THE SCOTTISH OFFICE DEVELOPMENT DEPARTMENT

THE WELSH OFFICE
Y SWYDDFA GYMREIG

THE DEPARTMENT OF THE ENVIRONMENT FOR
NORTHERN IRELAND

Inspection and Repair of Concrete Highway Structures

Summary: This Advice Note is intended to assist the Engineer responsible for carrying out Principal Inspections, Special Inspections and the repair of concrete highway structures.

Volume 3 home page

VOLUME 3 HIGHWAY STRUCTURES: INSPECTION AND MAINTENANCE

SECTION 3 REPAIR

PART

BA 35/90

INSPECTION AND REPAIR OF CONCRETE HIGHWAY STRUCTURES

Contents

Chapter

Volume 3 home page

1. INTRODUCTION

1.1 The Departments procedures for inspections are given in TRMM 2/88, Trunk Road and Motorway Structures - Records and Inspection. Guidance is also given in the Bridge Inspection Guide published by HMSO. To enable structures to retain their serviceability it is important that defects and causes of deterioration are identified as soon as possible so that remedial works can be carried out.

1.2 The most serious cause of deterioration in the Department's concrete highway structures is reinforcement corrosion due to the presence of free chloride ions in the concrete. These come mainly from de-icing salt, although at coastal sites wind-borne chlorides and sea-water are alternative sources. Alkali-silica reaction (ASR) is another cause of deterioration. In both cases a substantial reduction in the service lives of affected structures may occur.

2. SCOPE

2.1 This Departmental Advice Note is intended to assist the Engineer responsible for carrying out Principal Inspections, Special Inspections and the repair of concrete highway structures. It supersedes Departmental Advice Note BA 23/86.

3. SAFETY

3.1 The provisions of the various statutory or Authority's requirements for safety should be observed. The main safety aspects for work on the Departments properties are traffic signing, working near trafficked roads and running lines, and precautions for handling and use of material for impregnating concrete surfaces.

3.2 The Engineer should consult the Environmental Health Officer, the Health and Safety Executive, Water Authority and any other interested parties as soon as the scale and location of the work is known so that any precautions considered necessary can be arranged.

4. DEFECTS IN CONCRETE

4.1 General

The first indications of serious deterioration can usually be identified in General or Principal Inspections (refer to Trunk Road Management and Maintenance Notice TRMM 2/88). The visual signs are cracks, spalls and rust stains. However local pitting corrosion can develop with no obvious signs and additional internal stresses may be induced by ASR. On bridge decks, failure of the carriageway surfacing may indicate problems with the deck concrete. It is important to determine the causes of deterioration and their likely consequences before deciding on the type and scale of remedial works.

4.2 Durability

Durability may be defined in general terms as the ability of a structure to retain its serviceability. Durability is affected by original design faults, poor detailing, the use of unsuitable materials, shortfalls in workmanship and lack of routine maintenance. These inadequacies accelerate penetration of the concrete by atmospheric carbon dioxide and water borne chloride ions from de-icing salts and marine environments.

Serious breakdown occurs where concrete is permeable or concrete cover to reinforcement is deficient. Permeable concrete is caused by high water/cement ratios, low cement contents, inadequate curing and poor compaction. Lack of an effective waterproof membrane, blocked surface water drainage, leaking expansion joints or spray from passing traffic are all means by which water reaches the surfaces of decks, piers and abutments. Concrete with a high alkali content, reactive aggregates and sufficient moisture is also at risk from alkali-silica reaction (see Section 11).

4.3 Surface Deterioration of Concrete

Frost damage and leaching are probably the most common causes of surface deterioration.

4.3.1 Frost Damage

This usually starts as scaling and may be followed by loosening of the surface aggregates and is most prevalent on horizontal surfaces. Frost damage on vertical surfaces is often associated with spalling and progressive cracking. Poor quality or poorly compacted concretes and concretes containing shrinkable aggregates are readily saturated and vulnerable. De-icing salts and urea exacerbate the problem as thermal shock is caused when latent heat required to melt snow and ice is extracted from the concrete. The resulting differential temperature induces secondary tensile stresses, which in combination with stresses already in the concrete exceed the tensile strength and scaling occurs.

4.3.2 Leaching

Water migrating through permeable concrete reduces both its alkalinity and its strength as it leaches lime from the concrete and deposits it on the surface. Evaporation of moisture leaves unsightly deposits on the surface. Leaching is frequently associated with construction joints although these are not essential for its occurrence.

4.4 Cracks

Fine cracks may occur in tensile zones of reinforced concrete members under normal design loading conditions because concrete can only sustain a relatively small tensile stress before cracking develops. Design codes limit calculated crack widths according to exposure condition. However cracks may also form for a variety of reasons eg structural defficiency, settlement or ASR. Thermal and shrinkage cracks may be associated with high cement contents. All cracking may affect the serviceability of a member. Where cracks significantly increase the permeability of the concrete and the risk of reinforcement corrosion they should be considered deleterious.

4.5 Corrosion of the Reinforcement

Corrosion can be classified into two types: general corrosion associated with carbonation and pitting corrosion associated with chloride ions.

4.5.1 Carbonation is a chemical reaction between atmospheric carbon dioxide and hydrated cement compounds which causes a reduction in the alkalinity of the concrete. The rate of carbonation is dependent on the permeability and moisture content of the concrete. As the pH of the concrete falls below 11.5 the protection afforded to the reinforcing steel is reduced, and corrosion may occur. General corrosion is characterized by superficial corrosion products being generated over relatively large areas of the reinforcement causing cracking and spalling of the cover concrete. Usually there is initially a minimal loss of section. Conditions for rapid carbonation are not entirely the same as conditions for reinforcement corrosion but the depth of carbonation is of significance in assessing the risk of corrosion. In modern well constructed highway structures the rate of carbonation is low giving a long period of time before the steel is depassivated. Urea used as a de-icing agent decomposes into carbon dioxide and ammonia. Its use may increase the risk of carbonation.

4.5.2 Penetration of concrete by chloride ions from de-icing salts and marine environments is the primary cause of reinforcement corrosion in highway structures. The miniscule and highly mobile free chloride ion is able to penetrate concrete through the water present in the pore structure. The passive layer surrounding the reinforcement is locally broken down, causing the anode of an electro-chemical cell to form and anodic pitting corrosion to develop. The cathode may be an adjacent area of steel, or other layers of reinforcement. Once corrosion is initiated the rate of corrosion is determined by the availability of oxygen and moisture and the resistivity of the concrete, but pitting corrosion is rapid and severe local loss of section can occur. Corrosion will be accelerated in damp concrete with a high water/cement ratio and low electrical resistivity. The products of pitting corrosion may initially be black with no external visual clues to their existence. Where sufficient oxygen is available the black corrosion product turns to red rust which is expansive and can lead to cracking of the cover concrete.

5. PRINCIPAL INSPECTIONS

5.1 General

The requirements for Principal Inspections are given in TRMM 2/88. In order to better assess the condition of reinforced concrete structures, a limited amount of site testing should be carried out in addition to a visual examination. It should be confined to those members most likely to be at risk from reinforcement corrosion eg areas below deck joints and/or subject to salt traffic spray. Testing should include half-cell potential, chloride level, covermeter and depth of carbonation measurements. Such non-destructive testing should enable the early diagnosis of corrosion and indicate where Special Inspections should be undertaken. For subsequent Principal Inspections it should only be necessary to carry out half-cell potential measurements.

5.2 Visual Examination

When carrying out the visual examination particular attention should be given to the following:

5.2.1 Cracking

The nature and extent of any cracks and crack patterns. Cracks generally show up better when wet concrete is drying.

5.2.2 Wet or Damp Surfaces

Surface staining due to leaking expansion joints, malfunction of the drainage system or water penetration through the structure.

5.2.3 Corrosion of the Reinforcement

The presence of spalling and rust stains indicates corrosion, though exposure of the steel may have accelerated the rate of corrosion. Examination of exposed reinforcement will confirm the depth of cover at that location and degree of corrosion. Local corrosion may not be detected by visual examination.

5.2.4 The effectiveness of any remedial work already carried out.

5.2.5 Hollow Surfaces or Delamination

"Ringing" with a light hammer or a short length of steel tubing can sometimes indicate these areas.

5.2.6 Bearings and Expansion Joints

Signs of distress in the structure caused by locked up forces. Leakage of water through joints.

5.2.7 Post Tensioned Segmental Construction

For post tensioned construction without an insitu deck slab where prestressing tendons pass through construction joints, the integrity of the joints between units should be checked to ensure they are watertight. If there is evidence of leakage or rust staining, a Special Inspection should be carried out to examine the condition of the tendons. Where drain holes are provided they should be inspected to ensure they are not blocked.

5.3 Testing for Chloride Damage

The test areas given below are for guidance only. Within each test area depth of cover and half-cell potential measurements should be taken on a 500 mm grid and dust samples for chloride analysis taken from positions of numerically high half-cell potentials. This may be supplemented if necessary by half-cell potential measurements over the whole member and spot chloride tests at positions of numerically high half-cell potentials. Where appropriate, permanent connections to the reinforcement should be made to facilitate half-cell testing.

5.3.1 Reinforced Concrete Piers, Abutments, Columns and Crossheads

The test areas should be 2m x 1m. For members with a deck joint above, the test areas should be located where staining has occurred and generally in accordance with Figure 1. Where these members are below deck joints and are also subjected to salt traffic spray, the test areas shown in Figure 2 should also apply.

5.3.2 Reinforced Concrete Wingwalls and Retaining Walls

Where these members are subjected to salt traffic spray the test areas should be as for leaf piers and abutments shown in Figure 2, except that the test area should be repeated every 5m along a horizontal line.

5.3.3 Reinforced Concrete Parapets and Parapet Plinths

For reinforced concrete parapets subjected to salt traffic spray the test areas should be on the traffic face. The test areas should be 2m x 1m or 2m x 0.5m or other convenient dimensions, 100mm below the top edge of the concrete parapet and repeated every 5m along a horizontal line. For reinforced concrete parapet plinths subjected to salt traffic spray the test area should be 1m long and repeated every 5m along the length of the plinth.

6. SPECIAL INSTRUCTIONS

6.1 General

Where deterioration of concrete members is observed or detected during General or Principal Inspections, or if the presence of ASR is suspected, the Engineer should consider a Special Inspection. The objective is to determine the cause and extent of deterioration for the purpose of assessing structural integrity and specifying remedial measures. Special Inspections may require a staged approach ie a preliminary investigation followed by monitoring or a detailed investigation or both.

6.2 Preliminary Investigation

The first step in a preliminary investigation is to refer to all previous records including forms BE 11/85 and BE 14/86 (refer to TRMM 2/88). If information from a recent Principal Inspection is not available, it may be necessary to carry out a visual examination. This will establish the need for monitoring or insitu and laboratory testing.

6.3 Monitoring

After the preliminary investigation it may be necessary to monitor the structure before coming to a decision on the need for a more comprehensive investigation. The rate of deterioration should be assessed in relation to external influences. In the case of cracks where more information is required before a diagnosis is made, periodic measurement will indicate if movement is still taking place or has ceased.

During the resurfacing/waterproofing of reinforced concrete bridge decks the opportunity should be taken to carry out chloride ion and half-cell potential tests. This allows bridge decks to be monitored without damage to the waterproof membrane and disruption to traffic. One chloride test should be taken at 3 metre intervals per lane width. Additional tests should be carried out when half-cell potential measurements are numerically greater than -350mV (with respect to a copper/copper sulphate reference electrode) and at vulnerable areas such as expansion joints and kerb lines.

6.4 Detailed Investigation

The Engineer should prepare a schedule of sampling and testing based upon the recommendations given in Section 7. On large projects or where deterioration is severe, a pilot investigation should first be carried out to determine the types and numbers of tests required.

6.5 Post Tensioned Segmental Construction

Where a Special Inspection is required to examine the condition of tendons this should be carried out by careful selective drilling/water jetting and observation using an endoscope. To avoid damage to the tendon an automatic-stop drill should be used. All voids within post tensioned segmental decks should be provided with drainage holes (refer to Section 10).

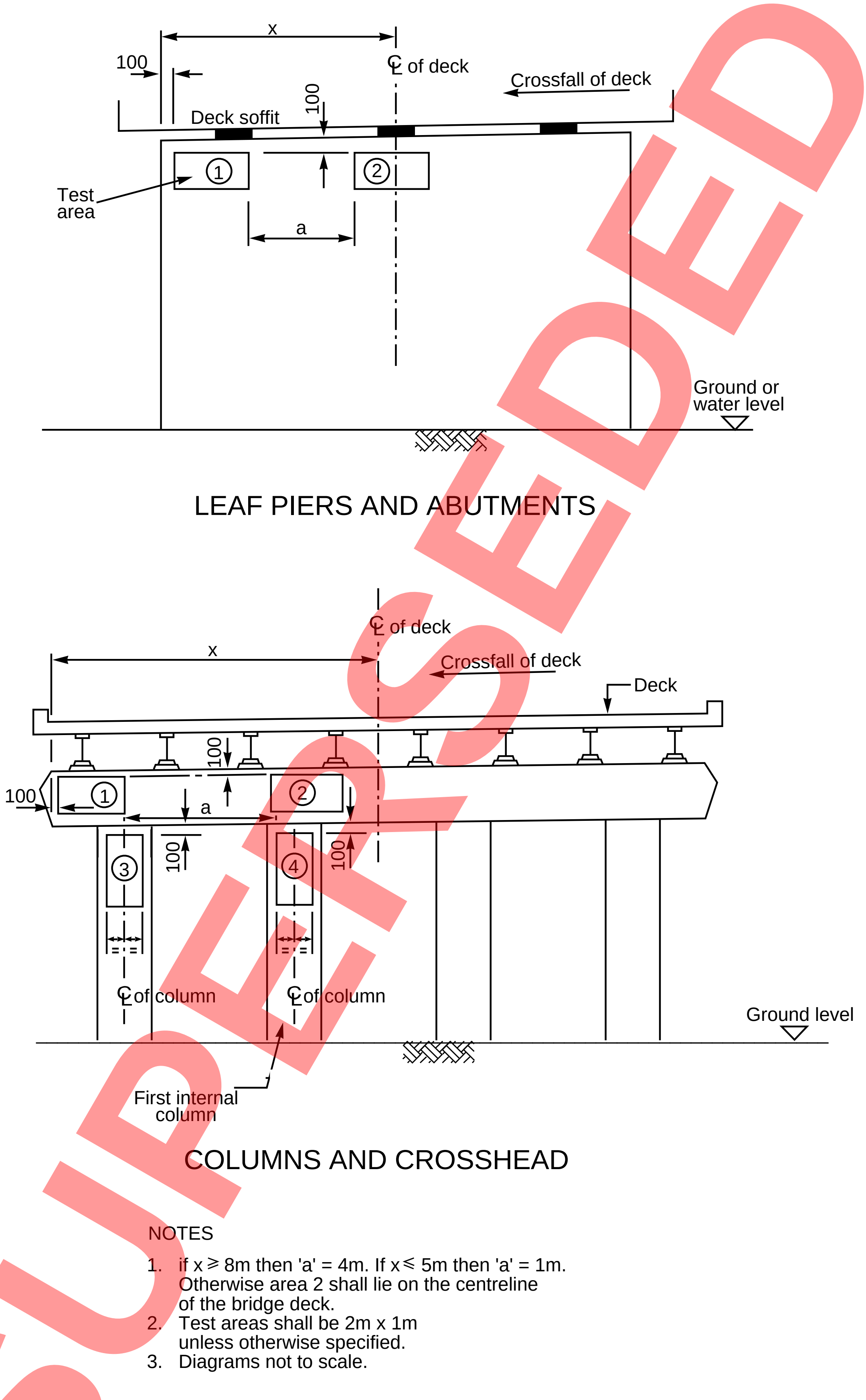
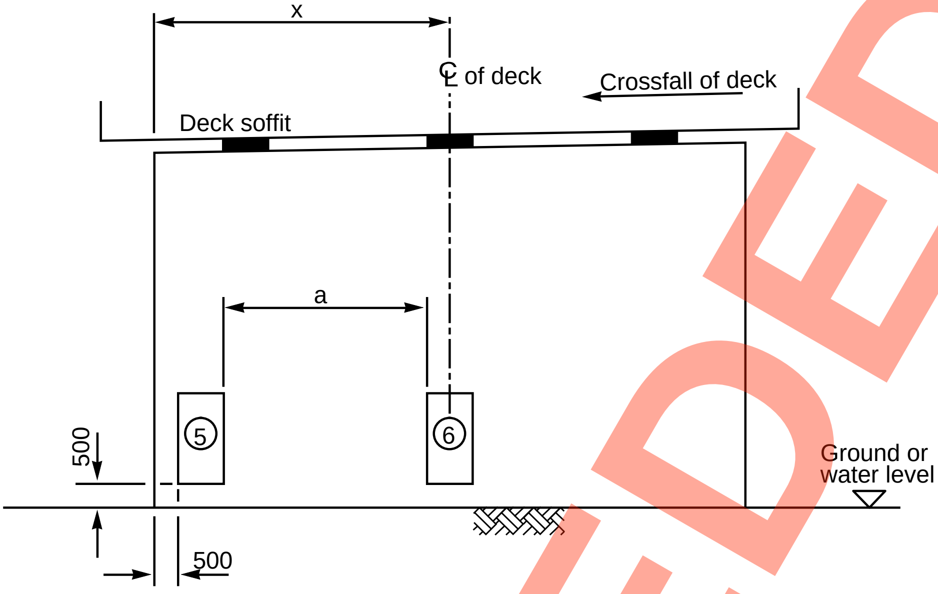
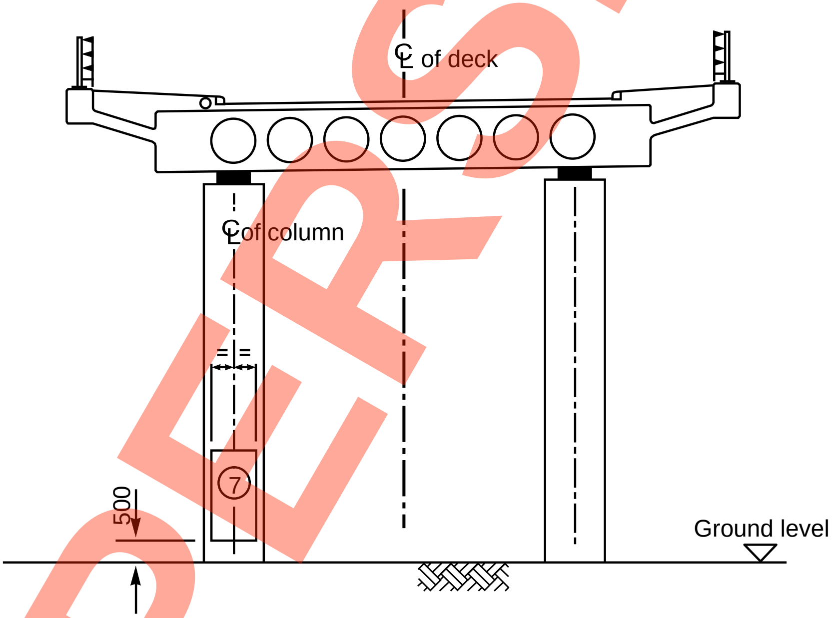


Figure 1: LOCATION OF TEST AREAS FOR REINFORCED PIERS, ABUTMENTS, COLUMNS AND CROSSHEADS WITH A DECK JOINT ABOVE.



LEAF PIERS AND ABUTMENTS



COLUMNS

NOTES:

1. if x ⩾ 8m then 'a' = 4m. If x ⩽ 5m then 'a' = 1m. Otherwise area 6 shall lie on the centreline of the bridge deck.
2. Test areas shall be 2m x 1m unless otherwise specified.
3. Diagrams not to scale.

Figure 2: LOCATION OF TEST AREAS FOR REINFORCED PIERS, ABUTMENTS AND COLUMNS SUBJECT TO SALT TRAFFIC SPRAY

7. TESTING

7.1 General

All sampling and testing should be carried out by specialist testing firms or laboratories approved by the National Measurement Accreditation System (NAMAS) for laboratory testing, or by equivalent accreditation bodies of member states of the European Community. Test information is of fundamental importance in assessing the condition of a structure and the likelihood of reinforcement corrosion. Tests should be accurately related to their location in the structure and it is recommended that the test area is gridded at 500mm intervals, or other convenient spacing for reference.

7.2 Condition of Concrete

7.2.1 General

Cores can provide information on:

(a) strength (7.2.2)

(b) compaction

(c) crack geometry

(d) cement content (7.2.3)

(e) water/cement ratio (7.2.4)

(f) aggregate type

(g) carbonation (7.2.5)

(h) chloride content (7.3.2)

(i) alkali-silica reaction (11.)

A diameter of 100mm is required for compression testing, but cores of 50mm diameter can also provide useful information on quality where 100mm cores are impracticable. Drilling, core cutting and making-good core holes and holes for chloride tests, should be carried out under the supervision of the Engineer to ensure that the integrity of the structure is not affected.

7.2.2 Strength

Tests for strength on cores should comply with BS 1881: Part 120. The BRE pull out test (BRE CP 25/77) and the Schmidt Hammer are in-situ methods of assessing concrete quality at the surface; the results give a comparative guide to strength. Ultrasonic methods (BS 1881: Part 203) can also be used where appropriate to give an indication of strength.

7.2.3 Cement Content

It is normally possible to determine the cement content to within ±15% of that originally used in the mix. Tests should be carried out in accordance with BS 1881: Part 6 and BS 4551.

7.2.4 Water/Cement Ratio

A test method for determining the water/cement ratio in the mix is described in BS 1881: Part 6.

7.2.5 Carbonation of the Concrete

The extent of carbonation should be determined by spraying a fractured surface with phenolphthalein indicator. The colour of any carbonated concrete is unchanged but the uncarbonated material turns purple. As the pH value at which the colour change occurs is about 9, this test indicates concrete which has ceased to be protective. The depth of carbonation may also be determined by petrographic examination. The rate of carbonation varies with age, exposure and quality of concrete, which will influence choice of sample location.

7.3 Assessment of Corrosion Risk

7.3.1 Covermeter

The time to initiation of corrosion is reduced when cover to the reinforcement is less than specified. The depth of cover afforded by the concrete will need to be measured using a suitable covermeter. Measurements should be taken on a grid system, compatible with the reinforcement arrangement. An average site accuracy of about ±15 per cent may be expected with a maximum error of ± 5mm. A check to confirm that the instrument has been properly calibrated should be carried out at a convenient location on the structure before any field measurements are taken. This may be done using the covermeter to record the cover and then breaking out the concrete at the same location to expose the reinforcement so that a physical measurement of the actual cover can be recorded. Corrections should be applied for bar diameters outside the range of 10mm to 32mm, high yield bars, special cements, heavy or lightweight concretes and curved bars. Inaccuracies can result from tying wire, dense or multi-layered reinforcement and the close proximity of overhead power lines. The covermeter is particularly useful for determining the location of reinforcement prior to core drilling.

7.3.2 Total Chloride Ion Content of the Concrete

Total chloride ion content is determined by analysing dust samples taken from holes drilled into the concrete. For insitu concrete, holes should be 20-25mm in diameter. Alternatively core samples may be used. For prestressed concrete members the position of tendons and wires should first be located. Holes should be not greater than 10mm in diameter. Guidance on frequency and location of sampling is given in BRE Information Paper IP 21/86. Chemical analysis is performed by the method described in BS 1881: Part 6. A quick simplified method available as a field test is described in IP 21/86, but this method is not as accurate as the laboratory method.

Results of chloride analysis can be presented as a profile, chloride concentration against depth, and care must be taken to separate the incremental samples. The analysis provides the total chloride ion content from all sources, both during construction and in service. Most of the chloride that was present in the wet mix will have combined with other constituents to form stable compounds.

7.3.3 Half-Cell Potential

The half-cell potential method together with chloride ion content data enables the risk of corrosion to be assessed. Corrosion potential measurements are made using a reference electrode placed on the concrete surface, which is connected via a high impedance voltmeter to the reinforcement. The method is described in ASTM C 876. By convention, potentials are considered negative when measuring the steel with respect to the electrode. Potentials numerically less than -200mV indicate a low probability of corrosion. The half-cell potentials given in this Advice Note are in terms of the copper/copper sulphate electrode. The interpretation of results requires experience and due account must be taken not only of absolute values of current, but rate of change of potential and moisture content of the concrete. Where concrete has been impregnated with iso-butyl silane there will be a shift in half-cell potential measurements and due account of this must be taken in interpreting the readings. Before carrying out measurements the Structures File (refer to TRMM 2/88) should be examined to determine whether any concrete members have been impregnated.

7.3.4 Resistivity of the Concrete

Resistivity is related to the moisture content and quality of the concrete. Resistivity measurements can give information which help in the interpretation of half-cell potential tests made under the same conditions. Experience has shown that in regions where the half-cell potential is numerically greater than -350mV (eg -400mV):

(a) If the resistivity is greater than 12,000 ohm cm, corrosion is unlikely.

(b) If the resistivity is in the range 5,000-12,000 ohm cm, corrosion is probable.

(c) If the resistivity is less than 5,000 ohm cm, corrosion is almost certain.

The technique is similar to that used for measuring soil resistivity and uses electrodes temporarily attached to the concrete, across which measurements of voltage and current are taken. As standardised equipment for this test is generally not yet available it is recommended that where required, tests are carried out by an approved testing firm which has developed its own apparatus and trained staff in its use. There are a large number of possible factors which may affect electrical potential and resistivity measurements and hence the actual values determined. An experienced testing firm will be aware of these.

8. REPAIRS TO CHLORIDE CONTAMINATED STRUCTURES

8.1 General

It is not possible to forecast the life of concrete repairs. Where the results of insitu and laboratory tests confirm that extensive and severe deterioration has occurred, an economic comparison should be made between the cost of repair and replacing the member concerned.

Partial renewals, strengthening and repairs affecting the integrity or load carrying capacity of existing structures, are subject to the Technical Approval procedures given in Departmental Standard BD 2/89: Part 1.

Before any repairs are carried out it is essential to ensure that all surface water leaves the structure through properly provided drainage channels. It may be necessary to correct any deficiencies in deck waterproofing and expansion joints by repair or replacement. Specialised drainage systems have been developed for use in confined areas beneath deck joints.

8.2 Cracks

Repairs to cracks should not be carried out until the cause of cracking has been established and dealt with, and monitoring (6.3) confirms that movement has ceased. Where deleterious cracking has occurred in areas exposed to de-icing salts or in marine environments, and tests show that the chloride content at the level of reinforcement exceeds 0.3 per cent by weight of cement, the concrete should be removed (8.3). Sealing cracks by vacuum or injection methods alone will not prevent chloride ingress and if protection is required these areas should also be impregnated. Crack width measuring devices suitable for site use are now widely available.

8.3 Removal of Concrete

The Engineer should ensure that concrete removal is programmed to maintain structural integrity and continuity of load transference in the member. Propping may be required in some instances, particularly if it is considered necessary to simultaneously remove concrete from more than one face of the member. Large areas of concrete may be removed using high pressure water jetting. This method may not be appropriate in all situations due to difficulties with access and safety arrangements. Care should be exercised in removing concrete from prestressed concrete members. The position of tendons and wires should first be located, if necessary hand methods should be used to expose tendons and assess their condition.

The extent of the repaired area will depend on the degree of contamination, visual examination and the results of tests carried out. It will also be influenced by the interpretation of half-cell potential and chloride measurements. Generally where chloride ion contents at the level of the reinforcement exceed 0.3 per cent (total chloride ion content) by weight of cement and half-cell potential measurements are numerically greater than -350mV, there is a high risk of corrosion occurring and concrete in these areas should be removed.

To avoid 'patch' repairs the extent of concrete to be removed should be not less than 1 sq m in area and should extend a minimum of 100mm along the bar beyond any corroded areas (pitted bars as distinct from superficially rusty bars). The edges should be saw cut to a minimum depth of 10mm to ensure that repaired areas are kept to well defined lines, which should also improve the overall appearance of the repair. To enable the replacement concrete to be placed and to provide a mechanical key, concrete should be removed for a distance of 30mm behind the reinforcement. If power hammers are to be used to remove contaminated concrete, pick damage to reinforcing steel and micro-cracking in the remaining concrete can be minimised by using a pointed tool.

8.4 Cleaning of Reinforcement

Dry methods are not effective in removing corrosion products. Where possible high pressure water jetting should be used or alternatively wet blast cleaning. Particular attention should be given to pits and crevices. Where significant loss of section has occurred or pitting is extensive, reinforcement should be replaced. Additional reinforcement may also be required to restore serviceability if mechanical damage to reinforcement has taken place during removal of concrete. While repairs are being carried out, care should be taken to protect exposed concrete substrates and reinforcement from salt spray from passing traffic.

8.5 Replacement Concrete

8.5.1 General

Concreting should be programmed to minimise shrinkage cracks. Wherever practical, deficient cover to the reinforcement should be increased to meet current standards. To avoid ponding, all horizontal surfaces ie tops of bearing shelves, piers and crossheads should be finished to a fall of 1 in 5.

Blended cement concretes properly placed and cured reduce the penetration rate of chloride ions by changing the pore size distribution and chloride binding capacity of the cement paste. They will assist in reducing the risk of chloride ion diffusing back into repaired areas. The use of bagged aggregates and cement either singly or combined offer advantages for small amounts of concrete mixed on site. It will also enable quality control to be maintained.

8.5.2 Water Cement Ratio and Curing

These have a profound affect on reducing permeability and increasing the durability of concrete. It is important that a water/cement ratio of 0.4 is not exceeded. All repaired areas should be continuously water cured for a minimum of 5 days after placing the concrete. This can be achieved using a hose pipe 'sprinkler' system together with absorbent material covered by polythene sheeting. Proprietary curing membranes do not provide a sufficient reservoir of water in the early stages of curing and the pore blocking action of some may reduce the efficiency of subsequent impregnation.

The importance of curing to reduce shrinkage cracking and increase the durability of concrete repairs cannot be over emphasised. A separate item covering this work should be included in the Bill of Quantities.

8.5.3 Sprayed Concrete

In situations where reinforcement is not too congested sprayed concrete may be used. Care should be taken to avoid creating 'shadow areas' behind the reinforcement. In areas that are aesthetically sensitive, the final coat should be float finished to provide a smooth surface. Further information is given in the Sprayed Concrete Association's Manual. Procedure trials should be carried out to demonstrate the Contractor's competence, method of working and mix design.

8.6 Minor Repairs

These include repairs to arrises and reinstatement of core holes etc.

8.7 Repair of Particular Elements

8.7.1 Crossheads

Crossheads are particularly vulnerable to chloride attack from leaking expansion joints. Where deterioration is severe it may be necessary to provide temporary supports while repairs are carried out. This will require careful programming and traffic management. For repairs to the sides and soffits of crossheads using high flow materials, strict control should be exercised at all stages during mixing and placing of the concrete.

8.7.2 Bridge Decks

Contamination of bridge decks generally occurs through failure or damage to the waterproofing membrane. The location, extent and severity of the problem may only become apparent when resurfacing/waterproofing is carried out. Damp patches on the soffit while not necessarily indicating the position of faults in the membrane, can affect half-cell potential measurements particularly on thin deck slabs. Where contamination is severe, consideration should be given to replacing the deck slab. Where the concrete replacement option has been chosen, high pressure water jetting will reduce the risk of damage to the reinforcement. This is particularly relevant to thin deck slabs. Localised contamination at expansion joints can occur due to inadequate sealing. All repaired areas should be continuously water cured for a minimum of 5 days after placing of the concrete, and be surface dry for a minimum of 24 hours before application of the waterproof membrane.

9. SURFACE IMPREGNATION

Concrete is not impermeable to the migration of chloride ions. It is possible that without protection, soundly constructed and properly maintained structures subjected to sustained chloride attack will eventually deteriorate during their service life.

Impregnation is carried out by spraying concrete surfaces with a hydrophobising material that achieves maximum penetration of the concrete and reacts with the silicates and moisture present. This produces a water-repellant but vapour-permeable layer that inhibits the ingress of water and chloride ion. Effectiveness of this layer is determined by the quality of the hydrophobisation and the strength and permanence of the bond between the silane molecule and the substrate. Impregnation is known to be effective for at least 15 years provided it is applied correctly. Longer service lives are anticipated. However it is considered advisable to assume that re-application may be necessary after about 20 years.

The depth of penetration will vary depending on concrete quality and moisture content. To obtain feedback, a record should be kept of the quantity of material used on treated areas and forwarded through the Regional Office to BE Division.

After curing, the whole of repaired structural members including the tops of piers, crossheads and bearing shelves should be impregnated. Graffiti and encrusted surface deposits ie algae should be removed by light dry grit blasting or wire brushing. Areas with cracks not exceeding 0.3mm in width should be waterproofed by impregnation. Bridge deck surfaces which are to be waterproofed in accordance with Technical Memorandum BE 27 should not be treated. A minimum interval of 14 days after placing the concrete is required before impregnation to allow the concrete pore structure to develop.

10. DRAINING OF VOIDS WITHIN BRIDGE DECKS

10.1 Collection of water within the voids of bridge decks has occurred in some instances. Water finding its way into the voids is likely to remain there unless provision has been made for drainage. It will increase the risk of corrosion of the steel reinforcement and possibly reduce the live load carrying capacity of the bridge. Voids within existing bridge decks should therefore be provided with drainage holes at the earliest opportunity.

10.2 Four types of non-recoverable void formers have been used in cellular or voided bridge decks:

(a) Spirally wound corrugated steel tubes.

(b) Waxed cardboard tubes and boxes.

(c) Timber.

(d) Expanded polystyrene void formers to any shape.

10.3 The following methods have been used to provide internal formwork to precast pretensioned box beams:

(a) Recoverable formwork where the concrete is cast in two stages.

(b) Recoverable, tensioned, high-tensile wire wrapped with wire netting followed by polythene sheets.

(c) Permanent formwork of hollow prefabricated box formers.

(d) Permanent formwork of expanded polystyrene.

10.4 Precast pretensioned U beams formed using collapsible steel forms and pseudo-box construction using M beams should both be drained.

10.5 As-built Contract Drawings should be examined to verify the method used to form the voids and the position of tendons and reinforcement. The latter should be verified by non-destructive testing before any drilling is started. For voided and cellular bridge decks one 22mm diameter hole should be drilled, while for precast pretensioned box, U beams and pseudo-box construction, two 15-20mm diameter holes should be drilled near each end on the centre line of each void or at the low point away from adjacent vertical concrete members using a small gauge diamond drill.

10.6 Before commencing drilling operations as-built Contract Drawings should be checked for the locations of any Statutory Undertakers Services ie gas, water, electricity, telephone etc, and their positions confirmed on site where they are in the vicinity of the holes to be drilled. Extreme caution should be exercised during the drilling operation to avoid causing damage to tendons. Water held in deck voids can be highly alkaline and consequently potentially harmful. Precautions should be taken to avoid the water coming into contact with eyes or skin. If water is present in the void it should be tested for its chloride ion content, its source of entry should be identified if possible, and measures taken to prevent further ingress into the voids. If there is any evidence of rust staining or corrosion to reinforcement then consideration should be given to treatment or repair of such areas.

10.7 Once the holes have been drilled a 13mm nominal bore and 50mm length tube of inert material eg UPVC or similar with a screw thread should be fixed into the hole using a polyester resin so that it projects 25mm below the soffit (or lesser dimension where the headroom is affected), as shown in Figure 3 for a circular voided slab bridge deck.

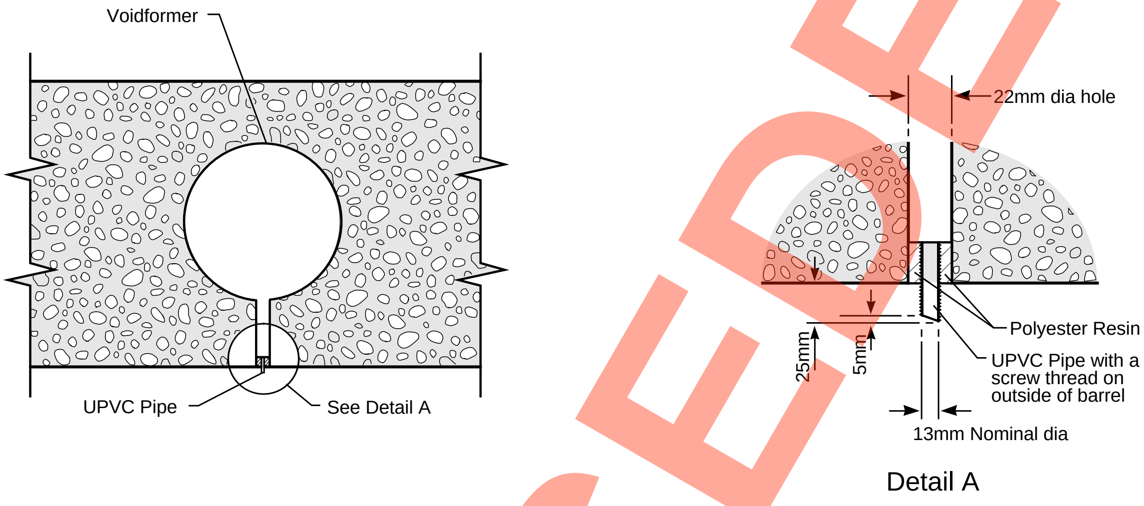


Figure 3: DRAINING OF EXISTING VOIDED SLAB BRIDGES

11. ALKALI - SILICA REACTION

11.1 Introduction

As part of the 15 year programme for upgrading the Department's bridge stock, it is intended to establish a common national basis for the investigation, monitoring and repair of those Departmental structures on motorway and trunk roads which are affected by alkali silica reaction (ASR). Attention is drawn to recently published guidance on this subject by the British Cement Association (BCA) on the diagnosis of ASR, and by the Institution of Structural Engineers (ISE) on the appraisal of the structural effects of ASR.

ASR is an expansive chemical change causing cracking in hardened concrete. Reactive silica present in some aggregates combines with the alkali in the cement when there is a sufficiently high concentration of hydroxyl ions in the pore solution. The reaction product of this combination is alkali-silica gel which swells as it imbibes water and can exert pressure, causing cracking in the concrete. The gel may form in relatively dry conditions but for the reaction to cause damage to the concrete, there must be a sufficient supply of moisture available. Limiting the supply of water will generally slow down the progress of ASR though it can occur in the presence of water vapour alone. Thus it is important to detect ASR at an early stage so that measures can be taken to exclude water and de-icing salts in order to keep concrete structures in service as long as possible.

A number of publications have been issued in recent years dealing with various aspects of ASR. The Concrete Society set up a Working Party under the chairmanship of M R Hawkins in 1983 which concentrated on producing specification clauses to minimise the risk of ASR in new construction. In 1984, the Cement and Concrete Association formed a Working Party to produce guidance on the diagnosis of ASR. This led to the publication in 1988 of the BCA document 'The Diagnosis of Alkali-Silica Reaction'. More recently, an ad-hoc committee was drawn together by the Institution of Structural Engineers to produce guidance on the assessment and management of ASR-affected structures and in 1988 published, 'Structural Effects of Alkali-Silica Reaction - Interim Technical Guidance on Appraisal of Existing Structures'.

The presence of ASR in a steadily increasing number of structures means that there is a need to adopt a common approach to this problem. Up to the present the incidence of ASR in the UK has mainly been confined to the South West and the Midlands. It is now clear however that other areas, hitherto thought to be free of ASR, could be affected. This has largely been brought about because of the importation of materials or structural elements, now known to be at risk from this reaction, into these other areas of the UK.

11.2 Identification

ASR can progress at very different rates and affected structures may take a long time to become unserviceable, with some never reaching that condition. However, it is necessary to identify and monitor the progress of ASR and thereby be in a position to take positive action to prevent deterioration leading to a structure becoming unserviceable during its lifetime. Regular monitoring of affected structures will also lead to the establishment of a comprehensive overview of the national situation with respect to ASR.

When ASR is suspected in the course of General or Principal Inspections, either because of signs of deterioration or other factors, a Special Inspection should be carried out to identify its presence positively. Visual inspection alone cannot confirm or discount the presence of ASR with any certainty. The objectives of a Special Inspection are to establish whether or not the structure exhibits any features which could be consistent with ASR, and also to identify and isolate any features which indicate other mechanisms. Identification of ASR should only be undertaken by specialist testing firms or laboratories approved by the National Measurement Accreditation Service (NAMAS) or by equivalent accreditation bodies of member states of the European Community. Although ASR may have serious implications for the future service life of the structure, it should be remembered that in some circumstances other problems may be of more immediate concern, eg reinforcement corrosion.

The flow chart in Figure 4 illustrates the primary investigative steps that should be undertaken when inspecting structures thought to be affected by ASR. Definitions of three categories of ASR used in the flow chart are as follows:

Suspect - structures which are suspected to be suffering from ASR due to the presence of map-type cracking and/or other factors eg structures categorised as 'potential'.

Confirmed - structures where laboratory tests have confirmed the presence of ASR.

Potential - structures with similar aggregates/alkali contents to structures where ASR has been confirmed.

The BCA document describes the various steps to be undertaken during this process which can lead to the identification of ASR as the primary cause of deterioration or eliminate it from further consideration.

11.3 Management and Monitoring of Structures Affected by ASR

When ASR is confirmed records of other structures in the vicinity should be examined, especially of those constructed in the same contract, to check whether or not the same aggregate and cement were used. If records are not available the aggregate can be identified from cores 50mm diameter by 100mm long. The choice of structures to be examined is determined by the number of different contracts, known geology of aggregate sources in the areas, known local cement alkali contents (at the time of construction if possible), and the number of structures in each contract. Where aggregates are identified as potentially reactive, structures should be observed for signs of ASR in the course of General or Principal Inspections and Special Inspections should follow if considered necessary. The identification of aggregates susceptible to ASR and the results of these investigations are important records for future investigatory work and should be included in the Special Inspection Report. A revised copy of BE 13 should be forwarded for amendment of the Bridge Database.

The need for a structural assessment should be considered if ASR is confirmed. The effects of ASR on the ultimate strength of reinforced concrete is still a subject for research but vulnerable details can be identified. Tests on concrete samples have indicated that ASR-affected concrete generally has a high cube crushing strength for uncracked cores but has a markedly reduced tensile strength as a result of the development of micro-cracking and internal stresses. The expansion of ASR-affected concrete is restrained by reinforcement which sets up stresses and strains in the concrete superimposed upon the structural effects. Details sensitive to the effects of ASR should be carefully noted. These include half-joints, pile caps, zones of high bond stress and concrete elements with little or no shear steel. Further guidance on the effects of ASR on concrete structures is given in the ISE report.

Where ASR has been positively identified, the long-term programme for the management of structures should also be reviewed. The current expansion should be determined by measurement of the crack widths especially at structurally sensitive locations. If possible, overall structural dimensions should also be monitored. The potential for expansion should be estimated from expansion testing of cores. For rapid diagnosis, the expansion testing is normally carried out at 38°C and 100% RH, but there is increasing evidence that over the longer term, lower temperatures will lead to greater expansion levels. The testing of cores for longer periods at lower temperatures should therefore be considered in terms of the structural importance of the element being investigated.

The ISE report suggests generalised guidelines for the management and the monitoring procedures for concrete structures suffering from ASR. It describes the relationship that exists between the site environment, the reinforcement detailing, an expansion index - determined from the current and total

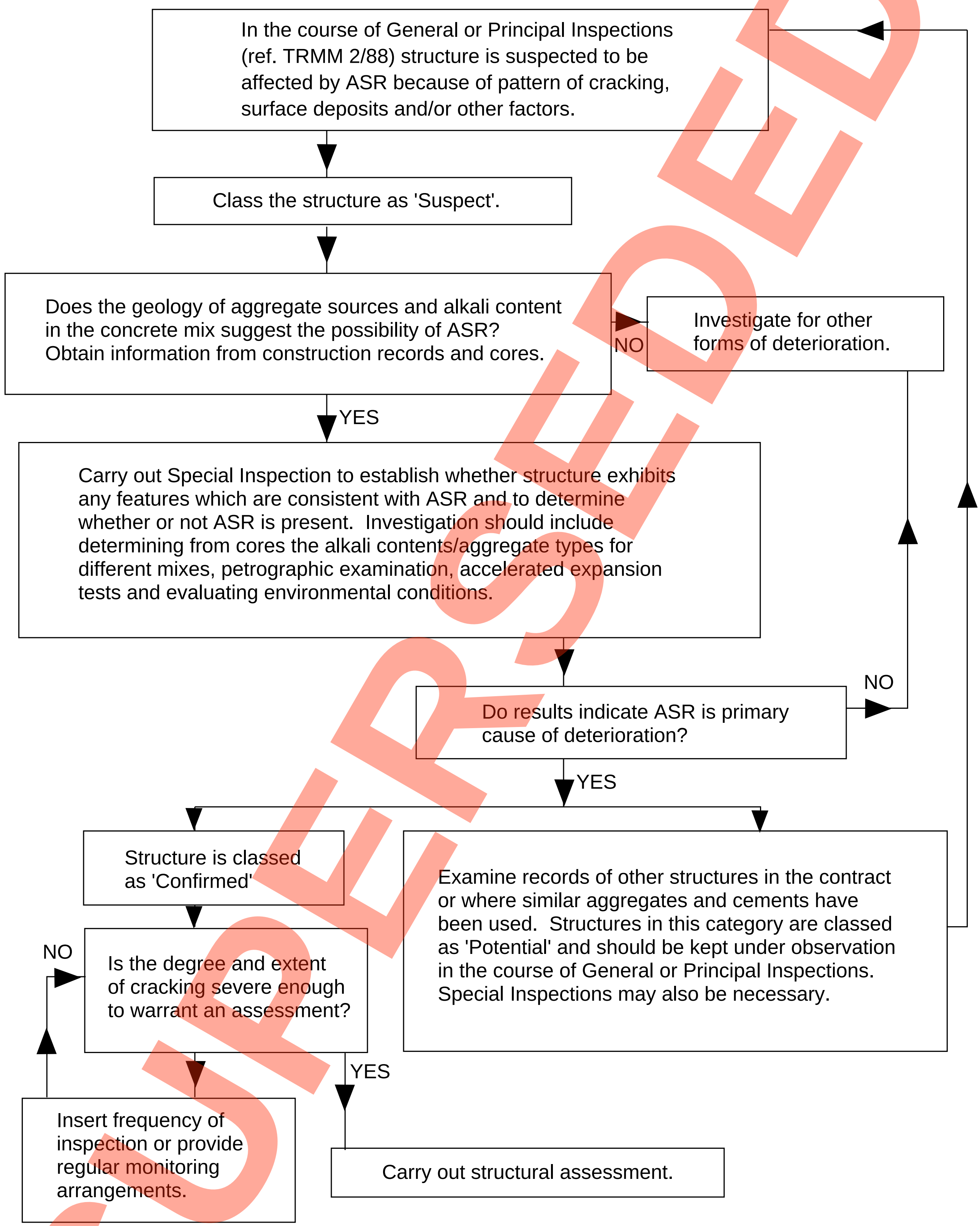


Figure 4 : PRIMARY INVESTIGATIVE STEPS THAT SHOULD BE UNDERTAKEN WHEN INSPECTING STRUCTURES LIKELY TO BE AFFECTED BY ASR.

estimated expansion - and the consequences of future deterioration. This relationship, which ranges from very mild to very serious, should be established for every element that is being investigated and translated into an appropriate management strategy. A structure that merits a 'very severe' rating must be the subject of a detailed specialist investigation. Interim measures such as limited strengthening or load restrictions may be necessary and ultimately complete replacement may have to be considered. Monitoring procedures and strengthening measures should ensure that the safety of the structure is maintained at all times.

11.4 Repair of ASR Affected Structures

There may be practical or economic difficulties in repairing structures severely affected by ASR. Where ASR has been positively identified and is extensive, long term planning should include the possibility that replacement of parts or the whole of the structure may be necessary. It is essential to slow down any further reaction taking place in the concrete, so the ingress of water must be prevented in structures categorised as "confirmed" or "potential". If the potential for expansion remains and water cannot be excluded then deterioration will continue. Any leaking drains and expansion joints should be repaired and all surfaces including parapet upstands, bearing shelves and vertical surfaces exposed to the atmosphere and at risk should be impregnated.

To be effective on structures categorised as "confirmed" or "potential", impregnation should be applied immediately cracking is observed. Where expansion is severe and large fissures have occurred the provision of a ventilated cladding would provide a more sheltered environment. Attempts to remove and replace affected concrete should not be made until tests have confirmed that expansion has ceased.

12. REFERENCES

Departmental and HMSO Publications

1. Bridge Inspection Guide (HMSO) 1983.

2. Trunk Road Management and Maintenance Notice TRMM 2/88, Trunk Road and Motorway Structures - Records and Inspection.

3. Technical Memorandum BE 27, Waterproofing and Surfacing of Bridge Decks.

4. Departmental Standard BD 2/89: Part 1 , Technical Approval of Highway Structures on Motorways and other Trunk Roads.

5. Departmental Standard, BD 43/90, Criteria and Material for the Impregnation of Concrete Highway Structures.

6. Departmental Advice Note BA 33/90, Impregnation of Concrete Highway Structures.

7. The Performance of Concrete in Bridges (HMSO), April 1989.

British Standards (Published by BSI)

8. BS 1881: Methods of Testing Concrete

Part 5: 1970 Methods of testing hardened concrete for other than strength

Analysis of hardened concrete

Part 105: 1984 Method for determination of flow

Part 108: 1983 Method of making test cubes from fresh concrete

Part 111: 1983 Method of normal curing of test specimens

Part 116: 1983 Method for determination of compressive strength of concrete cubes

Part 120: 1983 Method for determination of the compressive strength of concrete cores

Part 201: 1986 Guide to the use of non-destructive methods of test for hardened concrete

Part 202: 1986 Recommendations for surface hardness testing by rebound hammer

Part 203: 1986 Measurement of the velocity of ultrasonic pulses in concrete

Part 204: 1988 Recommendations on the use of electromagnetic covermeters

9. BS 4551: 1980 Methods of testing mortars screeds and plasters.

10. BS 5400: Steel, concrete and composite bridges
Part 4: 1984 Code of practice for design of concrete bridges

American Society for Testing Materials (Published by ASTM)

11. ASTM C876 Method for Half-Cell Potentials of Uncoated Reinforcing Steel in Concrete.

Transport and Road Research Laboratory Publications

12. LR 953: A survey of site tests for the assessment of corrosion in reinforced concrete. P R Vassie

13. LR 981: Case studies of the corrosion of reinforcement in concrete structures. R J Woodward.

14. AG3: Microprocessor controlled multiple half-cell measurements. M A Winnett.

15. RR 81: Repair of cracked concrete: assessment of injection methods. A J Calder.

16. RR 150: Repair of cracked concrete: assessment of corrosion protection. A J Calder and D M Thompson.

17. RR 109: Durability of concrete repairs: the effect of steel cleaning procedures. P R Vassie.

18. The half-cell method of locating corroding reinforcement in concrete structures. Application Guide 9. P R Vassie.

Building Research Establishment Publications

19. CP 25/77: A simple pull out test to assess the strength of in-situ concrete.

20. IP 21/86: Determination of chloride and cement of hardened concrete.

Other References

21 The Diagnosis of Alkali-Silica Reaction. Report of a working party, British Cement Association, 1988.

22. Structural effects of Alkali-Silica Reaction - Interim technical guidance on appraisal of existing structures. The Institution of Structural Engineers, 1988.

13. ENQUIRIES

All enquiries or comments about this Advice Note should be sent in writing to:

Head of Division
Bridges Engineering Division
Department of Transport
St Christopher House
Southwark Street
LONDON SE1 0TE Quoting Reference : BE 28/14/021

Orders for furthur copies of this Advice Note should be accompanied by the remittance shown on the cover and addressed to :-

DOE / DTp Publications Sales Unit
Building One
Victoria Road
South Ruislip
Middlesex
HA4 0NZ
Telephone No : 081-841-3425

P H DAWE
Head of Bridges Engineering Division
Department of Transport
St Christopher House
Southwark Street
LONDON SE1 0TE

June 1990