The Institution of Structural Engineers

Structural effects of alkali-silica reaction

Technical guidance on the appraisal of existing structures

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The Institution of Structural Engineers
11 UPPER BELGRAVE STREET

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Addendum, April 2010

1. Introduction.

1.1 Alkali Aggregate Reaction (AAR) was first recognised in structures in the UK in about 1980. At that time there was very little information worldwide on the effects of AAR on structural behaviour, on the time scale over which AAR damage developed, or on the practical management of cracked structures.

1.2 Testing and monitoring of a wide range of UK structures, combined with research and experience from around the world, provided the information for the 1988 edition of the Institution of Structural Engineers *'Structural effects of alkali-silica reaction. Technical guidance on the appraisal of existing structures'.* This was further developed and refined in 1992 [1] and launched at the 1992 ICAAR conference [2]. It has provided a practical basis for the appraisal of a wide range of structures [3] and its principles have been adopted in other countries [4]. However there have been some important developments internationally since 1992.

1.3 The 'Hawkins' rules [5], introduced and developed in the 1980s and updated by BRE [6], recommended limiting the alkali content in concrete in the UK. This has achieved the objective of minimising the risk of AAR in new construction. So there have been relatively few new UK structures with AAR, none of which are classified as 'severe'. RILEM is developing AAR-7-1 International Specification for Minimising Risk of AAR [7] which confirms and supplements earlier UK recommendations. It is linked to improved tests and petrographic interpretation for assessing aggregates.

1.4 UK research activity on AAR has diminished, but continuing monitoring and management of older structure has yielded some further valuable information [8]. A wide range of reactive minerals in aggregate can give rise to damaging alkali aggregate reaction and this, rather than 'alkali-silica reaction', has become the preferred general term.

2. International Developments.

2.1 The number of cases of AAR worldwide continues to increase and more structures have needed major remedial work or replacement. There are now 47 countries where AAR has been reported, up from 35 in 1992 [9]. However RILEM, which is drawing up an atlas of known reactive rock types, has found that many owners and countries are reluctant to publicise cases. The indications are that few, if any, countries are free from AAR. The growing international trade in cementitious materials and aggregates is increasing risks.

2.2 A number of countries, e.g. France and the Netherlands, which had considered themselves free from AAR in the 1980s, have found damage from AAR in bridges, dams, roads and buildings. They have initiated major research programmes yielding valuable new information from laboratory programmes and testing and monitoring damaged structures. Other countries, e.g. Canada and Japan, have maintained their active research programmes which are progressively improving our understanding.

2.3 Many of these developments have been published, but scattered in the specialist literature. However the ICAAR conferences [2 and 10 - 13], held every three or four years, have brought together papers on research and case studies of materials and engineering practice from around the world.

2.4 Increasingly AAR research is carried out in international cooperative research programmes. RILEM has become the focus for this international cooperation and maintains links with the ICOLD committee working on the particular problems that are developing worldwide from AAR in dams.

2.5 RILEM Technical Committee TC 191-ARP Alkali-Reactivity & Prevention:. Assessment, Specification and Diagnosis is actively developing improved guidance, drawing on the full range international research and practical experience of structures. It has published recommendations 6-1 Diagnosis [14] which supersedes the UK procedures recommended in 1992. This is linked to improved test methods, some of which have been validated, while others are still being developed and evaluated in international inter-laboratory trials. It recommends that, from the start of an investigation, there should be close links between the materials science team inspecting and testing for diagnosis and structural engineers carrying out appraisal [15].

2.6 The drafting of RILEM TC 191-ARP 6-2 Guide on Appraisal and Management of Structures [16] is progressing, with publication hopefully in about 2013. It will be linked to a guide setting out the developments in the modelling the structural behaviour of concrete elements and overall behaviour of structures with AAR. These modelling studies are being calibrated against the long term laboratory testing of concrete beams in controlled conditions and the observed behaviour of structures suffering AAR damage.

3. Significant developments

3.1 The overall philosophy and principles set out in the 1992 IStructE report have gained wide acceptance around the world. It, with material from similar national recommendations, is the framework which RILEM is using to develop its guidance on Appraisal and Management. The RILEM recommendations will include some changes in emphasis, the most important of which are summarised below. More detailed information on developments can be found from ICAAR proceedings.

3.2 Over-reaction to Traces of AAR.

3.2.1 There have been some UK cases where a petrographic examination of cores has identified signs of AAR, but there is little cracking attributable to AAR in the structure. If properly assessed these would be classified in Table 5 ‘Structural severity rating’ as ‘n – negligible’. In some instances demolition has been recommended to the owners before the structure has been assessed for ‘structural severity’ following IStructE recommendations.

3.2.2 It needs to be stressed that 95% of structures with signs of AAR from cracking and confirmed by petrography, need no action other than improved protection and drainage and inspection in accordance with Table 7. Isolated particles in many UK concretes will show signs of AAR under the microscope. This should not be judged without site inspection followed, if appropriate, by full structural assessment.

3.3 Monitoring Cracking and Expansion.

3.3.1 A major uncertainty when the IStructE recommendations were being drafted was the duration of crack development and damage in structures. There were suggestions that it exhausted itself after about 10 years.

3.3.2 Monitoring of structures in the UK [17] and internationally, has now shown that, once cracking is established, generally after about 5 years, cracks will grow at a broadly linear rate as further expansion occurs in the mass. There are few reported instances of the damage development slowing up or stopping, even after over 50 years, unless measures have been taken to reduce the moisture in the damaged elements. The rigorous long term monitoring (16 to 18 years) of expansions of cast blocks with OPC, as part of BRE AAR exposure site programme [18], has confirmed this linear expansion with time with no sign of it slowing up.

3.3.3 This clear evidence that AAR damage and cracking does not stop makes it essential that structures diagnosed in the 1980s and 1990s have a full specialist re-inspection and reappraisal.

3.3.4 In the UK monitoring has mostly been carried out using groups of Demec gauge lengths in 'Triples' across and parallel (to give seasonal thermal and moisture movement correction) to cracks in representative areas. The crack movements reflect internal expansions. Trends often take 2 or 3 years to become clear and need to be based on groups of cracks.

3.3.5 Improvements in instrumentation are making it easier to monitor overall expansions. In France, LCPC [19] have used this on several structures. The influence of restraint in each direction on expansion needs to be considered. Compressive stress from normal structural behaviour and prestress, (and that developed by the restraint of expansion by reinforcement), must be considered in planning and interpreting this data. Monitoring of overall movements and differential movements at joints has been of particular value in dams. Relatively small expansions can cause problems, but are measurable over the height or length of a dam.

3.3.6 In almost all structures there is a trend of steady growth of expansion and cracking with time unless the moisture state is changed. Drying can slow the reaction, but a later increase in moisture in the concrete will accelerate damage. The breakdown of waterproofing onto AAR concrete can trigger rapid damage development. This needs to be considered when inspecting and maintain waterproofing on bridge decks and flat roofs. These are often slabs with few shear stirrups (i. e. with only 2-D restraint), so they are vulnerable to delamination and loss of shear strength. On bridge decks the interaction of corrosion, frost damage and AAR with wheel loading can initiate serious and rapid deterioration. This needs to be highlighted when drawing up inspection and maintenance procedures.

3.4 Inspection frequencies

3.4.1 Inspection frequencies in Table 7 can be relaxed for severity ratings C 'moderate' and D 'mild', once trends for that structure have been established and moisture conditions are stable. This is because long term monitoring in most instances show that crack width growth is slow and linear with time, so a 0.3mm crack on a 20 year old structure will need another 20 years to reach 0.6mm width.

3.5 Containment of Expansions by Reinforcement

3.5.1 The effectiveness of containment by well anchored reinforcement in 3 dimensions, Class 1 in Section 8, has been confirmed in many shear and flexural tests. However for Class 3 details where there is no through thickness reinforcement there is now more evidence of delamination and loss of anchorage developing with 'Severe' AAR. Some Dutch bridges of this type [20] have been demolished and sections tested. This has shown a significant loss of shear strength. This is of greater concern in structures designed to old shear design and detailing rules which have low safety factors when assessed to current standards.

3.5.2 The Section 8.1 of the 1992 guide with Figures 14 and 15 merits further development to cover a wider range of sensitive details. For example laps at the bottom of cantilever retaining walls are particularly vulnerable to delamination. Loads on them increase from passive pressures developed as the wall curves pressing back against the soil, due to expansion of the outer compression face relative to that of the tension face, where reinforcement limits expansion.

3.6 Steel fracture.

3.6.1 In Japan over 40 bridges [21] have developed problems on crossheads and foundations, where reinforcing steel of a type which becomes brittle on bends, has been used. The forces from AAR expansions have fractured the bend reducing anchorage and containment against delamination. This has necessitated substantial remedial works. This seems to be a particular metallurgical problem with Japanese reinforcement.

3.7 Expansion to date.

3.7.1 Estimates of the ‘expansion to date’ and ‘potential for further expansion’ in different structural elements are essential for assessment of structures with AAR. The crack summation procedures for estimating expansion to date work well in directions where there is little restraint from structural stress, reinforcement or prestress. Further research data to help in making adjustments for the effects of compressive or tensile stress state are now available. It is also necessary to adjust for other causes of cracking.

3.7.2 There were indications when the 1992 edition was prepared that the substantial fall in Young’s modulus (E) of concrete and the development of hysteresis found in testing cores using the low stress range Stiffness Damage Test (SDT) [22] gave a useful measure of the degree of microcracking from AAR in the sample.

3.7.3 This has now been confirmed internationally in a number of research programmes and there is a good correlation with petrographic evaluation. The changes in E and hysteresis have now been calibrated to estimate the ‘expansion to date’ in the sample. It is of particularly value in evaluating the condition of bridge decks where internal damage occurs through the depth but visible cracking is suppressed by heavy reinforcement. Similarly in foundations, the damage can be evaluated by coring down rather than full excavation. As the SDT has a low stress range (0.5 to 5.5MPa) it does not alter the sample. Therefore, the SDT can be used to evaluate ‘expansion to date’ on all cores prior to strength, expansion or analytical tests.

3.8 Expansion testing of cores.

3.8.1 Expansion testing of cores provides a basis for estimating the ‘potential for further expansion’. Difficulties have arisen with many of the procedures for testing expansion of cores from structures and with testing cast samples. Many of the methods are summarised in Tables A1(a) and (b) in the 1992 guide, but variants have been developed over the years

3.8.2 Inter-laboratory trials have highlighted problems arising from leaching of alkali from the sample, variation in moisture conditions and the difficulties of accelerating test procedures at higher temperature without distorting the results. This is highlighted by comparison of accelerated tests on prisms at BRE [18] which typically expanded 3.5mm/m and stabilised while the corresponding large block exposed externally developed very severe cracking from 15mm/m expansion still continuing after 16 years.

3.8.3 The very high variability of the reaction and expansion within cores and between cores, due to the variations in composition, adds to the difficulties of interpretation. Because of this variability, which is the root cause of AAR cracking from differences in expansion, it is necessary to use large batches of samples to obtain the average behaviour and to quantify the variability of expansions. Tests over a few months on two or three samples will almost always give misleading results.

3.8.4 The laboratory ambient, 20 to 25°C, Type a) capillary water supply test, with moisture weight changes monitored, is simple and reliable for gauging potential further expansion, but it is slow. It accelerates by a factor of about 10 relative to the site expansions. Normally it is used over a period of at least 2 years in parallel with monitoring of cracks and/or overall expansion on the structure.

3.8.5 RILEM is collating results from a range of expansion test procedures in ongoing inter-laboratory trials and will be publishing the results and, in due course, it will make recommendations.

3.9 Modelling

3.9.1 In initial assessment, finite element modelling is not yet appropriate. Once severe AAR with structural sensitivity is established, the growing research literature on modelling can be used to guide detailed appraisals of critical parts. RILEM is developing guidance.

3.10 Interaction of AAR Cracking and Microcracking with Frost and Corrosion.

3.10.1 AAR interacts with other deterioration processes. The developing cracking and microcracking opens up the structure making it more vulnerable to water and Chloride ingress and frost damage. The water and Na from salts aggravates the AAR.

3.11 Delayed Ettringite Formation (DEF).

3.11.1 DEF is an expansive reaction in the cement phase of concrete and can cause long term development of cracking similar to that from AAR This can occur when early age temperatures over 70°C give rise to monosulphates, which in time convert to larger ettringite crystals. Petrography will clearly differentiate DEF from AAR. Sometimes DEF and AAR interact and LCPC in France are studying this on structures and in the laboratory.

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Foreword

This document updates advice given in the Institution report *Structural effects of alkali-silica reaction, interim technical guidance on appraisal of existing structures* published in December 1988. The Task Group has taken advantage of new data derived from, inter-alia, the following:

- work carried out by UK Universities, Polytechnics and other research organisations
- papers presented at the 8th International Alkali-Aggregate Reaction Conference in Kyoto in 1989 and other similar international seminars
- additional field experience of the problem.

The international dimensions of the problem, now reported to have occurred in over 35 countries, have been recognised and the Task Group now includes corresponding members from Denmark, Japan, South Africa and the U.S.A. Their contributions have been invaluable.

Communication with our sister committees has been improved by also inviting Michael Hawkins, Chairman of the Concrete Society Alkali-Aggregate Working Party, and Dennis Palmer, Chairman of the British Cement Association (BCA) Working Party on the Diagnosis of ASR, to be corresponding members . In this way the Group has sought to bring together slowly converging but still disparate views on a complex phenomenon. Where specific data have not been available, the Group has used engineering judgement in making its' recommendations. Additional research (see Chapter 11) may suggest further revisions to this report.

Before commencing work on the *Interim Guidance* arrangements were made for the Building Research Establishment (BRE) to establish a confidential data base of case histories. The initial response was encouraging but, in spite of renewed publicity, enthusiasm has waned. If this report in any way falls short of the Group's high expectations it will, in some measure, be due to the reluctance of some structure owners to release information. This is part of a general malaise in the construction industry by which vital data are denied to those who could influence the performance of structures. This problem must continue to be addressed.

I would like to pay tribute to the hard work and professionalism of the Task Group - particularly at a time of economic uncertainty. To Andy Lorans, Secretary to the Task Group, who has cheerfully kept us on the road to our goal go my thanks. I would also like to thank those organisations, such as the Department of Transport (DTp) and others, without whose data we could not have accomplished our work.

Constructive suggestions on this report would be welcomed by the Institution and should be addressed to the Director of Engineering.

David Doran
Chairman, Task Group
July 1992

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1 Introduction

1.1 Structure of report

This report is divided into Part I on the process of alkali-silica reaction (ASR) and its effects, Part II covering the appraisal of specific structures and Part III on research needs.

Part I reports on the development of damage in structures in the UK and then sets out the chemical processes before considering the physical effects on concrete. The overall behaviour of structures with ASR is also considered.

Part II starts in Chapter 6 with an outline of the overall procedure for appraising an individual structure. This is developed in detail in subsequent sections on diagnosis and assessment of severity and appraisal of structural strength. Management procedures for ameliorating the rate and consequences of deterioration. Procedures for inspection are also addressed.

Part III covers the research needs the Task Group have identified. The Appendices cover summaries of test procedures and a selected bibliography.

1.2 Relationship to other UK recommendations on ASR.

An authoritative overview of ASR was given in BRE Digest 330, 1988.[1.1] The test methods for the initial diagnosis of ASR were set out in the BCA 1988 report on diagnosis which has now been revised.[1.2] For structural appraisal and the development of long-term management strategies, the testing and monitoring must concentrate on quantifying the physical effects and especially their long-term trends. In the present report on current methods reference is made to the BCA report[1.2] where appropriate. Its procedure for petrography and chemical analysis can also be used to identify the other non structural forms of cracking. Several new test methods for the effects of ASR have been developed. These are referred to in the present document.

Engineers must consider the need to minimise the risk of damage caused by ASR in new construction. The Concrete Society's recommendations[1.3] for the specification of concrete and the DTp specifications[1.4,1.5,] for structures, such as bridges, which are particularly vulnerable to the reaction provide a basis for this in the UK.

1.3 The updated recommendations

This report updates the 1988 IStructE report *Structural effects of alkali-silica reaction, interim technical guidance on the appraisal of existing structures*. The principles of the interim report have been confirmed by research in the UK and internationally and by experience in the field. This provides a basis for a relaxation of the recommendations for structures with reasonable reserves of strength. In this context, good detailing, in particular a well anchored three dimensional reinforcement cage, is important. Conversely, more emphasis is put on the need for rigorous analysis, inspection and testing for full appraisal where there are inadequate reserves of strength or poor detailing. To reflect this, the structural severity rating has been revised from that given in the interim report.

There is now a better quantitative understanding of how load induced stresses and reinforcement restraint can influence expansion and cracking. The relationship between expansion and cracking and the deterioration of concrete is also better understood.

This report distinguishes between structures where ASR is of little consequence to structural safety and serviceability and those which are at risk. The vulnerability of a structure is a function of:

- structural type
- quality of detailing
- environment.

Where a structure is judged to be particularly at risk, specialist quantitative analysis of ASR effects may be appropriate.

The first priority in the study of a structure showing excessive cracking, from ASR or any other cause, must be to establish structural reserves, and the sensitivity of details, so that the detailed procedures to establish the causes and consequences may be related to those parts where the structure is most at risk. This overall appraisal of the structure should follow the Institution's appraisal procedures.[1.6] Experience shows that ASR may interact with other forms of structural and non-structural cracking. It may also be combined with corrosion and occasionally, in exposed positions, frost damage. ASR tends to be associated with high cement content, so cracking from early thermal effects and shrinkage are often found to have interacted with it. The evaluation of the causes of non-structural cracking, of which ASR may be but one of many, is well covered in Concrete Society report TR22.[1.7]

For most reinforced concrete structures, checks for ASR are required only as part of overall appraisal where signs of damage and deterioration have become apparent. The exceptions are sensitive structures that are likely to have similar cement and aggregates to those where significant damage caused by ASR has been observed.

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PART I: ASR - the process and its effects

2 The development of ASR damage in UK structures

ASR has been known internationally as a cause of cracking since the 1940s,[2.1] and was subject to detailed study by BRE[2.2] in the late 1940s and 1950s. However, no structurally significant case was diagnosed in the UK before about 1975. At that time some substantial cracking in some CEGB substation bases, structures in south west England and in a car park in Devon were diagnosed as being caused by ASR.

Since then the reaction has been diagnosed in a wide range of structures built in the 1930s through to the 1970s. Fig. 1 shows a histogram of the dates of construction of UK structures where ASR has been diagnosed and details are available from a database collated by BRE.[2.3] A number of these structures are bridges. Fig. 2 records the upper quartile of expansion of cores at a minimum of 90 days for those cases that have been studied in detail. The expansion observed in structures is much slower and may be much less than in cores (see Chapter 7). The relationship between free expansion of cores and restrained expansion in structures is covered in subsection 4.2. Table 1 is a list of types of structure affected.

A number of cases have been diagnosed in the south west of England.[2.4] The other substantial group of structures, in Britain, in which ASR has been identified, is located in the Trent Valley. Some cases have been identified in most parts of the UK, and the range of known reactive aggregate sources has substantially increased in the last five years and no area can be considered risk free. Precast concrete elements and some cements and aggregates used for special architectural finishes are sometimes transported hundreds of miles and have produced cases of ASR in some unexpected places.

Some hundreds of structures have clear signs of damage from ASR and the number of reported cases continues to rise. However, structurally damaging cases of ASR with significant cracking are a small proportion of structures in which some ASR could be identified by rigorous laboratory tests. These in turn are a small proportion of the total amount of concrete construction.

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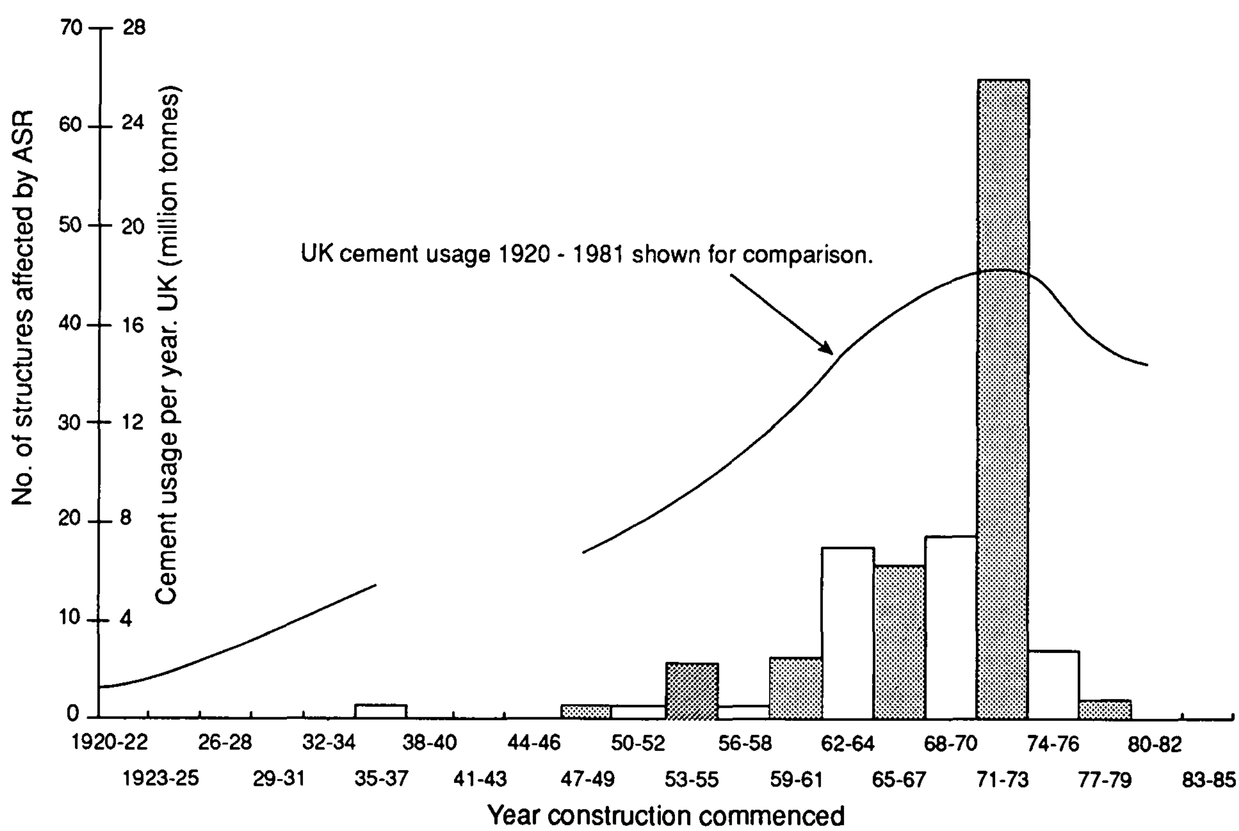

Fig. 1 Date of construction of 134 UK structures affected by ASR reported to the BRE, and cement usage.

Table 1 Types of structure affected by ASR in the UK

types	*examples*
bridges	reinforced and precast concrete, including prestressed elements
hydraulic structures	dams, reservoirs, river walls, sewage works
buildings	dwellings, offices, shops, hospitals, schools
open frames and foundation structures	car parks, grandstands, foundations, retaining walls, piles and substation bases

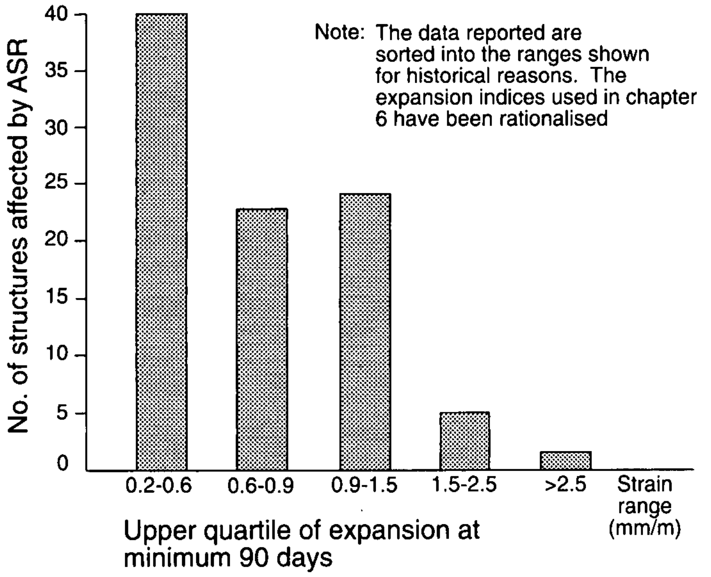

Fig. 2 Unrestrained expansion of cores taken from 93 UK structures affected by ASR indicating the range of severity.

3 The chemical processes of ASR

3.1 Alkali-silica reactions

For an overall view of alkali-aggregate reactions the reader is referred to BRE Digest 330.[3.1] The information below is included for completeness. ASR is the predominant form of alkali-aggregate reaction, but there are two other potentially damaging forms: alkali-carbonate reaction, in which certain dolomitic limestones react, and alkali-silicate reaction in which layered silicate rocks react. No significant case of alkali-carbonate reaction has so far been found in the UK, but some cases of alkali-silicate reaction have been identified.[3.2] These reactions are not covered in this report.

ASR is a chemical process in which alkalis, usually predominantly from the cement, combine with certain types of silica in the aggregate, when moisture is present. This reaction produces an alkali-silica gel that can absorb water and expand to cause cracking and disruption of the concrete.

For ASR and damaging expansion of the resulting gel to occur in hardened concrete, it is necessary to have:

- a sufficiently strong alkaline pore solution
- a proportion of reactive silica in the aggregate, lying within the sensitive range
- sufficient moisture in the concrete.

Signs of ASR can often be found in sound concrete. Only in conditions with high alkalinity, a proportion of reactive silica within the sensitive range and high moisture availability will the expansion of the gel cause significant damage and change the physical characteristics of concrete.

3.2 Alkalis in concrete

The cement in concrete creates an alkaline environment, which protects the reinforcement steel from corrosion. Some of the alkalis are concentrated in the free water in the pores of the concrete and this maintains a high *pH* .

Calcium hydroxide (lime), common to all cement, will on its own produce a *pH* of about 12.5, which is sufficient to prevent corrosion in the absence of chlorides. Sodium and potassium hydroxides raise the *pH* to over 13 and *pH* can approach 14 with the highest alkali cements. It is a convention of cement chemists to express the sodium and potassium concentration in cement in terms of the oxides, that is Na_2O and K_2O. In relation to alkali-silica reaction these are normally combined in terms of the 'equivalent' sodium oxide on the basis of their molecular weights (Na_2O equivalent = $Na_2O + 0.658\ K_2O$).

Most UK produced cement now has a monthly average Na_2O equivalent in the range 0.5% to 1% by weight of cement. However, some cement works in the past have produced cement with Na_2O equivalents of 1.2% annual average[3.3] and over 1.4% on a daily basis. 'High' alkali cements do not provide significantly better resistance to corrosion than 'low' alkali cements, as 'high' and 'low' relate to the Na_2O equivalent, not to the $Ca(OH)_2$ hydration product which dominates the corrosion resistance.

The cement is, however, not the only potential source of alkalis in concrete. Admixtures, and sodium chloride from poorly washed sea-dredged aggregates can contribute significantly to alkalinity. Alkali can also be slowly released from some types of aggregate containing feldspars, some micas, and glassy rocks and glass. In the UK it is as yet unclear whether with currently exploited aggregates this can be a significant additional source of alkalinity. De-icing salts applied to concrete can locally increase the alkali content in the surface layers.

Ground granulated blast furnace slag (ggbs) or pulverised fuel ash (pfa), when used as part of the cementitious material, contribute alkalis to the pore solution, but also absorb alkalis into their hydrates. Overall they lower the *pH* of the pore solution and reduce the risk of reaction. There are, however, isolated overseas cases of ASR damage where low levels of cement replacement material have been used.[3.4]

The amount of alkali in concrete depends not only on the concentration of alkali in the cement but also on the total amount of cementitious material in the mix. For particular aggregate types and cementitious materials a broad relation exists between the total amount of alkali in the concrete and the risk of damaging ASR occurring. The most serious cases of ASR damage in the UK generally relate to mixes which had more than 4.5 kg of alkali per cubic metre of concrete. These high alkali contents are associated with mixes with high cement content between 450kg/m^3 - 550 kg/m^3 and/or high alkali cements (1.0% - 1.4% Na_2O equivalent).

Thus the alkali may originate from cement, pfa or ggbfs and admixtures, water, sodium chloride in aggregates and other soluble sodium and potassium in aggregates. When the total alkali is less than 3 kg/m^3, using UK natural aggregates is unlikely to cause damaging expansion. This is true providing the aggregate is free of opal and there is no evidence of strong alkali migration with consequential local concentration. Moisture movement, as identified by Nixon and Gillson,[3.5] can however concentrate alkalis to initiate reaction in parts of concrete pours with otherwise low average alkali levels. Alkalis can also be leached out of concrete immersed in water. Normal concretes with a Na_2O equivalent of low alkali cement below 0.6% and no other source of alkali are not known to have caused ASR expansion with UK natural aggregates. Opals, and glass used as a decorative aggregate, will react at a lower concentration.

3.3 Reactive silica

Silica is found in many geological deposits in a wide range of crystal structure and grain size. Only silica with disordered structure and/or fine particle size reacts with alkali in concrete to a significant extent. Opal, which has a disordered structure, is highly reactive and can cause damage even in concentrations as low as 1% or 2% of the mix.

The type, particle size and proportion of silica in the aggregate will influence the rate and severity of the reactivity of the concrete. The severity of the expansive ASR will increase as the proportion of reactive silica in the total amount of aggregate in the mix increases up to the 'pessimum'. The pessimum is the proportion of reactive aggregate that gives the most adverse effect. With an increasing proportion above the pessimum, the concentration of hydroxide in solution becomes insufficient to maintain the same degree of attack, and the expansion decreases again. The sensitive range of silica is the proportion on either side of the pessimum within which damage and expansion may arise.

Laboratory experiments and observations of concrete affected by ASR have shown that for flint or chert the greatest damage results when less than half the aggregate consists of these materials, but the exact proportion varies with the type and grading of the flint or chert. Significant damage can however occur with 5% of chert. In concrete containing the relatively chert-rich sand dredged off the south coast of England, ASR damage has been observed when it has been used with an inert coarse limestone or granite aggregate. A similar combination of Thames Valley fine aggregate with coarse limestone and high alkali contents has been used to produce ASR damage for the BRE, SERC and TRRL test programmes. With Trent Valley

Table 2 Some examples of reactive aggregate combinations in UK concretes with ASR

coarse aggregate	*fine aggregate*	*structure locations*	*reactive minerals*
limestone (inert) or granite	south coast sea dredged	Devon/Cornwall & Hampshire	chert in fines
Trent Valley gravel	Trent Valley gravel	Midlands	chert and quartzite
granite (inert)	Essex sand, quartz sand with chert	Midlands	chert
limestone, manufactured glass & chert	limestone, manufactured glass & chert	NW England	manufactured glass
limestone	limestone	North England	fine siliceous impurity
flint	quartz sand & siliceous limestone	SE England	siliceous limestone
greywacke sandstones & argillites	natural sand mainly quartz & quartzite	Wales & SW England	greywacke/argillite
meta-argillite	meta-argillite	North England	meta-argillite

Note: This is not intended to be a comprehensive list; many other combinations of aggregates that are reactive may be found to exist in the UK. World-wide there are examples of most rock types reacting due to relatively small proportions of reactive minerals.

aggregates, which contain only a small proportion of chert, some damage is observed when both coarse and fine Trent Valley aggregate are used together. The minerals and aggregates in many UK cases of ASR are varied and contain potentially reactive minerals such as cherts and quartzites. An example of this is found in Trent Valley gravel where cherts and quartzites co-exist. The interactions between minerals and their effect on the pessimum have yet to be quantified in these mixed materials.

The grading, microstructure and proportions of reactive silica particles in aggregates vary widely between known reactive concretes world-wide. This produces substantial differences in the physical effects of the reaction. Generalized statements on the time scale and physical effects of the reaction must therefore be treated with caution. The characteristics of the alkali-silica gel formed by the reaction vary with its chemical composition, temperature, moisture content and pressure. Its consistency can range from that of heavy engine oil to that of polyethylene. Some aggregates, e.g. Danish flints, Beltane opal, generate sufficient quantities of gel for it to exude from cracks. Conversely, in most UK cases of ASR, gel is visible only when cores are petrographically examined.

The main currently known UK reactive aggregate combinations are listed in Table 2. Polished sections of ASR damaged concretes are held by BRE, Queen Mary and Westfield College and BCA.

3.4 Water

The alkali-silica gel may form in dry conditions, but it will expand and damage the concrete only if there is a sufficient supply of water. Laboratory experiments show that with concrete kept consistently in an environment below about 75% relative humidity (r.h.)* expansion will be insignificant. ASR damage is usually found in concrete exposed to the weather, in contact with or buried in the ground, immersed or partly immersed in water and concrete subject to heavy condensation. It may also occur in interior mass or sealed concrete which can expand due to the presence of residual mix water, particularly where high water-cement ratio is used with high alkali cement.

*Strictly this is the equilibrium relative humidity within the concrete, i.e. the humidity level of air in vapour pressure equilibrium in the concrete.

A reduced water supply may curtail the reaction and will halt the expansion. However, this will redevelop rapidly when the concrete is subsequently wetted again. Such rewetting may occur as a result of change of use of the structure or may be intentionally induced in an expansion test, see section 4.1. There are examples of rapidly developed damage following leakage through deteriorated waterproofing in a building.

Field and laboratory measurements of temperature, humidity, expansion and crack growth on experimental samples and on exposed structures suggest that at a r.h. consistently below 80% the movement is very slow. It starts to develop clearly in the 85% - 90% r.h. range and is most rapid in the 90% - 100% r.h. range. Concrete immersed in water may lose sufficient alkali by leaching to reduce expansion potential. In most areas of the UK the climate provides a relative humidity of 85%, or more, for most nights and for long periods in winter.

Tests may, in time, establish a basis for directly predicting field performance from expansion tests on cores. The size and low initial permeability of concrete in many structures is such that the rate of moisture migration into the concrete will control the rate of expansion. The expansion is generally substantially slower in structures than in test samples.

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4 Physical effects on concrete of ASR expansion

4.1 Local effects

ASR in a concrete is not significant unless it has produced or is likely to produce substantial expansion, cracking, loss of strength or threatens serviceability. Many concretes exhibit harmless signs of the reaction under petrographic analysis.

The swelling pressure exerted by the gel as it absorbs moisture sets up tensile stress locally in the surrounding concrete which may lead to micro-cracking. The pressure is reduced as the gel permeates into the surrounding sand/cement matrix or porous aggregate. Micro-cracking can be examined petrographically and its nature depends on the size, spacing and porosity of the reacting particles. Micro-cracking reduces the stiffness, ultrasonic pulse velocity (UPV) and strength of concrete as discussed in section 4.4. The gel formation and expansion rate, viscosity and volume changes vary with the form of silica, alkali and conditions of temperature and moisture.

Experimental determinations of the pressure from the swelling of pore gels have been performed[4.1,4.2] but the data do not provide a good measure of the stress generated. Alternative estimates of this stress may be obtained from restrained expansion testing of concrete specimens and strain gauging of reinforced concrete specimens.[4.3,4.4,4.5]

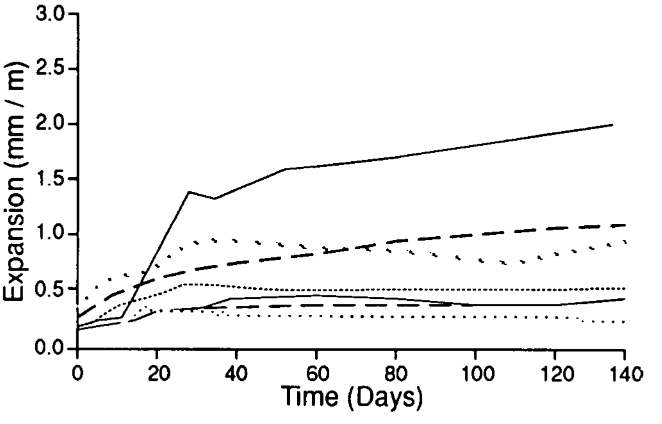

Fig. 3a Variations in overall mean expansions on 7 cores from the same slab pour

4.2 Expansion

When the reaction develops at a sufficient number of points within the concrete an overall expansion occurs. The intensity of the reaction will be variable both in terms of distribution in the body of the concrete and in the time scale of development. This results in the variation in expansion as shown in Figs 3(a) -3(c). Reasons for this variation include:

- the cement and aggregates may have been supplied from a number of sources for different pours of concrete
- cement content and alkali level vary both between and within pours
- type and quantity of reactive materials may differ
- variation in batching, mixing and placing affecting concrete permeability, void ratios and local aggregate-cement ratio
- differing exposure environment such as moisture, drying wind and sun etc., leading to variation in moisture content and temperature
- moisture and alkali migration may result in alkali concentration or reduction due to leaching from the surface
- variation in reinforcement density, structural loading or external restraint modifying what would otherwise be unrestrained three-dimensional expansion.

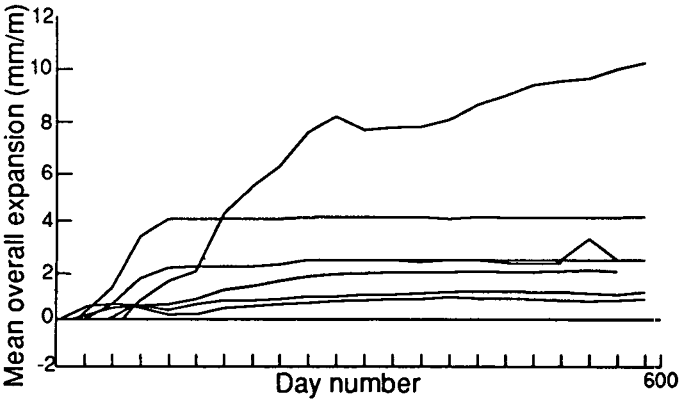

Fig. 3b Variability of cores from the same structure from7 different pours

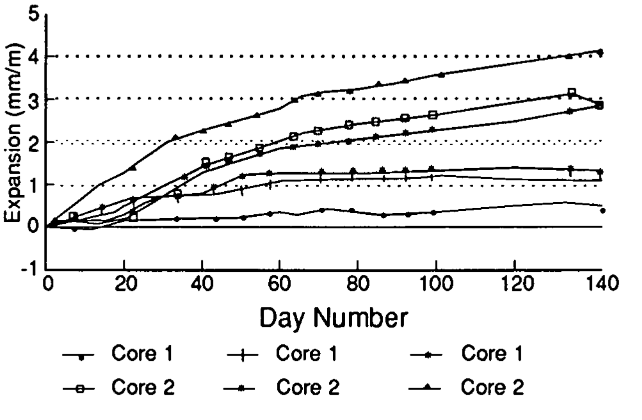

Fig. 3c Variability of expansion within cores

Differential expansion may cause cracks between expanding and non-expanding parts. Cracking of the outer layers may result where there is differential expansion between heart and surface concrete. This cracking is typically the first indication of damaging ASR. Work is currently being undertaken[4.6] to investigate the relationship between expansion and cracking, changes in stiffness, and the stress/strain curve in compression and ultrasonic pulse velocity (UPV). These in combination provide a basis for estimating the expansion.

The expansion of the concrete depends on the availability of water for absorption into the alkali silica gel. This is governed by the penetration of water into the concrete and is subject to seasonal variations. Changes in use or defective drainage can increase water availability and expansion rate dramatically. The reaction and expansion will not normally be fully completed.

Estimates must therefore be based on substantial completion of expansion, say 90%.

A measure of the potential for further unrestrained expansion can be obtained by subjecting cores extracted from the structure to controlled damp conditions. (See appendix A for test procedures). As moisture is absorbed concrete swells, typically by 0.2 mm/m, due to the recovery of drying shrinkage. In very severe cases of ASR, further expansion, typically 3 mm/m, may be expected. The expansion of a core is dependent on moisture, previous expansion, curing history and the restraints that were experienced by the core in the structure.

4.3 Cracking

Expansion is not uniform throughout the volume of the concrete. It is greater in the immediate vicinity of each reactive particle or around a cluster of such particles. The resulting differential effects can result in micro-cracking. Furthermore the micro-cracking is not uniformly distributed because it is influenced by restraints and effects at the edges of the concrete mass (see Fig. 4).

The micro-cracks within a mass of unrestrained concrete are orientated randomly. In the surface layer the degree of reactivity may be changed due to leaching of alkalis by water [4.7, 4.8] or by a reduction in alkalinity due to precipitation of sodium and potassium carbonates.[4.9] Greater porosity in the outer layer may also lead to less expansion as gel may permeate into the more porous concrete.[4.7] The combination of the variability of expansion and greater expansion of the interior concrete results in tensile strain at the surface, which can develop into macro-cracking.

In unrestrained concrete the pattern of macro-cracks is an irregular one of intersecting and bifurcating cracks which is often referred to as map cracking or 'Isle of Man' cracking. It should be noted that this pattern is also observed where differential expansion or shrinkage has occurred, and on surfaces subjected to rapid cooling, for example, when formwork on mass concrete is struck too early. In the presence of restraint to expansion, the macro-cracks will tend to be parallel to the direction of the restraint, as explained in Chapter 5.

The depth of macro-cracks does not usually exceed the lesser of the cover and about one-tenth of the member thickness,[4.10] or is also crudely related to the width at the surface. Fig. 5 shows data from both Japanese and U.K. observations. The sectioning of members from demolished structures has shown that reinforcement effectively checks the surface crack propagation and a single surface crack spreads out into branching finer cracks and then merges into microcracking. The restraint from the surface layer of steel tends to concentrate micro cracking in its plane. This can develop into more severe delamination cracking. With high expansions this can lead to debonding and the development of stepped cracks on the end faces of members.

4.4 Changes in physical properties

Physical properties measured on unrestrained concrete specimens generally show a reduction in compressive strength, tensile strength, elastic modulus and UPV relative to their 28 day values.

The measurement of UPV is sometimes a useful non-destructive testing method. Changes in pulse velocity in concrete with ASR, calibrated relative to cores from the structure, can indicate different degrees of deterioration as outlined in Table 3. However, the pulse can be short circuited by reinforcement so that the procedure is not readily applicable to reinforced elements. The inherent scatter of UPV readings also renders it insensitive to changes in concrete condition. While pulse velocity falls rapidly in the early stages of ASR expansion it changes relatively little as further expansion develops.

The UPV may be markedly reduced by micro-cracking. Even small expansions can cause a decrease in pulse velocity to well below that expected in normal concrete. However, where cracks are filled with gel and/or water, the UPV may not be reduced.

Table 3 Typical pulse velocities related to concrete condition

Pulse velocity	*Concrete condition*
4.5-4.8 km/s	Normal
4.2-4.5 km/s	Some damage
3.9-4.2 km/s	Substantial damage
less than 3.9 km/s	Severe damage

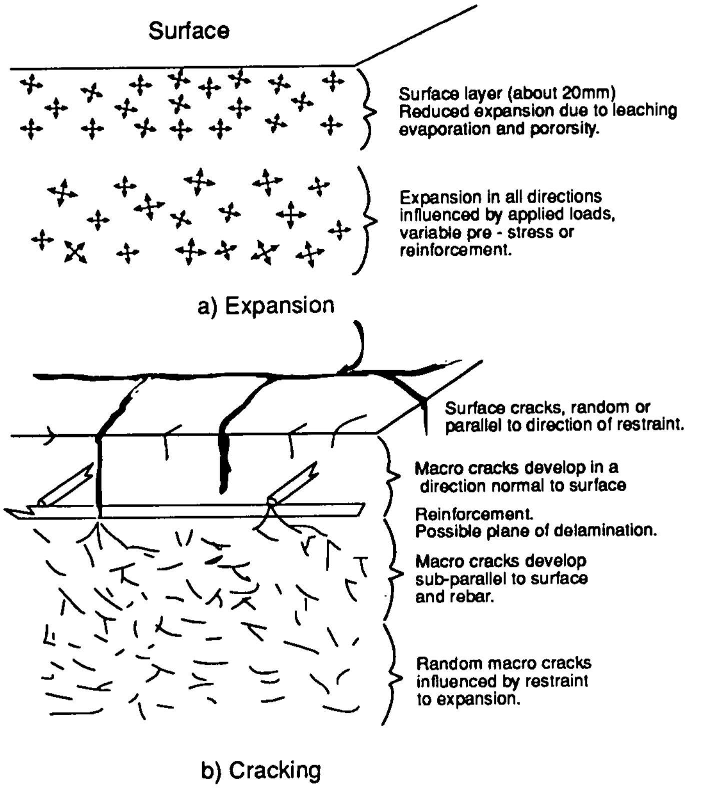

Fig. 4 Development of cracking due to ASR

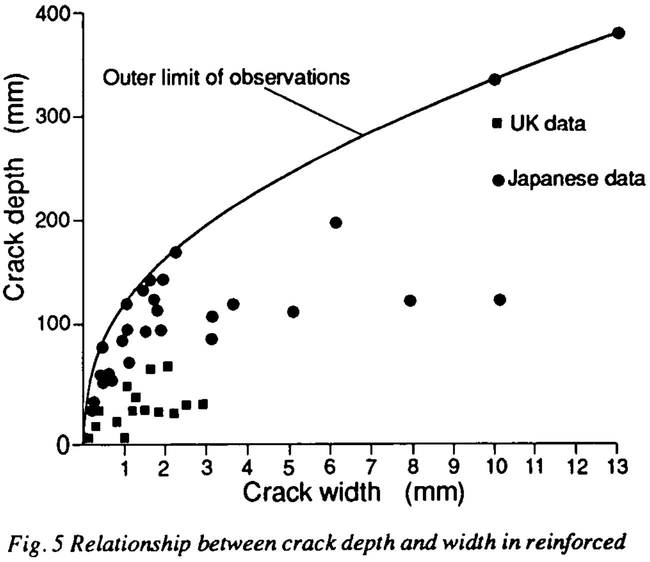

Fig. 5 Relationship between crack depth and width in reinforced concrete

Table 4 Lower bound residual mechanical properties as percentage of values for unaffected concrete at 28 days

Property	*Percentage strength as compared with unaffected concrete for various amounts of free expansion*				
	0.5 mm/m	1.0 mm/m	2.5 mm/m	5.0 mm/m	10.0 mm/m
Cube compression	100	85	80	75	70
Uniaxial compression	95	80	60	60	*
Tension	85	75	55	40	*
Elastic modulus	100	70	50	35	30

Indeed, after an initial decrease of UPV due to ASR cracking, there can be a substantial recovery as cracks become filled with gel.[4.8]

Table 4 indicates lower bounds to the residual mechanical properties of unrestrained concrete for various ASR free expansions as percentages of those actual properties of unaffected concrete at 28 days. The tabulated values were derived from lower bounds to laboratory data obtained from tests on cast cubes, cylinders, prisms, and from tests on cores extracted from structures. The data are those summarised in a literature review carried out by Clark,[4.11] augmented by additional data subsequently published[4.8,4.12] and by unpublished core data supplied to the Task Group. It should be noted that the uniaxial compressive strength as obtained from a long cylinder or core test is reduced by ASR to a greater extent than is the cube strength. It is the uniaxial compressive strength which is required for structural assessment. The residual tensile strength is affected by the test method. The values quoted in Table 4 are appropriate to the splitting tension or torsional tension strength.

It is emphasised that the residual strengths and stiffnesses in actual structures will be modified from the figures in Table 4. This is because the concrete in actual structures is generally restrained by adjacent material and is in a biaxial or triaxial stress state. These effects will tend to reduce the damage to the concrete and increase its residual mechanical properties.

The actual 28 day strength of concrete is often in excess of its design value by an amount which is greater than any subsequent reduction due to ASR. Hence, compressive strength reduction is not normally a problem in practice. However, tensile strength reduction may require special consideration.

4.5 Time-scale of the development of ASR damage

Observations of structures cracked due to ASR indicate that cracking is first discernable 1 to 5 years after construction. The time taken for expansion to reach substantial completion depends on the source of alkalis, the aggregate type and the moisture supply to the concrete as follows:

- for concrete in which the alkalis come primarily from the portland cement, as is usual in the U.K., substantial expansion is often incomplete after 10 years
- there is virtually no upper limit to the time over which expansion can occur in concrete containing an excess of reactive silica exposed to significant quantities of external alkalis, such as deicing salts and industrial chemicals
- for concrete in which a significant quantity of alkali is contributed by aggregates of volcanic origin, such as rhyolite, andesite or artificial glass, it appears that expansion can continue for 30 to 60 years
- the damage may not start in some cases until alkali migration has resulted in a sufficient concentration of alkali and the time to reach substantial completion will be in excess of the examples quoted above
- the onset of the damage may be deferred for structures subject to change of use or to drainage faults.

Experience of similar structures in comparable environments with similar cement and reactive minerals can provide guidance. Only quantitative monitoring of the actual structure as discussed in Chapter 10 will over the years reveal its pattern of deterioration rate.

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5 The effects of ASR on concrete structures

5.1 General

The more usual physical effects of differential expansion due to ASR take the following forms:

- internal microcracking associated with individual expansive particles
- surface macro-cracking
- overall dimensional changes
- induced tensile stresses in reinforcement and compressive stresses in concrete
- induced bond stresses between steel and concrete
- differential movements between separate pours of concrete expanding at different rates.

All these effects can be modified by the restraints as discussed in Chapter 4 and are considered in more detail in the following sections.

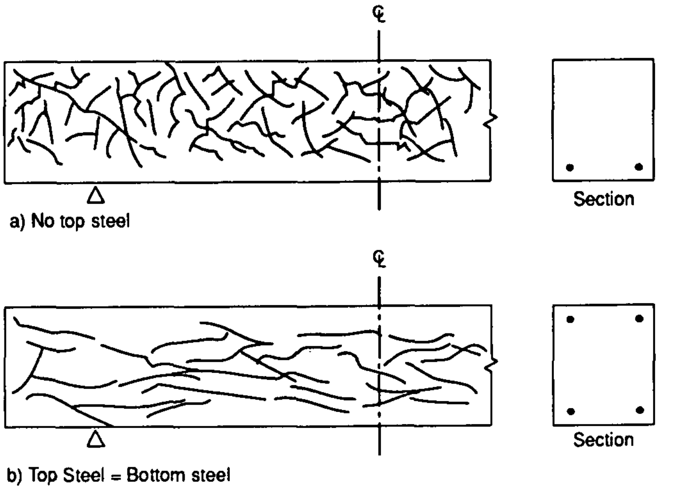

Fig. 6 Influence of reinforcement on ASR cracking

5.2 Influence of restraints

If concrete is restrained by surrounding non-reactive concrete, applied stress or reinforcement, the expansion is inhibited in the direction of restraint, and the dominant cracks form parallel to this direction.

The restraining influence of reinforcement is illustrated in Fig.6, which shows the crack patterns due to ASR on two beams.[5.1] During the development of cracking, the beams carried their self-weight. The upper beam had only bottom reinforcement, and map cracking can be seen in the unreinforced, hence unrestrained, top region of the beam. In contrast, the lower beam had equal top and bottom reinforcement which uniformly restrained the longitudinal expansion of the beam. The ASR cracks are parallel to the main reinforcement. Although vertical links were provided in these beams these were insufficient to prevent vertical expansion due to ASR. The non-uniform restraints to the expansion of either a singly reinforced beam or a doubly reinforced beam with unequal top and bottom reinforcement also result in a hogging curvature of such a beam.

The effects of reinforcement restraint in terms of expansion and induced stresses are:

- the net expansion is reduced from the free (unrestrained) value
- the reinforcement is stressed in tension
- the concrete is stressed in compression parallel to the reinforcement

The available test data[5.2,5.3,5.4,5.5] are collated in Fig.7. It can be seen from Fig.7 (a) that even a comparatively small amount of reinforcement significantly restrains expansion, but the scatter of data is wide. Restraint also delays the start and slows the rate of expansion.

From Fig.7 (b) it can be seen that the compressive stress induced in the concrete by the restraint increases as the expansion rate increases. Hence, stresses in actual structures are likely to be smaller than those in specimens accelerated in a laboratory. Excluding rapid laboratory tests the induced compressive stress tends to a limiting value of not greater than about 4 N/mm^2. In practical terms this implies that reinforcement yield could occur, with those concretes with an expansion sufficient to induce yield strain, if the steel percentage is less than about 1.6% or 0.9% for mild and high yield steel respectively. As noted in Chapter 11 on Research needs, more data on this are required and specific testing on this may be required for severely affected structures.

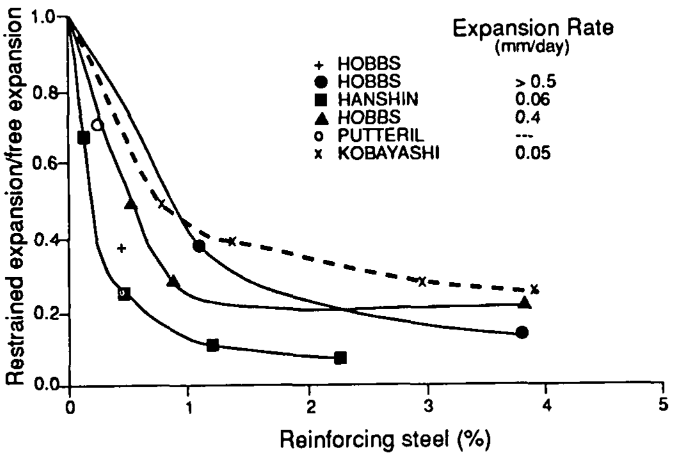

Fig. 7(a) Restrained expansion

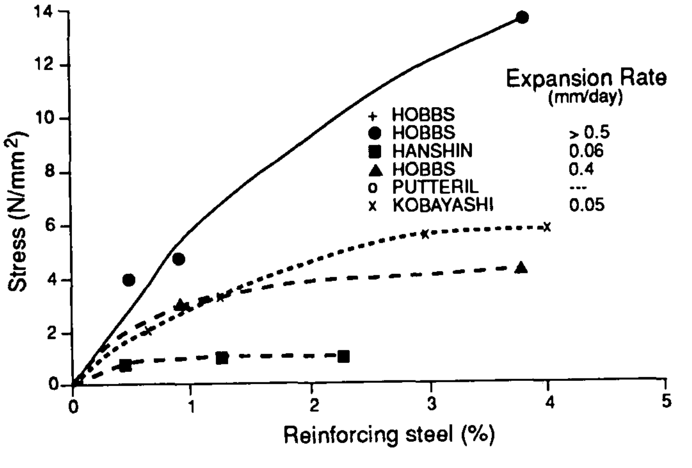

Fig. 7(b) Concrete compressive stress

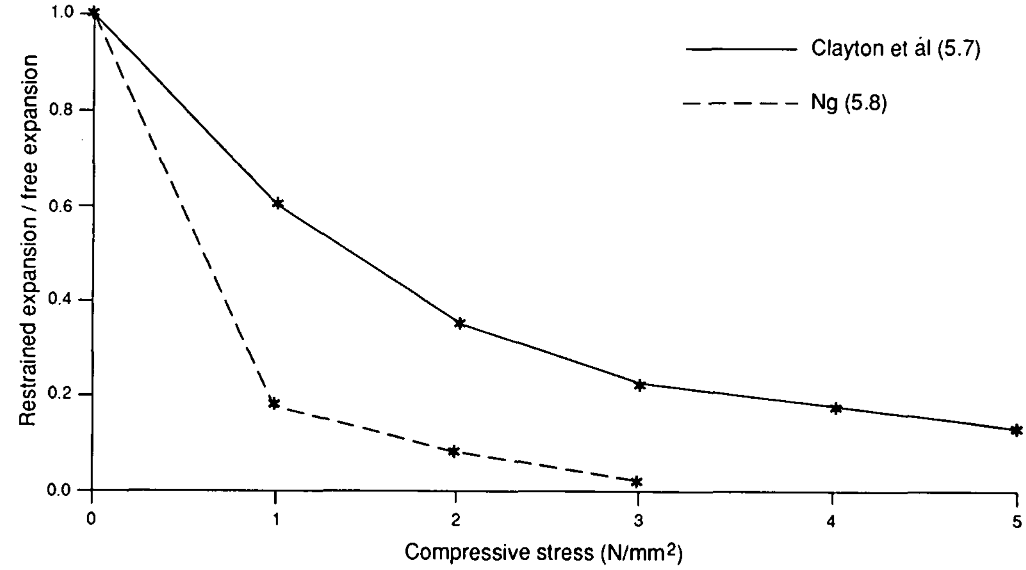

Fig. 8 Effect of compressive stress on expansion

The more heavily a section is reinforced, the smaller is the induced strain in the steel arising from expansion of the concrete. Hence, strains in shear links tend to be greater than strains in main longitudinal reinforcement. With more expansive concrete, local yield of reinforcement can arise from ASR effects alone. However, such reinforcement yield is not detrimental to the ultimate capacity provided that the reinforcement is well anchored by hooks and bends. Lack of anchorage at ends of bars can become a problem. Examples of details which are sensitive to the effects of ASR are discussed in Chapter 8. It should also be remembered that if a bar has yielded due to the effects of ASR its bond strength should be checked at its upper bound characteristic yield strength and not at its design stress of 0.87 times the characteristic yield strength. Baker[5.6] has studied the variability of the strength of reinforcing bars.

External restraint arising from applied stress has a similar effect to internal reinforcement restraint. Fig. 8 shows relevant data [5.7, 5.8] obtained from specimens under constant compressive stress. It can be seen that there are considerable differences between the two sets of data, for which the reasons are not clear. However, it is apparent that applied stress does have a significant effect on expansion. Hence, external compressive stress restrains the expansion and aligns the ASR cracking parallel to the direction of the applied stress. Fig.9 shows the longitudinal cracking that developed in a circular column. Transverse cracking was prevented by a combination of dead-load stress and reinforcement restraint. After installing alternative supports this column was removed, and the release of dead load allowed transverse cracks to develop as well as bond failures on the cut ends of the main steel.

Although the effects of internal and external restraint are understood qualitatively, it is not possible, at present, to accurately predict these effects quantitatively. The relationship between restrained expansion, free expansion, creep, reinforcement ratio, and applied stress is not yet sufficiently well understood.[5.9,5.10] The test data on the effects of triaxial restraint conflict. Some data indicate that expansion is not affected by restraint in a direction at right angles, whilst other data give the opposite indication.

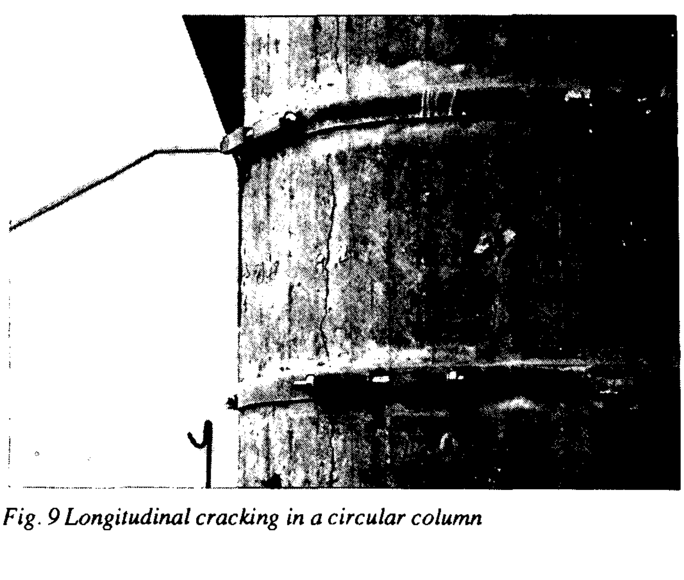

Fig. 9 Longitudinal cracking in a circular column

Although a compressive stress reduces expansion in the direction of the stress, it should be noted that a tensile stress increases expansion. However, few data are available.

5.3 Effects on member strength

The literature relating to the effects of ASR on structures published up to 1989 has been critically reviewed by Clark.[5.11] Since 1989 there has been further relevant work published at the 1989 AAR Conference in Kyoto[5.12] and publications by Clayton et al,[5.7] Chana,[5.10,5.13,5.14,5.15] Swamy,[5.16] Rogers & Tharmabala[5.17] and Cope.[5.18] The conclusions which can be drawn from recent research are summarised in the following sections.

5.3.1 Beams

ASR does not have a significant effect on the flexural strength of reinforced concrete beams provided that the free expansion does not exceed about 6mm/m. At higher expansions strength reductions of up to 25% have been observed. Flexural strength can be assessed by using the uniaxial compressive strength of the ASR affected concrete in the conventional models of behaviour with due consideration of delaminations in the structure. In U.K. codes, strength equations are written in terms of characteristic cube strength f_{cu} which, for normal concrete, is approximately $1.25 f'_c$ where f'_c is the uniaxial compressive strength. Since ASR reduces the uniaxial compressive strength at a greater rate than the cube compressive strength, it is suggested that, in the flexural strength equations, f_{cu} should be reduced to the appropriate characteristic strength of the ASR damaged concrete according to Table 4.

Tests have shown no significant decrease in shear capacity as a result of ASR if at least 0.2% links are present - indeed, an enhancement of strength of up to 47% has been observed in some tests. Included in the tests were beams with anchorages to main reinforcement as small as 3.4 times the bar diameter. This observed good behaviour in shear may arise from the self-prestressing effects of the restrained ASR expansion. It is advised that a lower bound value of the prestressing effect be used at the ultimate limit state. Doubts have been raised as to whether this prestress is maintained. However, data are now available which show that at least 40% of the ASR induced prestress is present after two years in drying conditions.[5.19] The test evidence from beams without links is conflicting with some showing an increase in shear capacity and some showing a decrease of up to 20%.

It is suggested that the shear capacity of an ASR-affected beam can be estimated by considering the influence of the compressive stress resulting from the prestressing effect of the restraint. The mechanical properties of the ASR-affected concrete should be used in making this estimate.

Repeated loading tests on reinforced concrete beams[5.20] have demonstrated that ASR does not reduce the fatigue life, probably because the reinforcement stress range is reduced by the prestressing effect of the restrained ASR expansion.

Tests on post-tensioned beams[5.21] and pre-tensioned prestressed concrete beams[5.7, 5.17] have led to similar conclusions to those for reinforced concrete beams.

5.3.2 Columns

ASR can have three effects on column behaviour:

- The concrete compressive strength can be reduced. The effects of the concrete compressive strength reduction can be allowed for by using the appropriate strength in the conventional models of behaviour, see subsection 5.3.1.
- A second effect of ASR on the behaviour of columns is that delamination can occur in the plane of the reinforcement so that the concrete cover may not be effective in resisting compression. Clark[5.11] has proposed equations for evaluating the strength of the delaminated concrete. Clark has also proposed equations to account for the reduction in strength due to the loss in buckling restraint to the main reinforcement. He confirmed that the compressive stress in the reinforcement needs to be reduced to less than its characteristic strength only if the ratio of link spacing to main bar diameter exceeds 44 and 32 for mild steel and high strength steel respectively. The effect of delamination of concrete cover should be considered; in the absence of test data, it is suggested that the cover concrete should be ignored if there is significant evidence of cover delamination. Such evidence would be longitudinal cracks wider than 0.3 mm in the vicinity of the main edge reinforcement. The reduced cross-section means that it is necessary to check the column stability.
- The compressive strain induced in the concrete as a result of the restraint to the ASR compression by the reinforcement could mean that, under load, the concrete could crush prior to the main steel yielding in compression. However, such crushing is extremely unlikely to occur in practice.[5.11]

There has been some testing of columns cracked because of ASR.[5.22, 5.23] However, the data are of limited value because the test specimens were either subjected to low axial loads with high shear or ASR was accelerated by a method which significantly distorted the concrete properties. Recent test data[5.10] indicate that ASR of up to 4 mm/m free expansion should not reduce column strength by more than the reduction in cube compressive strength, and that the above approach to assessing failure loads is reasonable.

5.3.3 Slabs

In addition to the flexural and flexural shear failure modes considered in subsection 5.3.1, a slab can fail in punching shear. Tests[5.24, 5.25] have shown no significant reduction in punching shear strength for free expansions of up to 6 mm/m. Furthermore, ASR increases the ductility of a punching shear failure. For free expansions in excess of 6 mm/m, delamination in the plane of the reinforcement effectively divides a slab into three layers if there are no links tying together the top and bottom reinforcement. In such cases strength reductions of up to 30% can result.[5.8] If there is significant surface cracking of a slab due to ASR, it is recommended that possible delamination should be checked by coring and petrographic inspection of the cores for sub-parallel cracking.

The effects of ASR on punching shear capacity can be evaluated using conventional models of behaviour for prestressed concrete slabs[5.8]. The appropriate compressive and tensile strengths of the ASR-affected concrete are used, and the precompression, due to the restraint to ASR expansion provided by the reinforcement, is allowed for by calculating the decompression load in the same way as for a conventional prestressed concrete slab.

5.3.4 Bond

Anchorage bond and lapped bar tests performed by Chana[5.13,5.14] with both plain and ribbed bars have shown that ASR up to a free expansion of 4 mm/m does not have a significant effect on either type of bar when restrained by links and/or a substantial thickness of cover concrete of the order of four times the bar diameter. However, the bond strength of bars not restrained by links and with a cover of the order of 1.5 times the bar diameter is reduced by up to 50%. The reduction is proportional to the reduction in splitting tensile strength of the ASR-affected concrete.

U.K. codes of practice give simplified models of bond strength prediction which do not take into account the important influence of the cover to bar diameter ratio. Hence to assess the effects of ASR on bond it is necessary to use more sophisticated models, such as those of Tepfers[5.26] and Reynolds.[5.27] The application of these models to ASR concrete have been discussed by Clark[5.11] and Chana,[5.13] and the following formulae are proposed for bars without links:

Ribbed bar

$$f_{bs} = \alpha(0.5 + c/d)\, f_t$$

where

f_{bs} characteristic bond strength

α a coefficient, 0.6 in general, but 0.43 for a corner or top cast bar, or 0.3 for a corner and top cast bar.

c cover

d bar diameter

f_t characteristic splitting tensile strength of ASR affected concrete

Plain bar

$$f_{bs} = \beta f_t$$

where

β a coefficient, 0.65 in general, but 0.47 for a corner or top cast bar, and 0.33 for a corner and top cast bar.

These strengths need to be related to the bond forces including those from restraint of ASR expansion. The reduced bond strength of a top cast bar occurs in all concrete and is not a consequence of ASR. Structural checks should relate to measured cover in the bond zone.

5.3.5 Torsion and bending

Test data are not available on torsion. Although the discussion of flexural shear in subsection 5.3.1 is applicable to torsion, it should be remembered that torsional shear capacity is more dependent than flexural shear capacity on the anchorage of links. This is because of the greater tendency for the cover concrete to spall under torsional shear.

No testing has been performed on the effects of ASR on bearing strength. At the edges of corbels and local bearing details the failure mode is by splitting, so that the concrete tensile strength reduction is relevant. In respect of bearing under bends in reinforcement, the tensile strength reduction is also relevant when the cover is small, say less than four times the bar diameter. In such situations, if there are no links or secondary reinforcement to restrain a splitting failure mode, allowable bearing stresses should be reduced in proportion to the reduced concrete splitting tensile strength.

5.4 Effects on serviceability

The serviceability effects of ASR are dealt with in Chapter 9

5.5 Effects of expansion on overall behaviour

The effects of ASR expansions on the complete structure have also to be considered with due regard to the restraints to expansion provided by reinforcement, adjacent less expansive pours and the restraints on the structure as a whole. Examples of effects which can occur are:

- a member expanding in a framed structure imposing lateral forces on other members
- bending moments induced in a frame as a result of the curvature of non-symmetrically reinforced members
- increased punching shear on a bridge deck or floor slab arising from a single bridge pier or column expanding
- failure of expansion joints and adjacent structural elements when the ASR expansion has exceeded the residual movement capacity of the joints
- changes in the articulation of a structure.

As a member expands and restrains an adjacent element or structure, the effect will be to reduce the tendency for further expansion until, eventually, a force equilibrium condition is reached. Thus the final movement is less than that which would have occurred in the absence of restraint, and the induced force is less than inferred from considerations of an elastic restraint to the movement.

Particular problems have arisen from distortion, fracture and jamming of steel structures or equipment in reactive concrete structures, especially dams.[5.28] A conservative approach is recommended if there is a doubt that sufficient ductility is available to reach the force equilibrium condition.

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PART II: Assessment, appraisal and monitoring of structures

6 Overall consideration of structures

6.1 General

The overall appraisal procedure is illustrated in Figure 10, which also indicates the appropriate sections of this report to which reference should be made at each stage of the appraisal.

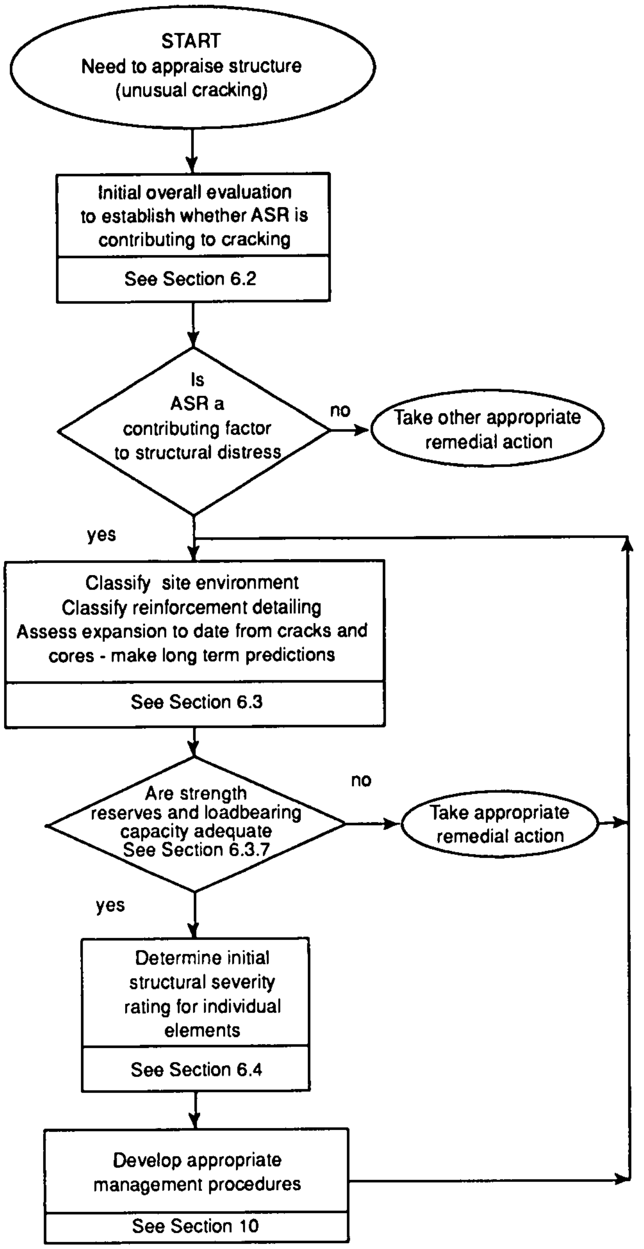

Fig. 10 Appraisal leading to diagnosis and action following ASR

6.2 Initial overall evaluation of a structure

Checks specifically for ASR are normally a small part of the early stages of overall assessment of a structure in which unusual cracking has been reported.[6.1] Only when ASR is diagnosed and found to be significant does it become a major factor.

The following points should be considered when performing an initial desk and site study to establish whether ASR is contributing to cracking:

- actual stress levels in the structure under principal loadings
- reinforcement detailing
- environment, including in particular, moisture availability
- condition surveys of cracks and areas of spalling, corrosion and frost damage
- records and recollections of the construction sequence and mix characteristics
- the nature of the cracking should be related to the points above, and the range of possible structural and non-structural forms of cracking in the concrete
- mix and aggregate identification, which can be based on local surface grinding to expose aggregates and evaluation of construction records
- core samples - initially a minimum of three cores, preferably 350 mm long and 70 mm diameter, should be taken from members with relatively severe cracking potentially attributable to ASR. To help with diagnosis two end sections 50 mm long from each core can be used for initial petrographic examination, and the 250 mm length for expansion testing and water uptake monitoring as follows:
  - 20°C water supply expansion tests should give an indication of whether the material is significantly expansive or reasonably dimensionally stable. A potential future expansion can be estimated initially from trends at 3 months
  - a simple petrographic examination may be sufficient to indicate if the concrete is of a known reactive type or is clearly not reactive. However, more detailed thin section petrography will be required where the cause of any dimensional instability is not clear. This can be carried out on the expansion test samples at a suitable stage. Petrography should also be used to determine overall mix characteristics and the full range of deterioration processes in the concrete to help identify other non-structural cracking effects.

6.3 Initial checks for structural significance of ASR

6.3.1 General

If the initial overall evaluation indicates that ASR might be one of the contributing factors to cracking then the following initial procedure can be adopted to make a judgement on the potential structural significance of ASR with respect to each particular structural element. As further data become available from the

Table 5 Structural element severity rating

Site environment	Reinforcement detailing class (Chapter 8)	Expansion index									
		I		II		III		IV		V	
		Consequence of failure (see subsection 6.3.5)									
		Slight	Significant	Slight	Significant	Slight	Significant	Slight	Significant	Slight	Significant
dry	1	n	n	n	n	n	n	n	n	n	n
	2	n	n	n	n	n	n	n	n	n	D
	3	n	n	n	n	n	n	n	D	D	C
intermediate	1	n	n	n	D	D	C	D	C	D	C
	2	n	n	D	C	D	C	C	C	C	B
	3	n	D	D	B	C	B	B	A	B	A
wet	1	D	D	D	C	D	C	C	B	C	B
	2	D	D	C	B	C	B	B	B	B	A
	3	D	C	C	A	B	A	A	A	A	A

Structural severity ratings: | n = negligible | D = mild | C = moderate | B = severe | A = very severe

structure, the judgement can be refined. The following factors are considered to be those which affect the structural significance of ASR in a particular element:

- expansion characteristics of the concrete.
- site environment.
- reinforcement detailing
- consequences of failure
- stress levels
- residual strength of the concrete (see Chapter 4).

It is emphasised that the above factors are not listed in order of importance. Each factor is assigned a classification as indicated in the following sections.

6.3.2 Expansion index

A concrete with the potential to exhibit a large expansion in the absence of any restraint, i.e. its free expansion, is potentially more damaging than a concrete with a low potential for free expansion.

The current free expansion can be estimated from the structure as explained in subsection 7.3.2. The total future free expansion can be determined by combining the estimated current free expansion with an estimate of the future expansion obtained from cores as explained in subsection 7.3.3. The expansion characteristics of the concrete are then defined in terms of an expansion index in the range I toV which is related to either the estimated current free expansion, for an assessment of the current state of the structure, or to the estimated future total expansion, for an assessment of the future state of the structure.

I expansion less than 0.6 mm/m

II expansion of at least 0.6 mm/m but less than 1.0 mm/m

III expansion of at least 1.0 mm/m but less than 1.5 mm/m

IV expansion of at least 1.5 mm/m but less than 2.5 mm/m

V expansion equal to, or greater than 2.5mm/m

6.3.3 Site environment

The potential expansion is achieved only if there is a sufficient supply of water. This effect is taken into account by assessing the environment local to the element (the site environment) to be dry, intermediate or wet. These are broadly equivalent to the BCA report categories of low, medium and high environmental risk. The definitions of these environments are given below:

Dry

- reliably dry, always < 75% r.h.
- currently dry, < 75% r.h. but at risk from change of use or leakage.

Intermediate

- intermediate, always < 85% r.h.
- currently intermediate but at risk from change of use or leakage.

Wet

- all buried parts or those resting against soil
- all external elements in UK climates whether subject to rain or condensation
- internal elements where frequent wetting can occur, e.g. washing, frequent showering, swimming pools, cooking, etc.
- where condensation can occur due to thermal gradients.

The second category listed under the dry and intermediate classifications should be subject to rigorous inspection and maintenance to control their moisture condition.

6.3.4 Reinforcement detailing

The ability of reinforcement to provide restraint to the free expansion is dependent on its detailing. Three classes of detailing are illustrated in Fig. 14 and discussed in Chapter 8.

6.3.5 Consequences of failure

The structural significance of an element is a function of the consequences of its failure. These are judged to be slight or significant as defined below:

- slight - the consequences of structural failure are either not serious or are localised to the extent that a serious situation is not anticipated
- significant - if there is risk to life and limb or a considerable risk of serious damage to property.

6.3.6 Stress levels

If the stress levels for current non-ASR loadings are low when determined using the Institution of Structural Engineers procedures for assessing structures[6.2] or the Department of Transport's procedures for assessing highway structures,[6.3,6.4] then the structural significance of ASR is less than if the stress levels are high. The way in which the non-ASR stress level affects the structural severity is discussed in section 6.4.

6.3.7 Residual strength of concrete

For initial appraisal the reduction factors in Table 4 should be used with the specified 28 day strength or the characteristic 28 day value from the original cube data if available. In cases of severe damage or high stress, actual strength should be obtained from cores. Where possible at least twelve cores should be taken; four from badly damaged regions; four from moderately damaged regions; and four from undamaged regions for tensile or unaxial compressive strength tests as appropriate to the potential mode of failure.

6.4 Initial structural severity rating

The classifications of expansion index, site environment, reinforcement detail and consequences of failure are combined by means of Table 5 to give an overall Initial Structural Severity Rating for each element as one of Negligible (n), Mild (D), Moderate (C), Severe (B) and Very Severe (A).

Stress level is taken into account by decreasing the rating by one, e.g. from A to B if the non-ASR stress level is less than 60% of the allowable, and by increasing the rating by one, e.g. C to B if the non-ASR stress level exceeds the allowable.

It is emphasised that engineering judgement must be used to interpolate between classifications when using Table 5. The rating obtained from Table 5 should be reviewed as more data become available.

6.5 Development of detailed evaluation of structural elements and overall behaviour

The wide variety of structural forms and severity of effects of the reaction make generalised recommendations for detailed evaluation inappropriate. The initial evaluation will normally classify 90% of the structures in the Negligible (n) or Mild (D) categories and only a few critical details will be in the Severe (B) or Very Severe (A) categories. Sometimes it will be quicker and cheaper to strengthen to improve a poor detail than to fully evaluate the current and future risks from ASR.

The detailed evaluation is concentrated on the critical details, but representative sampling for testing will often be based on lower stressed non-critical areas with similar deterioration.

The initial evaluation as in 6.2 can be progressively upgraded by:

- quantitative evaluation of structural behaviour using the approach summarised in Chapters 4 and 5, initially with the Table 4 lower bound physical properties and then data from tests on the structure.
- a more detailed evaluation of the risks and consequences of potential failure modes including spalling.
- insitu r.h. measurements and evaluation of methods of reducing the amount and/or rate of water ingress.
- refining the expansion to date estimates from
  - crack evaluation
  - consideration of stresses
  - restraint effects

  related to stiffness characteristics from cores and/or UPV measurements.
- testing a more comprehensive set of cores which would be taken from areas with a range of crack severities for stiffness, UPV, expansion tests, petrographic and analytical tests and strength tests related to the specific failure modes to be evaluated. This would provide data on current and future physical characteristics for assessment of current and potential future margins of safety and the rate at which further damage may develop.
- monitoring the trends of crack movement and overall movement of the structure. Once data are available for a two year period, the current rate of deterioration can be estimated and can be progressively reviewed thereafter.
- tests of restrained expansion of samples removed from structures or measurements of the locked in stresses in the concrete or reinforcement can provide data for the appraisal of specific structures where the available budget and the timescale for testing make this possible.

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7 Diagnosis and assessment of expansion and cracking

7.1 Diagnosis

The BCA Working Party Report[7.1] recommends available techniques for diagnosis. Unless ASR induces free expansions of 0.6 mm/m it cannot be regarded as damaging, except in large mass concrete structures such as dams. Thus the criteria for differentiating between damaging ASR which might influence structural performance from innocuous ASR which will show up in petrography are:

- cracking attributable to ASR indicating at least 0.6 mm/m expansion
- a reduction in core UPV (see Table 3)
- core stiffness reduction indicative of 0.5 mm/m expansion (see Table 4)
- expansion of cores from wet or intermediate environments exceeding 0.6 mm/m in long term expansion tests
- test results indicating potential future expansion which will in total exceed 0.6 mm/m.

7.2 Testing

7.2.1 General

Table 6 attempts to bring together tests outlined in the BCA report and this report. It also gives guidance on the relative usefulness of the tests. The type and amount of testing will depend on the severity of the ASR damage, as summarised in Table 7. The tests shown in Table 6 may also help to evaluate the contribution of other non-structural cracking that may co-exist with ASR.

The methods of sampling, testing procedures and interpretation are still under active development and further research is required. The Engineer should review new developments when selecting a testing programme for a specific structure.

7.2.2 Sampling

The diversity of mix characteristics as well as the range of damage and environment must be reflected in the sampling rate. At least three cores from the worst cracked areas of a structure are needed for preliminary tests. For more extensive appraisal, three to five cores should be obtained from each pour to be tested so that the variability of characteristics is established. Individual cores cannot give representative results. Further guidance is given in appendix A.

7.2.3 Expansion tests

These tests measure the average expansion and variability of expansion of concrete cores over a period of months or years. Normal concretes show a little expansion, typically 0.2 mm/m, from recovery of drying shrinkage. ASR-affected concretes expand more, and the potential severity of future damage to the concrete can be assessed from the long term trends of expansion. Because of the high variability of expansions in cores, even from one pour of concrete, the expansion for estimation of structural effects should be based on the greatest average value for the most expansive of up to four cores from different pours of the same aggregate and cement combination. Where a fuller set of cores is taken, the upper quartile value of expansion should be used for assessment. The core locations should be selected to cover the range of cracking from the most severely cracked to uncracked for that mix, in broadly similar conditions of exposure to moisture.

There is no definitive relationship as yet between the unrestrained expansion of a core and the restrained expansion within a structural member.

7.2.4 Strength tests

Measurement of loss of concrete strength with ASR is complicated by the high variability of the effects of ASR, which are superimposed on the normal variability of concrete. For further details, see Table 6 and appendix A.

7.2.5 Chemical analyses

Chemical analysis of samples of the concrete may be useful where structures are in the very severe (A) or severe (B) structural severity ratings to determine the range of alkali and cement contents in the concrete at the time of testing.[7.1] Sampling and analysis is, however, imprecise. One must also consider the potential non-cementitious sources of alkali, both within the original mix and from ingress. For example, checks can be made for chlorides associated with the use of de-icing salts. Sampling and analysis also needs to determine migration of alkalis by measuring profiles where appropriate.

7.3 Assessment of expansion

7.3.1 General

It is necessary to estimate the two components of expansion separately to asssess the total expansion : first, the expansion that has occurred up to the time of the investigation (current expansion); and second, the potential for future additional expansion that will have occurred when ASR is exhausted (potential additional expansion). The relationship between current and potential additional expansion is illustrated on Fig.11. It should be noted that, in Chapter 8, structural appraisal is considered in terms of levels of severity of estimated free expansion of cores, not of the restrained expansion of the structure.

7.3.2 Estimated current expansion

Determination of the current expansion is difficult, and normally can be made omly indirectly because of the absence of datum measurements. The Task Group found some difficulty in proposing a suitable method. However, test data[7.2,7.3,7.4] indicate that crack summation is the most useful method for initial appraisal and laboratory comparisons have enabled some correlation to be established.

The proposed method of obtaining a rough indication of the expansion in the structure is by measuring the widths of all the cracks crossed by a straight line drawn on the concrete. It is recommended that at least five lines be drawn, not less than 1 m long, with at least 250 mm between them, in a direction perpendicular to the principal crack orientation on the most severely affected face of each pour of concrete being assessed. Where the dimensions of the concrete dictate line lengths of less than 1 m, more lines should be used. The preferred method of crack width measurement is by means of a crack microscope. A transparent crack comparator is difficult to use accurately for crack widths less than about 0.2 mm.

The expansion strain, together with the strains from other causes, can be assumed to equal the sum of the widths of the cracks divided by the length of the line drawn on the concrete,

Table 6 Laboratory tests on cores for diagnostic and structural investigation of ASR

Test	*Purpose*	*Use and importance of test results*		
		Diagnosis of ASR	Prognosis	Structural assessment
Preliminary laboratory examination	Record of samples taken on site. Look for signs of possible ASR, and highlight areas for more detailed laboratory investigations.	***	***	⊕⊕
Examination of sawn, finely ground and impregnated sections	Look for evidence of ASR and assess internal crack patterns. Areas for more detailed examination, e.g. by use of thin sections can be highlighted.	***	*	-
Measurements such as length, widths and frequency of cracks	Assists in quantification of the damage and its severity.	*	*	⊕
Volumetric proportions	Quantify selected constituents of the concrete. Useful for quantifying amounts of reacted and/or potentially reactive aggregates.	*	**	-
Examinations of thin sections	Identify reacted and potentially reactive aggregate. Confirm the presence of any alkali-silica gel and reaction sites, to observe the extent and configuration of related cracking.	***	*	⊕
Determination of alkali content in concrete	Assess alkali levels within the concrete in order to consider the sources of alkalis and the possibility of local concentration.	**	*	-
Determination of cement content	Assist in estimating the original alkali content of the concrete.	*	*	-
Determination of chloride content	Used in conjunction with the determination of alkali content to infer the possible contribution of sodium chloride to the alkali content of the concrete.	-	*	⊕
Examination of excudations and crystalline material	Identify exudations and crystalline material e.g. to identify the presence of alkali-silica gel.	***	*	-
Diagnostic expansion testing (38°C)	Assist in diagnosis and in assessing the potential for further expansion.	**	***	⊕
Alkali-immersion test	Assists in the identification of reactive aggregates.	*	-	-
Expansion testing	Determine potential for expansion in site conditions of temperature range, moisture and restraint.	-	-	⊕⊕
UPV	A simple measure of micro-cracking damage.	-	-	⊕⊕
Uniaxial compressive	To quantify compressive strength.	-	-	⊕
Tensile strength	To quantify tensile strength.	-	-	⊕
Stiffness	A sensitive measure of physical effect of micro-cracking, which can be related to expansion to date.	-	-	⊕

Tests discussed in *The diagnosis of ASR*, [7.1] BCA, 2nd ed.
*** Important test. ** Do when possible. * Results could be useful if tests can be done.

Tests discussed in this report
⊕⊕ This test is useful for initial assessment, and can be used more extensively for more detailed appraisal. ⊕ Tests useful for more extensive appraisal.

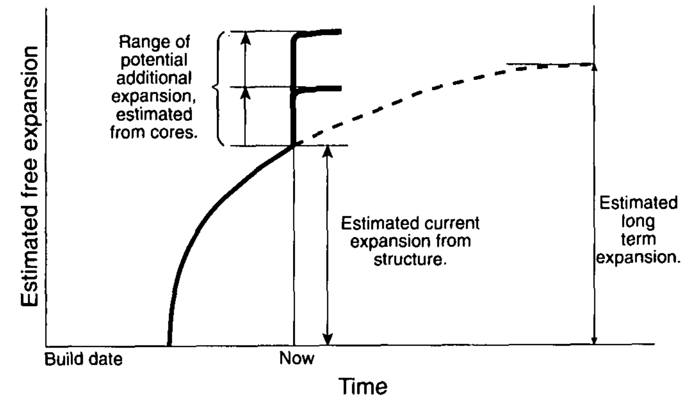

Fig. 11 Definitions of various free expansions

hence the current total strain equals the sum of widths divided by length, expressed in mm/m.

It is important to appreciate that the strain, thus calculated, is the algebraic sum of the strains arising from structural and non-structural effects in addition to the ASR strain. These other strains should be subtracted from the total strain to give the ASR strain. Furthermore the ASR component is not usually the free expansion that would have occurred in cylinders or cores, since it is generally restrained by the presence of reinforcement and/or applied stress, as discussed in Chapter 5. Fig. 7(a) can be used as a guide to the relationship between restrained and free expansion.

If possible lines of crack width measurement should be drawn in regions where the concrete is not strained by effects other than ASR so as to minimise the required adjustments to the calculated strain.

The proposed expression for total strain gives an average fit to scattered test data. The actual strain may vary by ± 50% from the calculated value. This compares with the ± 30% accuracy of the formula for flexural crack widths given in Codes of Practice.[7.5]

7.3.3 Potential additional expansion

The potential maximum additional free expansion that may occur in a structure once the reaction is exhausted may be estimated from free expansion tests on cores taken from the structure. The 38°C conditions for the expansion tests recommended by the BCA[7.1] may not be the most representative environment for predicting long term trends or the maximum potential expansion in the structure.

It is important to relate all expansion test results to weight uptake from water drawn into the sample during the test. Where a core is relatively dry before the test, the recovery of reversible drying shrinkage will increase the core expansion. Where the core absorbs more than 15g/kg of water this should be checked and corrected for as described in appendix A.

For structural assessment expansion tests, it is necessary to model the conditions in the structure as closely as possible and relate this to the monitoring of the structure (see Chapter 10). For example, typical UK mean seasonal temperatures are 5°C (Winter), 20°C (Summer) and 13°C (Annual). Alternatively, laboratory temperature (generally 20°C ± 5°C) can be accepted. There are indications that, with some aggregates, the expansions over periods exceeding one year at 13°C to 20°C are substantially higher than in the 38°C rapid diagnostic test in which expansions start rapidly and then decrease after a month or so. This is shown in Fig. 12 from reference 7.6 for aggregates from southwest England, e.g. coarse limestone combined with sea dredged fines containing chert. It is not known if this occurs with other aggregates.

With many tests, long term expansions continue at a slow rate so it is not possible to define a limit of expansion. However, in the structure the expansion will develop at a rate controlled by the reactivity of the material, the moisture supply and the restraint.

The expansion of a core is also dependent on the restraint that was experienced by the core in the structure and which is relieved on extraction of the core. The greater its previous restraint, the greater is its subsequent expansion. Hence, the additional free expansion estimated from cores can be an over-estimate of the total free expansion of the concrete.

7.3.4 Estimated long term expansion

While the short term laboratory expansion tests are in progress, cores should be placed in their site environment adjacent to the structure and the results compared. The site tests are more likely to be reliable in the long term. By plotting expansion against log time for both tests the slowing down of the reaction due to its partial exhaustion may become apparent. The use of steel control plates with the cores to monitor Demec gauge wear is desirable during monitoring over a period of years. Long term field measurements of structural deformation will also provide information for assessment of long term expansion.

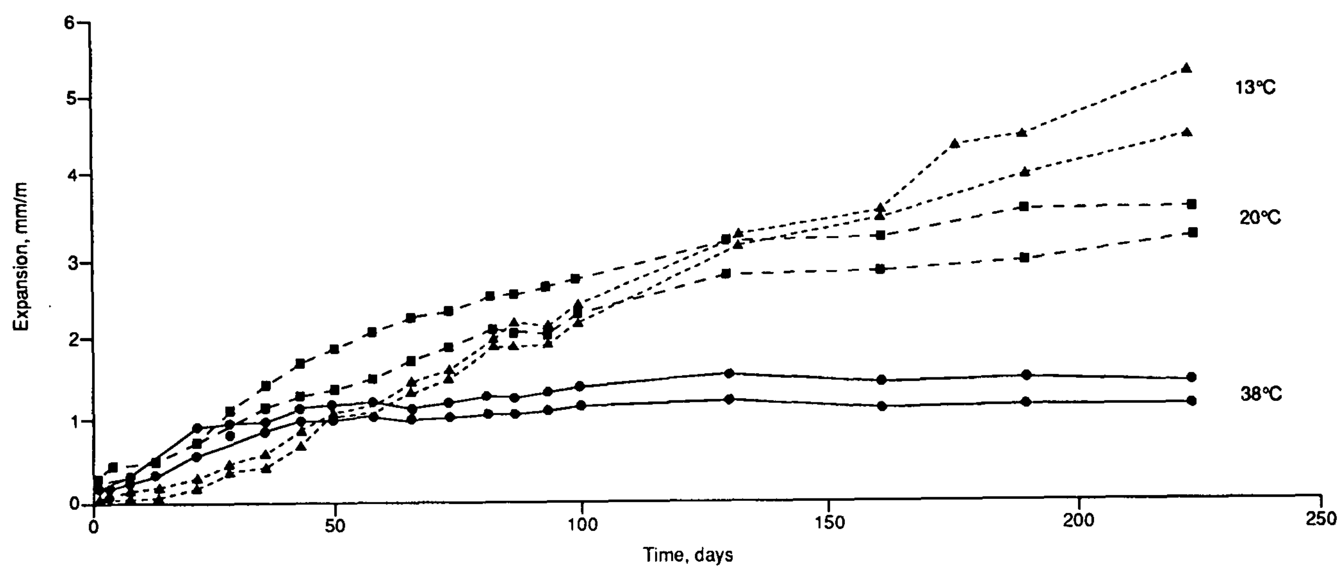

Fig. 12 Core expansion at 100% r.h. and different temperatures, aggregate from southwest England

References

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7.2 Ng, K.E.: 'Effect of alkali-silica reaction on the punching shear capacity of reinforced concrete slabs,' PhD Thesis, The University of Birmingham, 1991

7.3 Chana, P.S. & Korobokis, G.A.: 'Structural performance of reinforced concrete affected by alkali silica reaction: Phase II,' Transport and Road Research Laboratory, Contractor Report 311, 1992

7.4 Nishibayashi, S., Yamura, K. & Sakata, K.: 'Evaluation of cracking of concrete due to alkali-aggregate reaction,' *8th Int. Conf. on AAR,* Kyoto, Japan, July 1989, pp 759-764

7.5 Beeby, A.W.: 'An investigation of cracking in slabs spanning one way,' Cement & Concrete Association, April 1970

7.6 *Report on full appraisal and recommendations for future management of Marsh Mills viaduct,* Vol. 2, Appendices, Mott, Hay & Anderson, Department of Transport, SWRO, March 1986

8 Appraisal of structural strength

8.1 General

Since current information is incomplete in a number of areas, some of the recommendations that are given below may later need revision. At this stage, the general advice given is, of necessity, cautious, and it should be emphasised that many structures with expansions in excess of 1.0 mm/m have been satisfactorily retained in service throughout the world with, to date, only a few cases of additional strengthening being deemed necessary. Fig. 13 provides a scale for comparison of structural strains and two examples of estimated free expansion strains from ASR.

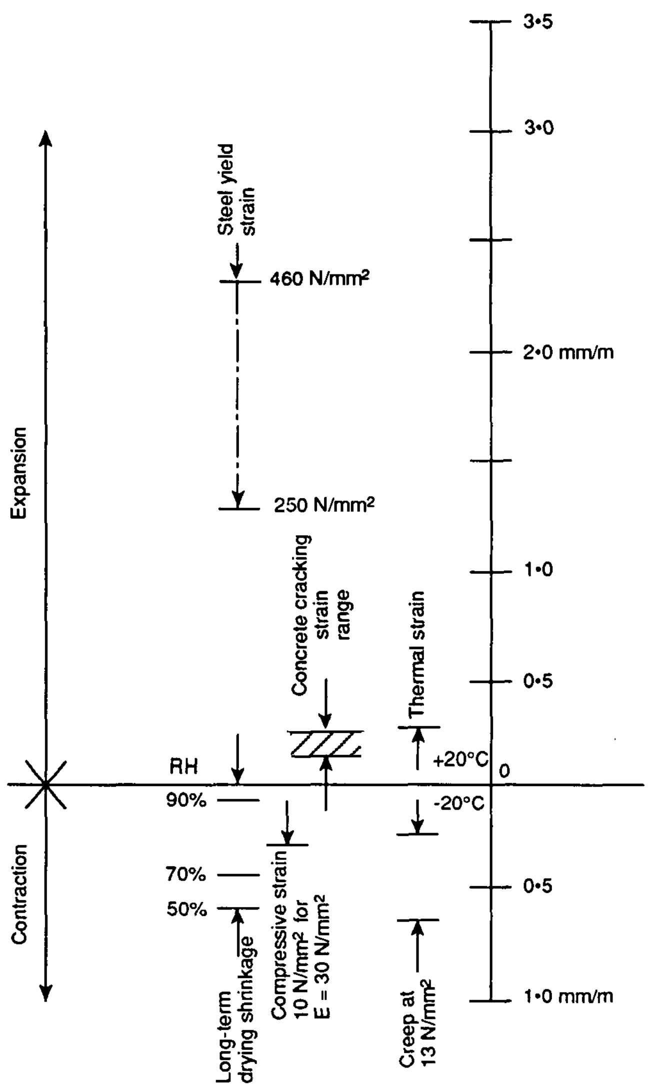


Fig. 13 Comparison of structural and ASR strains

Two structural appraisals are usually required: one based on the current condition of the structure; and one based on the estimated condition of the structure at some specified time in the future. In the following subsections, guidance is given on structural appraisal in terms of five levels of severity of the total ASR free expansion that is estimated to have occurred (or is predicted to occur) at the time of interest (i.e. either now or at some specified time in the future), and which have been described in section 7.3.

The five levels of expansion are related to three classes of reinforcement detailing:

- *Class 1:* a 3-dimensional cage of very well anchored reinforcement
- *Class 2:* a 3-dimensional cage of conventionally anchored reinforcement
- *Class 3:* a 2-dimensional cage of reinforcement in one or two faces; no through ties, no links or low cover.

Examples of these Classes for a wall are shown in Fig.14a. If turned through 90°, these three diagrams also illustrate the Classes for the reinforcement at the edge of a slab, or end of a beam. Fig.14b shows comparable classes for columns.

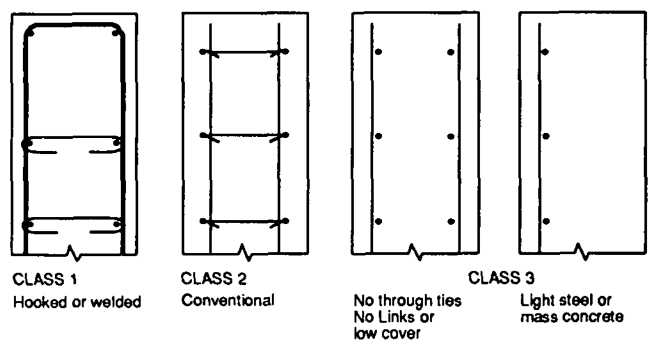


Fig. 14a Detailing classes for walls and slabs

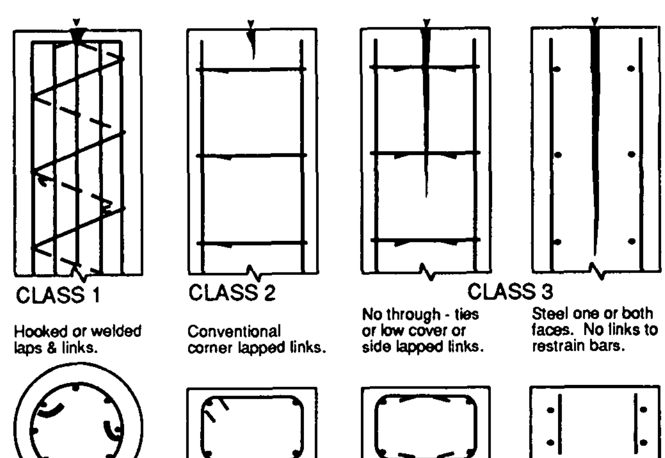
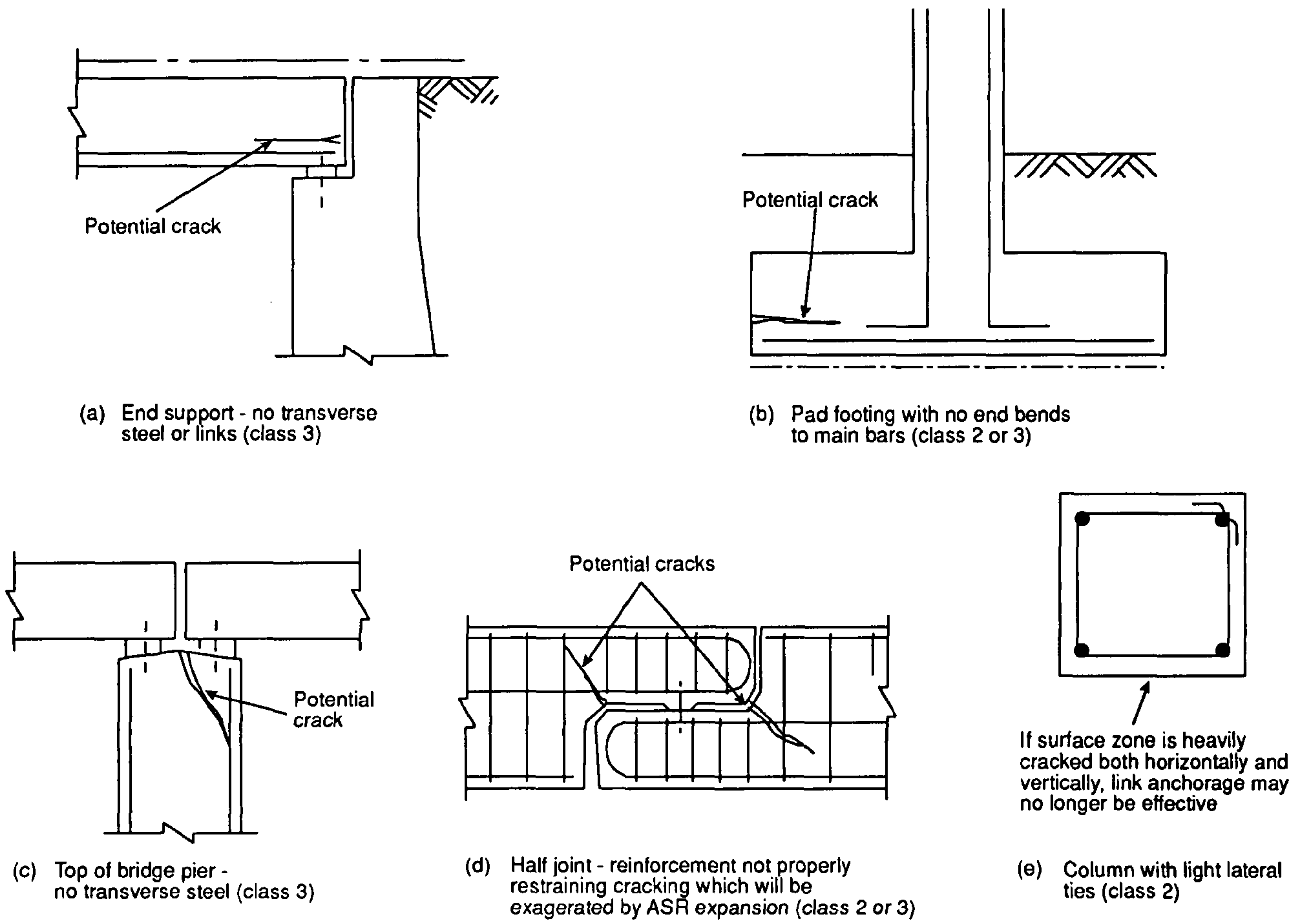


Fig. 14b Detailing classes for columns

Generally, the efficiency of reinforcement in maintaining the integrity of a structure depends on the degree of containment it provides to the differential expansion. Parts of the structure with reinforcement of Class 1 or 2 that are moderately stressed by dead and imposed loading will be slightly affected only even by fairly severe ASR expansions, whereas a similar degree of expansion in parts of a structure with Class 3 reinforcement may produce severe structural effects, regardless of the loading.



(a) End support - no transverse steel or links (class 3)

(b) Pad footing with no end bends to main bars (class 2 or 3)

(c) Top of bridge pier - no transverse steel (class 3)

(d) Half joint - reinforcement not properly restraining cracking which will be exagerated by ASR expansion (class 2 or 3)

(e) Column with light lateral ties (class 2)

Fig. 15 Sensitive details showing potential crack positions

8.2 Expansion indices

8.2.1 Expansion index I

Expansions of the order of 0.4 mm/m occur in normal concrete, and are of no concern even if ASR has been identified petrographically.

Provided that the structure has been properly designed and has reinforcement corresponding with Class 1 or Class 2, ASR expansions up to 0.6 mm/m will affect the strength only marginally, and any shortfall in structural strength may be assumed to be covered adequately by the normal safety factors. Details that are outside accepted good practice should be assessed for adequacy in accordance with normal assessment procedures,[8.1, 8.2] and no specific action need be taken in respect of ASR. If surface cracks exceed 0.3 mm, local problems associated with detailing may arise and the risk of corrosion increases. However, such crack widths are unlikely to be caused by ASR expansions within the above limit. Large lightly reinforced mass concrete structures such as dams may need more detailed consideration.

8.2.2 Expansion index II

Expansion in the range 0.6 mm/m to 1.0 mm/m does not affect the compressive strength of the concrete significantly. The behaviour of members in bending and compression may, therefore, be taken as adequate. The reductions in concrete tensile strength (see Chapter 4) may adversely affect behaviour in shear and bond, although, provided the structure is detailed as Class 1, any shortfall may, again, be assumed to be covered by the normal safety factors. However, Class 2 or Class 3 detailing must be assessed with care. Examples of details that are particularly sensitive to the effects of ASR are shown in Fig. 15.

8.2.3 Expansion index III

For structures with expansion in this class, a detailed appraisal will be necessary, with consideration of potential reductions in bending and compression capacities as well as of shear and bond following procedures described in Chapter 5. Although Danish[8.3] and Japanese tests[8.4] have indicated increases in shear strength of ASR-affected beams, a prudent approach to shear is suggested. This is because the majority of the Japanese beams that failed in shear had good end anchorage to the main reinforcement and the overall percentage of steel was often greater than in the UK to deal with possible seismic loading. However, it should be noted that excellent behaviour in shear has been observed with end anchorages as small as 3.4 bar diameters.

A *pro rata* reduction in strength should be taken in proportion to the reduction in bond tensile strength (see section 5.3). This reduction is conservative where links pass outside or around the main bars, but specific guidance on the contribution from links still has to be derived.

Class 2 or 3 details of the type indicated in Fig.15 will be at even greater risk in this range of expansion. Also, there are likely to be general surface cracks in excess of the conventional 0.3 mm limit. In view of this, consideration should be given to discounting the contribution to strength of the cover concrete, which may be affected by such cracks. Particularly severe local problems associated with corrosion and frost action may arise if individual cracks exceed 1 mm in width, unless the affected element can be protected from water ingress, driving rain and condensation. Thus the supply of water through failed joints in cases *(a)*, *(c)* and *(d)* of Fig. 15, a high water table in case *(b)* or driving rain in case *(e)* will exacerbate the problems.

8.2.4 Expansion index IV

For expansion of this order, a detailed insitu appraisal is required. The comments of subsection 8.2.3 are applicable. For well detailed structures, the main reinforcement will reduce free expansions in this range. However, secondary reinforcement is not likely to be of a sufficient area to reduce the free expansion below its yield strain.

8.2.5 Expansion index V

Each structure will have to be the subject of special study, testing, appraisal and monitoring, which might include load testing. In the absence of transverse reinforcement, large continuous cracks parallel to the main reinforcement are often noted. These cracks can form along the reinforcement resulting in subdivision of the concrete and corrosion of the reinforcement. Internal delamination arising from differential restraint has also been noted in slabs and walls without reinforcement normal to their major surfaces. Such cracking could reduce shear and bond strength, but test data reviewed in Chapter 5 can be used to estimate the extent of the reduction. Where the calculated shear and bond stresses are close to or above code permissible values, as outlined in BS 8110 : 1985 and BS 5400 : Part 4 : 1990, specialist advice should be sought.

References

8.1 *Appraisal of existing structures*, Institution of Structural Engineers, July 1980, London (currently under revision)

8.2 *The assessment of highway bridges and structures*, Department of Transport standard BD 44/90, October 1990, and advice note BA 44/90, October 1990

8.3 Load carrying capacity of bridges subjected to alkali-silica reactions, Interim report no. 1: *The shear strength of concrete beams subject to alkali-silica reactions*, Danish Ministry of Transport, July 1986

8.4 *ASR in Japan: report of a mission team*, Cement & Concrete Association, Slough, June 1987

9 Appraisal of structural serviceability

9.1 Cracking and corrosion in reinforced concrete

The relationship between cracking and corrosion in ASR-affected concrete is not clear; reliance cannot therefore be placed on existing practice. ASR cracking can be unsightly and the fact that cracking continues to develop over years means that concern may be repeatedly raised by users. In some structures severe ASR cracking has led to secondary damage from corrosion.

Many structures affected by ASR may be subject to severe or very severe exposure conditions. Current codes are more stringent than previous practice in these exposure circumstances. Where the concrete quality and the cover comply with current codes of practice and surface crack widths do not exceed 0.3 mm, it is reasonable to assume that the effect of ASR on corrosion rates should not be significant. When cracks wider than 0.3 mm develop and there is a plentiful supply of water, calcium hydroxide can leach out, giving unsightly staining at later stages. Such staining also occurs with cracking due to reasons other than ASR.

If any one of these conditions is not satisfied, an investigation into the actual and potential corrosion should be made.[9.1] Specialist petrographic techniques enable the ingress of carbonation and chlorides down cracks to be measured. This ingress is significant only where corrosion is initiated by cracks running generally along the line of the reinforcement and these are therefore particularly likely to corrode the reinforcement. The largest cracks are likely to occur in damp conditions.

Special note should be taken where the atmosphere is both damp and aggressive, as in marine or industrial environments or those exposed to road salt. Cracks unlikely to exceed 0.3 mm wide may not be serious but should be kept under observation. Where a structure is particularly sensitive, typical cracks should be sampled and petrographically examined. An approximate correlation between crack width and depth is shown in Fig. 5.

With wide cracks the possibility of pieces of surface concrete falling away can arise due to the combination of ASR, corrosion in the underlying reinforcement and frost action.

9.2 Cracking and corrosion in prestressed concrete

Because of the greater vulnerability of prestressing steel to corrosion, current design practice is either to design for no surface cracks in tendon regions, or to allow them to open only briefly under transient load. If cracks penetrate to tendons, there will be a risk of corrosion. Even slight corrosion of tendons is a matter for concern. ASR cracks in prestressed members will generally be longitudinal.

It follows that an investigation for corrosion will generally be required on all prestressed structures affected by ASR. A detailed investigation should however not be necessary where surface crack widths are less than 0.15 mm since such cracks are unlikely to penetrate to the tendons, and where in addition the concrete cover and quality comply with the requirements in current codes of practice.

9.3 Deformations in structural members

Flexural deformations in beams and slabs will tend to be in the direction opposite to that caused by load and should not be a cause for concern with expansions up to 1.0 mm/m.

In respect of overall expansion of members, the normal provisions for movement and expansion should cater for expansions up to 0.5 mm/m. Beyond this point, the consequences of expansion should be assessed, but it should be noted that estimated expansions from cores do not allow for the restraining effects of reinforcement.

9.4 Deformations in mass concrete

Even modest expansions will, in large structures such as dams and bridge piers, lead to substantial overall movements which can seriously affect adjoining parts of the structure or machinery incorporated within or bolted to the mass. Such situations therefore require special investigation.

9.5 Frost resistance

The formation of micro- and macro-cracks has raised doubts about the resistance to freezing and thawing of concrete affected by ASR. Unpublished laboratory studies on concrete specimens kept moist throughout their life have shown that freeze-thaw resistance is reduced by ASR cracking. There are differences of opinion regarding the effect of ASR expansion upon freeze-thaw resistance of field concretes. Where serviceability of the structure is in doubt, the interaction should be evaluated both petrographically and by freeze-thaw tests.

Horizontal exposed surfaces may be at risk, particularly if water can pond on the surface. Vertical surfaces can also be at risk if water can flow over the surface for long enough periods for the concrete to become saturated so that water freezes in ASR cracks. Where ASR has caused cracking, precautions against the effects of frost may be required.

9.6 Effect on fire resistance

While resistance to fire will rarely be a design requirement for most structures likely to be affected by ASR, it may be assumed that no reduction in performance will occur for expansions up to 1.0 mm/m. It may be helpful to note in this connection that the unrestrained overall expansions in structural members in actual fire conditions are often in excess of 1.0 mm/m.

Reference

9.1 Pullar-Strecker, P: *Corrosion damaged concrete - assessment and repair*, CIRIA, 1987

10 Management of ASR affected structures

10.1 Management procedures

10.1.1 General

In addition to other requirements, it is recommended that structural analyses be carried out to determine whether the effect of the ASR has been or will be to weaken the structure significantly and if so in what circumstances and over what time-scale. Examples exist of structures with a reserve of strength that is so large that this factor alone dominates the management procedure adopted for the structure. The approach to structural management should be guided by the factors outlined in the following paragraphs, summarised in Table 7.

Where it is necessary to reduce the access of water to the structure, this may be effected by improved drainage, cladding or coatings. Although coatings such as silanes, and epoxies are highly regarded by the Japanese, their reception in the UK has been less enthusiastic. Particular care in the use of coatings should be taken to ensure that sufficient areas of the structure are treated to avoid a tanking effect.

The effect of ASR on a large structure such as a dam or a power station is often likely to cause difficulties with the operation of electrical or mechanical plant as a result of the concrete expansion. Among the measures that have been suggested[10.1,10.2] in an attempt to control these effects are:

- large struts or buttresses to resist compressive loads
- large prestressed anchors
- cutting of slots to relieve compressive load
- injections of carbon dioxide into the concrete mass to neutralise the pore fluid
- waterproofing of the surface of the concrete

Experience suggests that where it can be accommodated within the structure, the solution of cutting slots in the concrete has been effective in isolating the swelling concrete from the vulnerable installed equipment.

10.1.2 Management of structures with a severity rating of 'mild' (D)

At this severity rating it is appropriate to install simple instruments, probably Demec gauge points across selected cracks at a few typical locations.

It may be informative to install Demec points in a triangular formation where shear movement along cracks needs to be checked. Alternatively, where appropriate, it may also be useful to consider overall movement monitoring. Readings should be taken in accordance with the frequencies in Table 7.

Cores taken from the structure can, if appropriate, be placed under test in the laboratory, and such testing should continue until a long-term trend is established, probably after a minimum of two years of testing. The data from laboratory and field measurements should be compared annually.

Consideration should be given to reducing the access of water to the structure where this is practicable. Inspections as stated in Table 7 may be varied depending on the observations made during the reading of the instruments.

10.1.3 Management of structures with a severity rating of 'moderate' (C)

Inspections every four months should be appropriate. Monitoring of cracks is likely to be necessary together with more detailed instrumentation and the laboratory testing of cores taken from the structure. Comparison of field and laboratory data should be made to assess the long term trend. The access of water to the structure should be reduced where possible.

10.1.4 Management of structures with a severity rating of 'severe' (B)

At this severity rating, inspections at the intervals outlined in Table 7 are likely to be appropriate. Full investigation of the structural detailing should be carried out, and remedial or strengthening work may be needed. Load restriction may be necessary. Extensive and carefully planned instrumentation is appropriate. Comparison of field and laboratory data should be made every two months. Load testing may be useful.

10.1.5 Management of structures with a severity rating of 'very severe' (A)

A structure in a condition that merits a 'very severe' rating must be the subject of detailed, specialist study, which may include load testing. Measures may be required to reduce the rate of deterioration from the reaction. (See also section 6.3.)

10.2 Inspections

Regular inspections must be carried out by experienced personnel, who should record salient details. Methods can range from writing or recording notes, making sketches or taking photographs through to carrying out representative crack surveys on proformas using a portable micro-computer for processing. Good advice on inspections is given in references 10.3 to 10.7.

Inspection procedures for structures with ASR need to be drawn up considering the Severity Rating of each part of the structure.

The key questions to be answered by inspection are:

- Are there any indications of overall movement which may have adversely affected the performance of the structure, e.g. expansion joints, distortion of adjacent structures or equipment?
- Are there areas where protection of the structure from water can be improved by preventing ponding or water running across the surface, e.g. below leaking expansion joints on bridges, poor condition flat roofs, blocked drains etc.? Monitoring should also review the effectiveness of measures to exclude water from the structure
- Are there new areas of cracking or a significant increase in severity?
- What changes are there in the trends in the monitored movements for cracks and the structure?
- Are there signs of developing secondary deterioration from frost or corrosion which need attention, particularly where spalling would create a hazard?
- What movements have occured in cores from the structure exposed adjacent to, or in similar conditions to the structure?

Based on this data, together with an update of laboratory test data, the current conditions and reserves of strength should be assessed along with a revised estimate of the potential for further expansion and its structural implications with a revised severity rating.

10.3 Instrumentation

Instruments are applied to a structure suffering from ASR to record the movement of the structure as a result of the reaction. Instruments that have proved useful for this purpose include the following:

Table 7 Management actions related to structural element severity rating

Summarised management procedure	*Structural severity rating (see Table 5)*			
	A	B	C	D
Improve drainage and protect surfaces from water run off and ponding.	✓ ✓	✓ ✓	✓ ✓	✓ ✓
Overall crack surveys including estimate of expansion to date.	✓ ✓	✓ ✓	✓ ✓	✓ ✓
Frequency, Years	1	1	3	6
Coring for stiffness and expansion tests for current and future expansion estimates.	✓ ✓	✓ ✓	✓ ✓	✓
Coring for stiffness and strength tests to evaluate specific failure mode.	✓ ✓	✓ ✓	✓	X
Evaluate benefits of load reduction, strengthening to improve detail class or replacement of critical elements.	✓ ✓	✓ ✓	✓	X
Detailed inspections and monitoring of cracks and, where important, overall movements.	✓ ✓	✓ ✓	✓ ✓	✓ ✓*
long term frequency, months Note: inspect twice as often for first 6 readings. * one set at a sample location on a structure only.	1	2	4	12
Inspection for spalling risk from secondary corrosion and frost damage.	✓ ✓	✓ ✓	✓ ✓	✓ ✓
Frequency, months	3	6	12	12

Key: X Seldom required ✓ May be required ✓✓ Desirable

- Demec gauges or other devices to measure crack movement
- tapes and tape extensometers, monitoring pins and micrometers or chains of Demec gauge points to measure overall length changes over several metres
- tapes or survey instruments to measure the differential movements at joints
- borehole extensometers to determine the location, within massive concrete, of movement
- inclinometers
- a remote monitoring system may be useful as may data-logging equipment, particularly where access is difficult.

10.4 Monitoring of ASR deterioration in structures

The development of alkali-silica reaction in structures and the expansion which is sometimes induced is generally substantially slower than in the laboratory. Laboratory monitoring of expansion rates on cores, as discussed in appendix A, at lower temperatures or with more limited moisture availability, or site exposure and monitoring on site, can provide an intermediate link between accelerated laboratory test methods and site performance. The rate of expansion depends on both the temperature and the availability of moisture and the rate at which moisture migrates into large structural elements, all of which can vary widely within a structure. There are also substantial differences in the expansive potential and rate within similar parts of a structure nominally all built with the same mix. Some pours may show no apparent damage while others may be severely damaged by cracking. The general principle for managing structures must be that measured structural behaviour on site provides the best indication of rates of deterioration and when the rate of ASR damage is slowing.

10.5 Monitoring of changes

The effects of ASR can be measured indirectly by:

- overall movement of structural members either absolutely or as a differential between adjacent members
- overall surveys recording the extent of uncracked and cracked areas
- the monitoring of the change in widths of cracks in selected areas
- measuring the strains in the concrete over chains of typically 5 to 15 gauge lengths of 200 mm
- measuring reinforcement strains
- changes in pulse velocity in the concrete (however, the interpretation of the data is complicated by the presence of reinforcement).

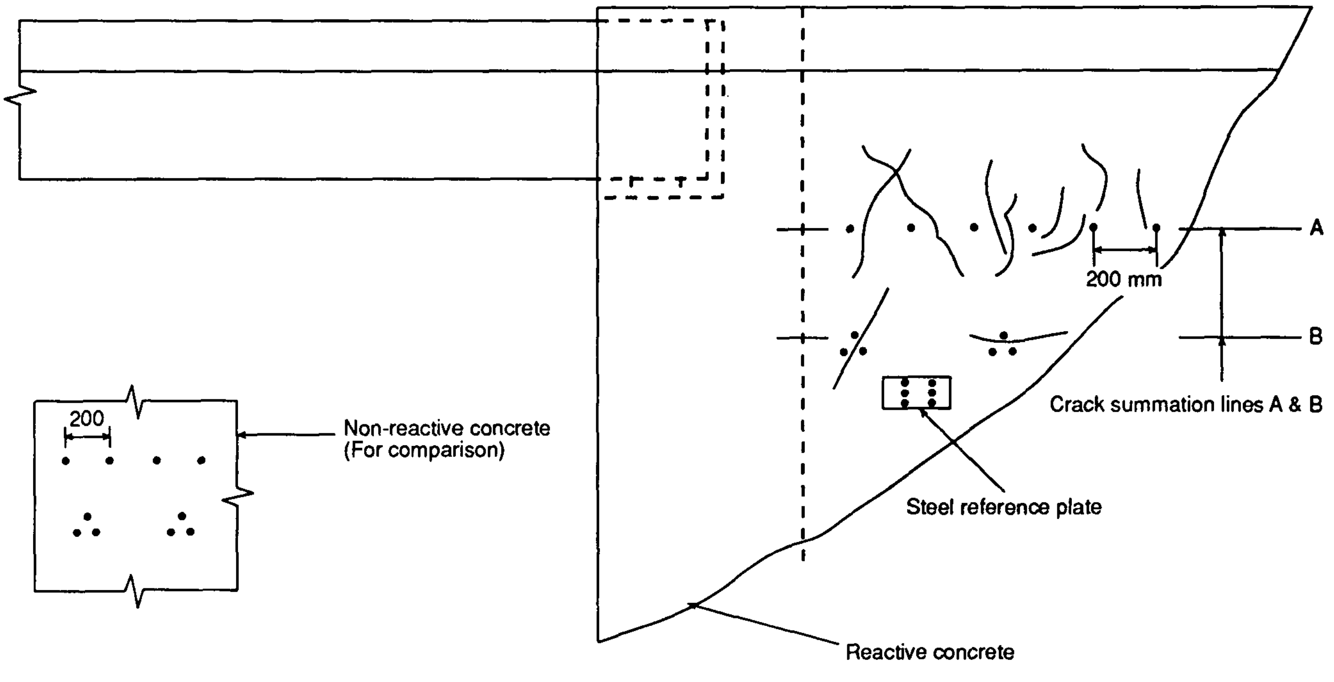


Fig. 16 Crack summation

Examples of methods used are:

- movements relative to inverted pendulums
- the measurement of change in stress in inserted reinforcement[10.8,10.9]
- expansion measurement on long lengths
- measurement of concrete across cracks and uncracked lengths
- measurements at expansion joints
- measurements of distortion [10.10,10.11]
- sonic velocity detection. [10.8,10.9]

10.6 Crack and movement monitoring

Fig. 16 shows how cracks and movements on a structure can be monitored. The technique adopted needs to be tailored to the structure and the following points need particular attention:

- All structures have sizeable temperature, moisture, shrinkage and swelling movements regardless of ASR. These will change both overall lengths and crack widths and must be determined and compensated. Annual cycles of concrete drying shrinkage and recovery of 0.3 mm/m are common. This compares to field ASR-affected concrete free expansion rates of typically 0.05 to 0.2 mm/m per year (i.e., giving 0.5 mm/m to 2.0 mm/m in 10 years). Fig. 17 shows a typical long term record of strain and crack movement for which no compensation for moisture or temperature has been made. The strain recorded in Fig. 17 is adjacent to and parallel to the crack.
- All measuring gauges are subject to changes in length, wear and tear and potential vandalism so sufficient robustness, durability and long term stability and/or recalibration is essential.
- The difference in temperature between metallic surface gauges and the more stable thermal conditions of the interior of the concrete can create substantial apparent strains which must be allowed for.
- Except in the most severe cases with rapid movement, it is normally difficult to differentiate changes in crack width due to ASR from the seasonal effects in less than 2 years monitoring. For measurements of overall expansion a longer period is required.

It is essential that a comprehensive set of predictable reference lengths is maintained to check and quantify normal strains and any instrumentation changes. Steel plates, which follow thermal changes, can be firmly attached to the structure, but not restrained, and used to supplement the standard Invar reference lengths for Demec or similar gauges. Preferably some parallel instrumentation should be fitted to similar adjacent pours of uncracked concrete of the ASR mix and to an adjacent structure with normal non-reactive concrete to provide a reference. Demec gauges may be arranged either as straight chains typically of 5 to 15 gauge lengths of 200 mm or as triples (triangles) of 50 mm gauge lengths which enable the crack opening and shear to be measured and the uncracked base length to be used for correcting crack movement for moisture, temperature and ASR strains. Fig. 17 is an example showing that some cracks will open for a period of years then become stable or vice versa. In instrumenting one location it is advisable to use an array of not less than 20 gauge lengths on one pour of concrete because of the very high variability of movements encountered. Normally this should cover a range of crack widths from the finest to the widest. It provides a useful correlation with the procedures referred to in subsection 7.3.2 if Demec gauges are fitted to cracks on the same lines chosen to measure crack widths for crack summation.

Overall movements on large structures may be recorded using a slide gauge or by measuring the movement across expansion joints using micrometer or Vernier gauge between stainless steel pins. On some structures with large steel members the differential movements may be recorded relative to the steel. When monitoring cracks, the following will be found useful:

- It is important to log the absence of cracks so that there is a positive reference should some be found to develop later, as well as changes in their extent and width. Crack width gauges and microscopes can be used to record crack width at specific locations for crack summation for estimating expansion to date but are not sensitive enough to detect

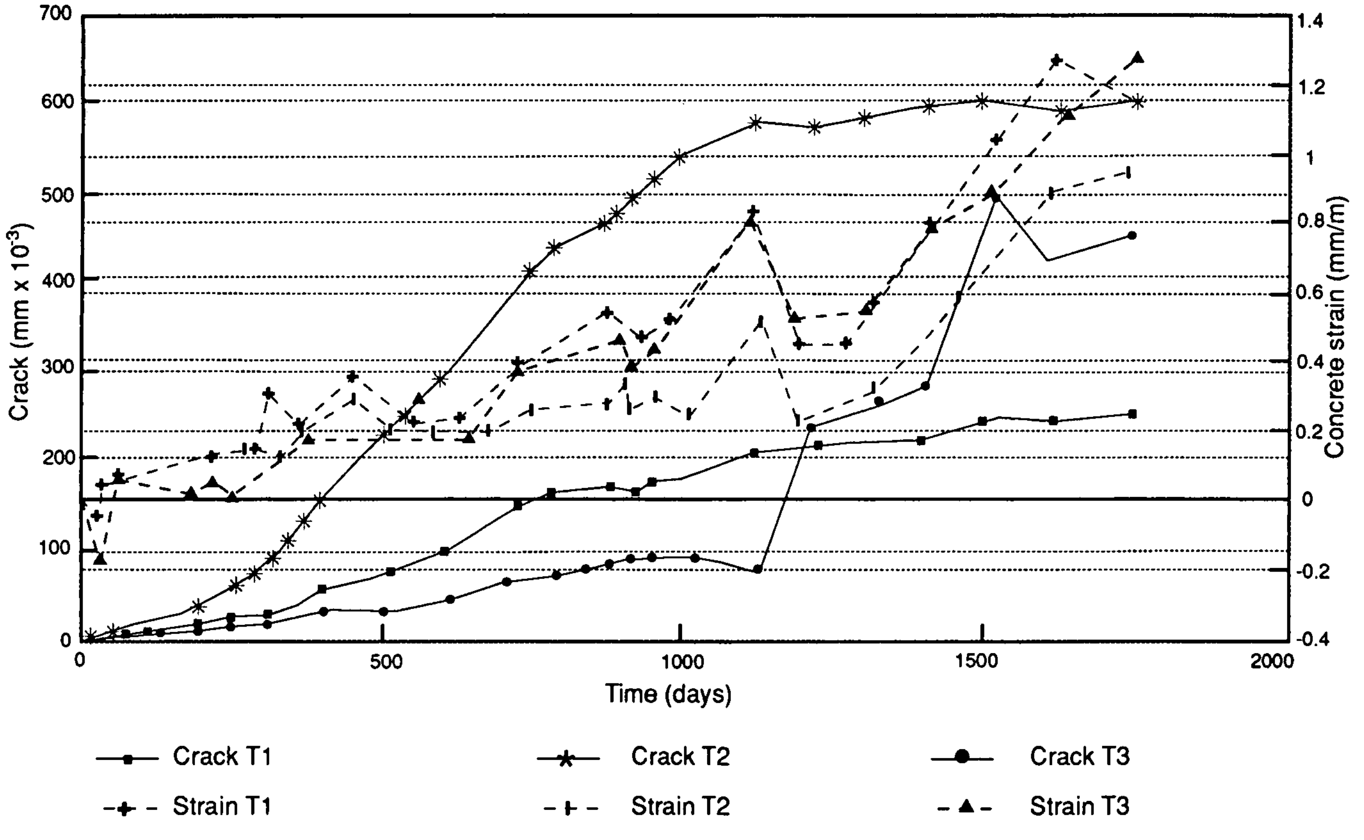


Fig. 17 Examples of concrete strain and corresponding crack movements for adjacent Demec triplets

crack growth rates except over periods of 5 to 10 years on the more severely reactive structures.

- Changes in crack geometry may provide warning of developing damage. Particular note must be made of cracks and the orientation of cracking relative to structural stress fields and reinforcement in sensitive structural elements for use in structural assessment.
- The exact locations for measurement must be clearly marked on the structure and fully described so that re-measurement by different personnel is simplified.
- A clear numbering system and a plotting proforma are essential for the consistent recording of data over a long period.
- Monitoring should also include the recording of deformations, movements and clearances at joints.

The suggested frequency of monitoring depends on the structural severity rating as discussed above and summarised in Table 5 (Chapter 6) and Table 7.

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Part III

11 Research needs

The Task Group believes the following areas of research should have priority. The research must achieve not only a better understanding of the problem, but also more cost effective diagnosis, appraisal, management and repair of affected structures. The focus of research should now switch from well detailed structures to those less well detailed and also indicate whether structures with little or no reinforcement, such as dams and foundations, prove sensitive to lower levels of expansion than is tolerable in normal reinforced concrete. The following should be addressed using desk, laboratory and field studies:

- how much damage has already occurred?
- how much future damage can arise and at what rate?
- what are the changes in the characteristics of the concrete as the reaction develops and how should these changes be considered in the appraisal of the structure?
- is the 'prestressing' action of ASR permanent or does it decay with time?
- what inspection procedures are required?
- what action can be taken to slow ASR damage and secondary corrosion and frost damage?

In particular, the following additional information is required

- comparison between long term field observations and the results of laboratory tests. Particular attention should be paid to the laboratory phenomena of scaling and acceleration.
- presentation of the effects of ASR in characteristic terms to permit easy correlation with BS8110, BS5400, Eurocodes and other documents written in limit state terms.
- improved methods of testing for bond
- improved methods of testing for aggregate reactivity.
- the relationship between ASR and other effects such as frost damage and reinforcement corrosion
- evaluation of cladding and coating techniques used to prevent or reduce ingress of moisture into structures
- the effects of restraint on expansion
- the relationship between expansion to date and loss of stiffness
- bearing strength.

Appendix A Summary of test procedures

A1 General

A1.1 Summary of overall testing

This appendix expands on Table 6 in Chapter 7. It sets out the basis on which a programme for physical site and laboratory testing can be developed to provide the data necessary for the structural assessment of a particular structure. It is complementary to the diagnostic testing covered by the BCA Committee. The testing can be selected to answer specific questions on the basis set out below. The extent of the testing required will depend on the structural severity rating of the critical parts of the structure.

A1.2 Why has damage occurred?

- Petrographic examination for minerals associated with gel and microcracking.
- Alkali audit based on analysis of concrete and source materials for alkali from cementitious materials, aggregate, water and admixtures and for the effects of alkali migration (wet chemistry and SEM-EDXA).
- Moisture determination of moisture levels in the structure (weight uptake of cores, wooden r.h. plugs).

A1.3 How much physical damage has occurred?

- Measurement of overall and differential movements particularly those which have caused distortion or damage.
- Determination of cracking as sum of crack width in mm/m length related to stress, reinforcement and restraint.
- Examination of cracking and micro-cracking in cores.
- Measurement of Stiffness (E_c) of cores using Stiffness Damage Test (SDT).
- Measurement of UPV insitu and on cores.
- Measurement of uniaxial compressive (f'_c) and tensile (f_t) strength and strain to failure.

A1.4 How much further damage may occur?

- Expansion tests, with monitored water up-take, on cores or samples maintained in a damp environment at similar temperatures to those in the structure, either on site or in the laboratory
- Expansion tests under stress or restraint
- Comparison of E_c and UPV before and after, and f'_c and f_t after expansion test.

A1.5 How fast will further damage develop in the structure?

- Rates in expansion test adjusted for site conditions
- Monitored rates of crack growth, expansion, movement and UPV change measured on the structure
- Rates of moisture supply and migration in the structure.

A2 Summary of core expansion test procedures

The expansion testing of cores to assist in the prediction of future structural behaviour needs to be matched to the range of conditions in the structure. It is a longer term exercise than the rapid 38°C diagnostic expansion test. The planning and interpretation of structural expansion test data needs to be closely co-ordinated with the site monitoring of movements.

Cores of 70 mm or 95 mm nominal diameter (i.e. 75 mm and 100 mm core holes) are preferred, although larger cores should be tested in specialised circumstances, e.g maximum aggregate size > 25 mm, etc. The cores are removed from selected representative areas of the structure, i.e. those quantified in terms of environment and degree of cracking and with due regard to restraints and stress state, by diamond drilling. Cores for structural expansion tests should not penetrate either cracks or reinforcement.

The process of coring necessarily wets the concrete. The core should then be washed, wiped clean and then allowed to surface dry in freely ventilated shade before it is indelibly marked with a core number, section identification and orientation, wrapped in clingfilm and sealed in polyethylene bags and wrapped or boxed to prevent damage during transportation. Weight and length changes of the concrete can be monitored while the core is in transit by weighing the core and fitting temporary Demec points on jubilee clips immediately after coring. Tests for expansion during coring require a specialist procedure with embedded strain gauges.

At the laboratory the cores should be unwrapped in an environment which will not result in rapid drying.The appearance of the concrete is recorded in detail and Demec points are fitted. The concrete is then cut and labelled. Preferred lengths are:

- 180 mm for 70 mm diameter cores
- 220 mm for 95 mm diameter cores.

These correspond to approximately 2.5 times the diameter and permit three rows of three 50 mm Demec strain gauge lengths. In addition to the preparation of the sample, a core may be cut into a number of pieces which are suitable for other tests, for example for chemical and petrographic analysis or compressive strength testing. The sample to be used should be then photographed, including close-ups of significant cracks in the matrix. SDT and UPV tests may be specified prior to expansion testing.

The expansion test procedure is based on the measurement of the average of the dimensional changes and their variability. These are then related to the changes in moisture measured as variations in weight of the core throughout the duration of the test. All concrete expands and contracts on wetting and drying. If the concrete is dry at the start of an expansion test, the recovery of drying shrinkage will increase the apparent expansion. If this reversible shrinkage recovery exceeds 0.3 mm/m then the total expansion will need to be adjusted. If the weight uptake on a core on test exceeds 15g/kg then the drying shrinkage recovery may exceed 0.3 mm/m and a check should be carried out at the end of the test by drying the core down to its initial weight prior to the test to determine the recoverable shrinkage. Any recoverable shrinkage in excess of 0.3 mm/m should be deducted from the total expansion to give the ASR expansion values for use in appraisal of the structure.

The expansions from shrinkage recovery are more uniform over the length of the core than those due to ASR. This may help in the interpretation of marginal expansion test results in the 0.6 mm/m - 1.0 mm/m range where 6 or 9 Demec gauge lengths can be compared. All expansion data must be corrected to constant temperature so that thermal expansions are excluded. The procedure can be modified to accommodate many combinations of moisture, temperature and restraint in order to simulate conditions encountered on site. The most frequently used environments are as described in Tables A1 (a) and (b).

Table A1 Environments used in core expansion/shrinkage testing

Table A1(a) Moisture environments

Moisture environments	*Comments*
100% r.h.	Not generally recommended due to uncertainties in achieving the required environment
Water supply	a) Capillary Core sits in a sealed, inclined container with free water maintained at 10g/kg of core weight b) Wetted As (a), but the free water is poured or sprayed over the core, which sits in the drained surplus
Wrapped	Core is wrapped in wet cloth in a sealed container. This more complicated procedure makes weight monitoring of the core more difficult
Immersion in alkali	Rarely used, except for testing concrete exposed to alkalies in the service environment
Immersion in water	Not recommended due to leaching of alkalies from the concrete
Natural on site	Cores are: a) exposed on racks b) in troughs (to wet and dry in collected rain), or c) sheltered
Drying	In ovens freely ventilated to ambient vapour pressure at the test temperature 30°C, 50°C or 105°C

N.B. Weight changes must be monitored as a measure of initial uptake and moisture fluctuations through all tests from time of coring.

Table A1(b) Temperature environments

Temperature environment	*Comments*
38°C	Only recommended for diagnosis; may initially accelerate the reaction but maximum reaction can be reduced
20°C	Typical UK summer temperature. Slower initial expansion but the long term expansion is more realistic
Lab ambient	Generally between 20°C-25°C
13°C	Average UK temperature
5°C	Average UK winter temperature
Freeze/thaw	Expansion and deterioration may be increased by the additional action of freeze/thaw cycles
Natural	Cores are exposed to natural temperature cycling on site

N.B. A maximum/minimum thermometer record should always be kept.

Cores may be restrained using steel with an area equivalent to either 1%, 2% or 4% of the concrete cross-section with a preload typically in the range 1 N/mm^2 to 5 N/mm^2 to measure the effect of stress on expansion.

Measurements should be taken three times before being placed on test in a damp environment and then on a daily basis for the first 5 days, then every 2 weeks up to 12 weeks, then on a monthly basis. Initial assessment of the severity of the reaction is made at 12 weeks (4 weeks with expansive cores). Further assessment of the potential for long term expansion can be made at 3 to 6 month intervals as the test is maintained in parallel with site monitoring.

Wear of equipment has caused problems when monitoring the slow long term movements. This problem can be overcome by quantifying the wear by measurements made on a set of 9 Demec gauge lengths fixed to a steel plate. The weight of the plate is similar to that of a core, i.e. approximately 1.5 kg, and this provides a check on the weighing procedure and equipment. The plate is read along with the temperature at the beginning and end of each day's measurements on the cores. Any movement recorded on the steel plate gauge lengths is then attributable to either temperature effects or wear on the gauge. The difficulties of maintaining Demec gauge calibration over long periods of use make it necessary to use steel plate data to recalibrate gauges after replacement or reconditioning.

At the conclusion of the expansion test the core can be retested for SDT, UPV, destructively tested for strength including strain to failure, chemically or petrographically analysed, or it may simply be stored for future reference. There are similar shrinkage tests where concrete with or without ASR is exposed to wetting and drying environments. The tests simulate a range of temperature and relative humidities, including 20°C/50% r.h., 30°C or 50°C/oven drying, etc.

A3 Strength tests

When determining the uniaxial compressive strength it is preferable to use a cylinder with a length to diameter ratio of 2.5:1 to 3.0:1 or by means of a special end-loading condition if only short cores are available. The results of the standard short-core compression test may be misleading, because the apparent strength enhancement, due to platen restraint, is more pronounced for ASR-affected concrete.

Where possible, cores should also be taken from unaffected areas, e.g. those parts sheltered or protected against moisture, to enable direct comparison of strength to be made.

A measure of the degree of tensile strength loss can be obtained from indirect cylinder splitting test on a comprehensive set of cores. A torsion shear strength test on cores has been developed which is less sensitive to irregularity of site cores and tests a larger volume.

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