44.I	Department of the Environment, Transport and the Regions. DETR 1999, 'The thaumasite form of sulfate attack : risks, diagnosis, remedial works and guidance on new construction report of the Thaumasite Expert Group, January 1999'
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Appendix A. Frequency of re-inspection for structures confirmed with IDC

A1 Introduction

IDC is caused by various chemical changes resulting in expansion of the aggregate, which, ultimately results in cracking of the hardened concrete. Causes of IDC include alkali silica reaction (ASR) now more commonly termed alkali aggregate reaction (AAR) and delayed ettringite formation (DEF).

BA 52/94 (now withdrawn) provided advice for the assessment and inspection of concrete highway bridges and other structures which were affected by alkali silica reaction. The advice was based on contemporary research e.g. Institution of Structural Engineers (IStructE) publication Structural Effects of Alkali Silica Reaction; Technical Guidance on the Appraisal of Existing Structures 1992 ISE ASR 1992 [Ref 37.I].

Later research, IStructE Addendum 2010 ISE ASR 2010 [Ref 36.I] countermands the original BA 52/94 recommendations and clearly shows how different material properties of concrete are affected to a different degree by the aggregate expansion, e.g. for 2.5 mm/m of element expansion, typical 28-day concrete strengths are affected as follows. Compressive strength values show a 20% reduction, tensile splitting values show a 45% reduction, whilst elastic modulus can be reduced by up to 50%. Consequently, the theoretical strength equations that use these material parameters are no longer applicable. These known variations in material properties due to IDC need to be considered in detail as they will have a direct impact on the behaviour of concrete elements and their assessed capacity.

This change in recommendations i.e. that the internal cracking of the concrete caused by forms of IDC is progressive and the concrete material properties can deteriorate over time, now requires re-assessment and reappraisal of structures known to be affected by IDC. This approach is also confirmed by CS 454 [Ref 2.N].

There is still a need for continuing inspection and assessment of existing concrete structures which are subject to ASR or other deterioration mechanisms. This supporting document provides guidance on inspection and testing to be adopted for the monitoring of structures affected by IDC.

A2 Inspection and assessment of concrete suffering from IDC

As it is now known that an initial inspection and assessment of structures with suspected IDC is no longer adequate, the current condition of the concrete should be determined at appropriate time intervals.

Inspection and assessment of structures affected by IDC to be undertaken in two phases.

Phase one:

Extraction of cores from affected areas of the structure to obtain actual material properties prior to assessment of the structural capacity of the elements in question. See CS 455 [Ref 37.N] for advice about the assessment method.

Cores described in BS 1881-121 [Ref 39.I] are extracted using the method in BS EN 12504-1 [Ref 32.N], then tested for:

1) compressive strength using the method in BS EN 12390-3 [Ref 35.N];
2) tensile splitting strength using the method in BS EN 12390-6 [Ref 40.I];
3) elastic modulus using the method in BS 1881-121 [Ref 39.I].

In addition to the above testing of material properties, petrographic analysis is to be performed on samples from cores extracted from the structure in accordance with APG SR2 [Ref 1.I]. The petrographic analysis reveals the concrete composition as well as the degradation mechanism and enables prediction of concrete strength.

Phase two:

Analysis of testing results and assessment - the method is described in CS 455 [Ref 37.N]. The sensitivity/reactivity of the structure is derived from the petrographic analysis in conjunction with the worst credible strength results determined from core testing.

Note: For the determination of the worst credible strength, the number of test specimens selected will be in accordance with BS EN 13791 [Ref 3.N], BS 6089 [Ref 4.N] and CS 455 [Ref 37.N].

Table A.1 recommends time intervals for repeating extraction of concrete cores from areas of the concrete affected by IDC, and subsequent testing. Petrographic observations would be used to determine a reliable degradation factor (F_{DC}), to be applied to worst credible compressive strengths determined by testing in support of a structural assessment. (Source of data SR2 [Ref 1.I]).

Table A.1 Degradation grade, degradation factor, retest interval

Degradation grade	Description from petrographic examination	Compressive strength degradation factor (F_{DC})	Retest interval (yrs)[1, 2, 3]
1	Normal homogenous concrete with few micro cracks. Void content in keeping with the amount of paste. Paste structure in keeping with water/cement ratio. Paste abundance in keeping with water/cement ratio.	1	N/A [1]
2	Slight deterioration, possibly through slightly excess voidage, excess micro-cracking, uneven paste composition, low levels of alkali aggregate reaction, drying shrinkage, low temperature curing possibly slightly lean mixture.	1	N/A [1]
3	Moderately low deterioration, possibly with enhanced voidage, micro-cracking frequency fairly high, excessive past porosity, evidence of leaching or other forms of secondary alteration, possible lean mixture.	0.95	6 [2]
4	Moderate deterioration, possibly with evident macro cracking or fine cracking, enhanced voidage, high frequencies of micro cracking or fine cracks, evidence of significant leaching or other forms of secondary alteration, evidence of ettringite in cracks and voids, evidence of significant alkali aggregate reaction with gel in cracks.	0.90	6 [2]
5	Moderate deterioration, possibly with much fine cracking and some macro cracking, high frequency of micro cracks, very high excess voidage, evidence of paste re-crystallization, excessive porosity, carbonation highly penetrative, evidence of significant alkali aggregate reaction in some abundance.	0.75	4 [2]
6	As for 5 but with enhanced level of deterioration and with concrete remaining intact.	0.60	4 [2]

Table A.1 Degradation grade, degradation factor, retest interval (continued)

7	Concrete shows deterioration and may be partly decomposed or friable. May be difficult to cut and polish.	0.50	2 [2]
8	As 7, but enhanced friability, and tending to break into fragments. Loose aggregate particles, honeycombed.	0.30	2 [2]
9	As 8, but enhanced deterioration. Much cracking and fragmentation.	0.20	N/A [3]
10	All cementitious value, coherence and strength is lost.	0.00	N/A [3]

Note 1: For grades 1 & 2 the re-test intervals have been stated as 'N/A' because the extent of concrete degradation is only slight and future significant degradation is not expected unless there is a change in environmental conditions. Future inspection of the structure can be done at the normal General or Principal Inspections interval.

Note 2: For grades 3 to 8 continuing core extraction and testing should be carried out at the recommended time intervals to establish contemporary worst credible strengths of the deteriorated concrete. This re-testing will demonstrate whether the level of deterioration assumed in the sensitivity analysis is realistic, but may indicate that the structure is in better condition than predicted.

Note 3: For grades 9 & 10 the re-test intervals have been stated as 'N/A' because the concrete is evidently in very poor condition and further testing would not provide more useful information. Measures to repair or replace affected elements are required as a priority.

IDC affects concrete material properties at different rates, the usual equations defining the relationship between the material properties are not valid. Therefore, when tensile strength or elastic modulus is required in a structural assessment, it is necessary to apply degradation factors (F_{DC}) which may be used to derive appropriate values of tensile strength (F_{DT}) and elastic modulus (F_{DE}) for structure assessment.

By reference to ASTM C856 [Ref 34.I] and ISE ASR 1992 [Ref 37.I] and F_{DC}, the F_{DT} and F_{DE} can be derived.

A3 Summary

The steps to be taken to determine the concrete properties which reflect the change in condition due to IDC are set out below:

Obtain sufficient core samples from the structure to allow testing to determine the worst credible compressive and tensile strengths (WCS_{dc} and WCS_{dt}) together with elastic modulus.

Determine the predicted values for tensile strength and elastic modulus using the codified relationship between those parameters and the value for worst credible compressive strength. Compare these predicted values with the test results for these parameters for evidence of IDC.

Carry out phase one of the assessment of the concrete structure in accordance with CS 455 [Ref 37.N], Technical Guide 9 [Ref 18.I] replacing characteristic concrete material properties within the various strength equations with those worst credible strength material properties determined by testing.

Carry out petrographic examination of core samples to decide whether any form of IDC has affected the concrete and to confirm the extent of deterioration, compressive strength degradation factor (F_{DC}) and to decide an appropriate re-test interval during which the deterioration is assumed to occur and material properties need to be confirmed again.

Structure assessment activities aligning with these inspection and test activities can be found in CS 454 [Ref 2.N] and CS 455 [Ref 37.N].