MANAGEMENT OF EXPANSIVE CHEMICAL REACTIONS IN CONCRETE DAMS & HYDROELECTRIC PROJECTS

Cover photograph: Diamond wire saw cutting equipment in use re-cutting slots in the gravity dam intake at Mactaquac Generating Station, New Brunswick, Canada, 2011

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The lead authors of the final version of the Bulletin are Dr. Robin G Charlwood (USA), Dr. Mario Berra (Italy) and Eng. Francesco Amberg (Switzerland) with additional review and inputs from Dan Curtis (Canada), Dr. Jerzy Salamon (USA) and Eng. Riccardo Stucchi (Switzerland).

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The case history summaries are based on presentations at the ICOLD Workshop on AAR, Fontana, TN, USA, 2013; the ICOLD/SPANCOLD/International Journal of Hydropower & Dams Workshop, Granada, Spain, 2007; USCOLD International Conference on AAR, Chattanooga, TN, USA, 1995.

FOREWORD

This Bulletin “Management of Expansive Chemical Reactions in Concrete Dams and Hydroelectric Projects” is to update and amplify the information of the ICOLD Bulletin 79 published in 1991 and presents a review of the methods and effectiveness of recent practices in the management of expansion effects in concrete dams and spillways.

This new Bulletin updates our understanding of the alkali aggregate reactions (AAR) and their effects on large dams and hydroelectric projects based on recent experiences and considers a broader set of chemical expansion phenomena than alkali-silica reactions (ASR) or alkali-carbonate reactions (ACR) that have been found to cause expansion in concrete, including the reactions due to sulphur compounds and other possible phenomena. In addition, it will address the important issue of supplementary supplies of alkalis released from the aggregates in some cases which can significantly extend the duration of AAR.

The current state of the art for AAR and Sulphate Attack prevention and best practices for assessment and testing to minimise the risk of AAR and Sulphate Attack occurring in large dam and hydroelectric plant structures is not addressed in this Bulletin.

The options for managing expansive reactions in existing dams and related hydraulic structures are reviewed in this Bulletin. These include testing and modelling as essential precursors to the development of a management strategy and planning of any physical interventions.

The effectiveness of management options is reviewed in terms of field performance of noteworthy case histories, including identifying both positive and adverse side effects, to assist owners select and plan appropriate strategies in the future. The management options for ongoing expansive reactions in large dams are in fact quite limited, there is no known practical way to stop continuing reactions in such large structures. In these situations, it becomes a matter of continuous management and monitoring of the structures to maintain safety and operability.

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March 2019

1 INTRODUCTION

1.1 Background

This Bulletin “Management of Chemical Expansion of Concrete in Dams and Hydroelectric Projects” is to update understanding of expansion phenomena in concrete dams and hydroelectric projects presented in the ICOLD Bulletin 79 published in 1991 and presents a review of the methods and effectiveness of recent practices in the management of expansion effects in concrete dams and spillways. It will consider all kinds of chemical expansion phenomena, including that due to alkali aggregate reactions (ASR and ACR), such as reactions due to sulphur compounds and those in which free lime or magnesia has a role.

Since that time, many new cases of expansion instances involving expansive chemical reactions in dam materials affecting dam safety and operations have been reported at the USCOLD 1995, ICOLD/SPANCOLD 2007 and ICOLD/USSD 2013 Workshops, among other meetings. These situations arise from the fact that sufficiently reliable testing and prevention methods for concrete in new dams have only recently been developed, and in fact, are still under development. This current situation is accentuated by the fact that a large number of existing dams are ageing and that most of the pathologies evolve very slowly.

Only a few years ago an ICOLD paper stated that nearly 10% of concrete and masonry dams damaged by ageing undergo expansion. Recent experience suggests this figure is higher as many cases of dam expansion have been reported lately and it must also be noted that the phenomenon can also occur even if it is not directly apparent. Over 100 large dams and hydroelectric projects have been identified to be seriously affected by alkali-aggregate reactions in terms of dam safety and operations.

This pathology therefore currently accounts for one of the most significant scenarios with respect to the deterioration and long-term performance of concrete dams and hydro projects.

The economic impact is difficult to quantify. In the majority of the cases until now it was sufficient to intensify the monitoring with new instruments, with more detailed safety assessment and with specific laboratory and site investigations. However, in the last two decades, effective interventions have performed in a number of cases, typically by means of slot cutting to release the horizontal stresses (e.g. Fontana, Hiwassee (USA), Beauharnois, Mactaquac (Canada), Chambon (France), Poglia, Pian Telessio and Colombo (Italy), Illsee, Salanfe (Switzerland)) or rehabilitation by grouting and anchoring. Only very limited cases of decommissioning and reconstruction of the dam have occurred (e.g. American Falls (USA); Maentwrog (UK), Lady Evelyn (Canada) and Sera (Switzerland)).

Over the past 20 years a number of conferences and workshops have been held to focus on the nature and impacts of expansive reactions on dams and hydroelectric projects and provide the knowledge base for this Bulletin update.

USCOLD held a special meeting in Chattanooga, Tennessee in 1995 on “Alkali-Aggregate Reactions in Hydroelectric Projects and Dams” at which a worldwide survey of over 100 cases was presented with reports on diagnosis and remediation of a number of large dams and power plants.

A special “Workshop on Chemical Expansion of Concrete in Dams and Hydropower Projects” was organized by the ICOLD Committee on Concrete Dams jointly with the SPANCOLD Committee on Concrete for dams and hosted by the International Journal for Hydropower and Dams in Granada, Spain in October 2007. Invited experts made presentations on the key topics.

In 2013 a Special ICOLD Workshop on Alkali-Aggregate Reactions in Dams was held at Fontana Dam in Tennessee, USA after the ICOLD 2013 Seattle Annual Meeting. At this workshop a number of presentations were made on case histories and recent developments.

The information and noteworthy case histories collected in these workshops is incorporated into this new Bulletin.

1.2 The Current Situation

In 1991 ICOLD Bulletin 79, "Alkali-Aggregate Reaction in Dams – Review and Recommendations" discussed the problem of Alkali-Aggregate Reactions (AAR) in dams and referred to a number of cases worldwide. In 1992 over 100 large dams and hydroelectric projects were identified to be seriously affected by AAR impacting dam safety and operations [1.1]. Recent experience [1.2] [1.3] [1.4] [1.5] [1.6] [1.7] [1.8] suggests this figure is higher as many cases of dam expansion have been reported lately due to various forms of expansive chemical reactions in the concrete and it is now realized that the phenomenon frequently continues for longer periods of time than was initially realized. Given the pervasiveness of the expansion problem and realization that most dams will be required to continue to operate well into the future, way beyond the 30 to 50-year service life that was frequently considered initially, these long-term behaviour issues are going to become increasingly important.

Most of the "alkali-aggregate reactions" (AAR) cases are "alkali-silica reactions" (ASR) but other less well recognized chemical reactions appear to be playing an important role as well. The characteristics of these "other reactions" and opportunities for prevention in new structures or management in existing structures are not as well understood as those for ASR.

A key aspect, that is becoming increasingly apparent in existing structures, is that in many cases the reactions and associated expansions appear to be quite small but are continuing unabated after 40 or more years. Only a few of the known cases have stopped expanding. The magnitude and duration of the residual expansion can be a key factor in estimating the remaining service life of a dam but is very difficult to estimate in the cases where expansion is continuing. The notion that with AAR the alkali source was the cement is now realized to be only part of the story; in many cases alkalis become available from certain reactive aggregates with time. This will clearly affect the duration of the reactions and in many cases will cause the reaction to continue effectively indefinitely. In some structures with potentially reactive aggregates, Supplementary Cementitious Materials (SCMs) (such as pozzolans and fly ash), have been used in sufficient quantities and appear to be effective by increasing the amount of alkalis fixed by the cement hydrates and so lowering the pH of the pore solution, such that the reactivity of the aggregates (and consequently their release of alkalis) is minimized although this needs to be confirmed as discussed below.

The situation in many cases where expansion is continuing is that ongoing management of the behaviour is required. This is the focus of this Bulletin.

Upon reviewing the current state of knowledge of expansion phenomena, it can be concluded that greater knowledge is required on several of the fundamental processes involved in concrete expansion and how it can be avoided or managed if they are present. Deficiencies in the knowledge and the treatment of chemical expansion phenomena include the following issues.

1.2.1 *Diagnosis in Existing Dams*

Information arising from visual inspections, monitoring and technical records is extremely important. On-site prospecting, tests and material analyses are very relevant. The use of mathematical models is also very helpful for assessment purposes.

With regard to tests and analyses, the complex reactivity in the concrete is difficult to reproduce in the laboratory conditions where reproduction and measurement of the broad range of variables is limited and difficult and are not even well understood at this time.

Clearly laboratory activities need to be improved. The knowledge currently available to assess reaction factors and mechanisms is still not exhaustive enough; sometimes, no clear facts can be found with regard to the chronology of the formation of compounds found and the product which triggers the expansion phenomenon when the analysis is performed. Furthermore, most analyses, particularly those focused on the reactivity potential of aggregates, are neither reliable nor applicable to all types of minerals. In addition is difficult to assess the long-term expansion damage in a structure by means of short-term laboratory tests and their conversion to actual structure, In particular, it is difficult to determine the residual expansion in practice.

Expansion characteristics therefore make it difficult to establish a detailed diagnosis and laboratory tests which produce results that can be used with confidence to assess the extent of the problems and develop remedial action. In fact, traces of anomalous reactions can sometimes be found in sound concrete even though the magnitude of the reaction is not sufficient to produce significant problems to the serviceability or structural function. Moreover, the fact that tests performed on different materials are generally qualitative and cannot be applied universally adds further complications to the conclusions drawn in the analysis.

There is a limited number of reliable internationally applicable methodologies for performing diagnosis and prognosis on concrete structures and each country and even laboratory uses their own procedures, which are very often “tailored” to address only a specific type of structure or to solve only specific local types of expansion damage. A state of the art for diagnosis of expansion phenomena in existing concrete structures has been presented in the new RILEM “Guide to diagnosis and appraisal of AAR damage to concrete in structures” published in 2016, where it is highlighted that a complete diagnosis process requires a series of steps involving multi-disciplinary tests and investigations performed in a systematic manner. Furthermore, the effects of expansion phenomena need to be considered as part of a complex of non-structural and structural actions and deteriorations which lead to a loss of performance or decay of concrete structures.

Based on these guidelines, this Bulletin will focus on the various steps of the diagnosis process to be adopted in the case of expanding concrete dams, providing an overview of the symptoms coming from routine inspection, the visible effects of damage and the laboratory and field investigations required. The assessment of the severity of the situation and the potential for future expansion will be also considered.

The behaviour of expanding concrete dams and the most appropriate expansion modelling for structural analysis will be discussed in the subsequent sessions. They are required for an accurate assessment of the actual behaviour of the expansive concrete dam and to identify the suitable strategies for its management

1.2.2 *Prevention*

The available assessment and test procedures in the prevention of expansion phenomena in concrete dams are not as reliable as is required and continue to be not well understood and accepted in many cases. A key issue is the required lead time to carry out suitable tests at the beginning of construction.

There is presently a lack of nation-wide studies on aggregates containing specific tests to take into account behavioural differences in similar aggregates available in different countries.

RILEM’s technical committees on AAR have made important contributions to this requirement through their recent publication [1.9]. Research continues on the development of assured preventative measures for new constructions.

This selection of appropriate test and precautionary measures for large dams was addressed by Sims et al [1.10] and will be the subject of future publications and is not discussed further in this Bulletin.

1.2.3 *Management in Existing Dams*

Large numbers of corrective actions have been carried out on dams, ranging from minor repairs to major structural modifications and even total replacement in a few cases.

Instrumentation and monitoring combined with direct inspection, field and laboratory tests, and computer simulation are required as essential first steps in planning a management strategy to deal with the ageing process.

Rehabilitation actions have made it possible to maintain the level of safety of the dams in most cases, although experiences have shown that repairs are not always definitive, and in some cases might be detrimental, and normally require ongoing management and repeated interventions.

A compressive strength decrease may be observed in older cases of very reactive AAR concrete (very high expansion rate). For the expansion rates usually found in slowly reactive aggregates of concrete dams, the reduction of compressive strength is not clearly observed. Tensile strength and moduli of elasticity are more affected than compressive strength.

1.2.4 *Need for Research and Development*

Reviews of the current state of expansion phenomena show that greater knowledge is required on the fundamental processes involved in concrete expansion and how it can be avoided or managed if it is present. Dam managers are concerned about the lack of uniformity found with regard to the degree of reliability in procedures currently carried out within the diagnosis. Furthermore, the remedies needed to deal with this type of anomaly are not clear. This uncertainty must be addressed. Therefore, this problem requires a concerted effort to improve the situation.

In addition, expansion data should be compiled internationally in a systematic manner to allow reliable scientific analysis and a basis to produce practical recommendations for avoidance and management of the phenomena.

In summary, deficiencies in the knowledge and the treatment of chemical expansion phenomena include the following topics:

- Chemical reaction development and formation mechanisms.
- Laboratory testing and analyses of materials.
- Reliable mathematical models and parameters for diagnosis and control.
- Practical procedures for determination of model parameters
- Methods for estimation of residual expansion in affected dams.
- Techniques for long term rehabilitation of affected dams.
- Preventive measures of phenomena for new dams.

1.3 The Objectives and Scope of this Bulletin

Based on the background and current situation outlined above, this Bulletin “Management of Chemical Expansion of Concrete in Dams and Hydroelectric Projects” addresses the following:

- It updates our understanding of the Alkali Aggregate Reactions (AAR) and their effects on large dams and hydroelectric projects and considers a broader set of chemical expansion phenomena than just Alkali-Silica Reactions (ASR) and Alkali-Carbonate Reactions (ACR) that have been found to cause expansion in concrete, External and Internal Sulphate Attack (ESA/ISA), including Delayed Ettringite Formation (DEF), and those reactions in which free lime or magnesia has a role. In addition, it will address the important issue of supplementary supplies of alkalis released from the aggregates in some cases which can significantly extend the duration of AAR.

- The potential physical effects of these expansion mechanisms on dams and hydroelectric facilities are reviewed in terms of observations on a number of illustrative case histories. These include effects on stability of dams, powerhouses and associated structures; reductions in clearances of discharge facility gates; interference with operating equipment clearances, such as turbine runners and generator rotors and impact on deformation of embedded components.
- Based on international guidelines, this Bulletin will deal with the diagnosis process to be adopted in the case of expanding concrete dams, considering the various steps required to this aim, from inspection to laboratory and field investigations,
- Structural analysis and the modelling of expansive phenomena are also discussed. This plays and important role in confirming the diagnosis, assessing the dam behaviour and optimizing the dam management.
- Options for managing these expansive reactions when they are occurring in dams, related hydraulic structures and hydroelectric projects and their effectiveness are discussed. These are in fact quite limited, there is no known practical way to stop these reactions in such large structures. It becomes a matter of "managing" the structures to maintain safety and operability.
- Monitoring and performance parameters are identified to facilitate their consideration in safety assessments and monitoring planning.
- Techniques such as anchoring for stability and slot cutting to de-stress critical areas and/or maintain clearances for equipment have been used on a number of dams.
- Based on the practical experiences over the last 30 years or so, the short- and long-term effectiveness of these techniques are reviewed, and conclusions and recommendations made for the management of new cases.

1.4 Related ICOLD Bulletins

Bulletin N° 22, Pozzolans and Slags for concrete for large dams (1972)

Bulletin N° 36a, Cements for Concrete for Large Dams (1981)

Bulletin N° 71, Exposure of dam concrete to special aggressive waters (1989)

Bulletin N° 79, Alkali-Aggregate Reaction in Concrete Dams (1991)

Bulletin N° 107: Concrete dams – Control and treatment of cracks (1997)

Bulletin N° 136, The Specification and Quality Control of Concrete for Dams (2011)

Bulletin N° 145, The Physical Properties of Hardened Conventional Concrete in Dams (2012)

Bulletin N° 165, The Selection of Materials for Concrete for Dams (2013)

1.5 References

[1.1] R.G. Charlwood, S.V., Solymar, D.D, Curtis, "A Review of Alkali Aggregate Reaction in Hydroelectric Plants and Dams", CEA and CANCOLD, Fredericton, pp. 1-29, September 1992.

[1.2] USCOLD, Proc. 2nd International Conference on Alkali-Aggregate Reactions in hydro-electrical plants and dams, Chattanooga, Tennessee, USA, 22-27 October 1995.

[1.3] ICOLD/SPANCOLD/ International Journal on Hydropower & Dams, Workshop on AAR in Dams and Hydroplants, R.G. Charlwood and J.M. Buil Sanz Eds., Granada, 18-19 October 2007.

[1.4] Sausse, J., Fabre, J.P., “Diagnosis of Dams Affected by Swelling Reactions: lessons learned from 150 Monitored Concrete Dams in France” Proc. 6th International Conference on Dam Engineering, LNEC, Lisbon, Portugal, pp. 1087-1100, February 2011.

[1.5] Camelo, A., “The EDP experience on management and rehabilitation of AAR affected dams”. AAR Short Course at the 6th International Conference on Dam Engineering, Lisbon, Portugal, February 2011.

[1.6] Amberg, F., “Performance of dams affected by expanding concrete”. Proc. Dams and reservoirs under changing challenges, 79 Annual meeting of ICOLD, Swiss Committee on dams, Lucerne, CRC Press, pp. 115-122, June 2011.

[1.7] ICOLD/USSD International. Workshop on Alkali Aggregate Reactions, 81th ICOLD Annual Meeting, 20 Aug 2013.

[1.8] French Committee on Dams and Reservoirs, Proc Symposium on the Dam Swelling Concrete, DSC 2017, Chambéry, France, 13-15 June 2017; Sellier, A., Grimal, É., Multon, S., Bourdarot, E., Eds.; Wiley, London, 2017.

[1.9] RILEM State-of-the-art Report, “RILEM recommendations for the prevention of damage by alkali–aggregate reactions in new concrete structures”, Springer, Nixon, P.J. and Sims, I. Eds., vol. 17, p. 168, 2016.

[1.10] Sims, I, Nixon, P., Godart, B., & Charlwood, R., “Recent developments in the management of chemical expansion of concrete in dams and hydro projects - Part 2”, RILEM proposals for prevention of AAR in new dams”, Hydro 2012, Bilbao, Spain, October 2012

2 EXPANSIVE CHEMICAL REACTIONS IN CONCRETE

This Bulletin will focus on the important fundamental long-term issues that are associated with continued operation of dams and hydro-electric projects:

- Definition and characterization of the underlying chemical mechanisms that are claimed to be affecting many dams: Alkali-Aggregate Reactions (AAR) including Alkali-Silica Reactions (ASR) and Alkali-Carbonate Reactions (ACR); sulphate related deteriorations including External Sulphate Attack (ESA), Thaumasite formation, and Internal Sulphate Attack (ISA) including pyrite oxidation and Delayed Ettringite Formation (DEF).
- Identification of mechanisms affecting very long-term AAR expansion: Examples of long term expansion driven deformation will be shown where the rate of expansion is continuing unabated after forty or more years even with very low alkali content cements. The relative roles of alkali supply from various sources including the original cement, alkali release from certain aggregates and alkali "recycling" as the gels transform, will be discussed. Whereas the alkali supply from the cement may start early and be limited in duration, those coming from recycling or release from certain aggregates may become available in significant quantities later and can cause the AAR to start very slowly and show as expansion much later and continue essentially indefinitely, even if the precautions of using low alkali cements were taken. The expected long-term behaviour of concretes using Supplementary Cementing Materials (SCMs) in the presence of alkali release also raises some questions which will also be briefly addressed.

2.1 Alkali-Aggregate Reactions (AAR)

By far the most relevant process leading to the expansion of concrete in dams is Alkali-Aggregate Reaction, AAR. There are two main types of AAR: ASR (Alkali Silica Reaction) and ACR (Alkali Carbonated Reaction) although the latter is far less widespread and it is disputed to be also a form of ASR due to fine and ultrafine inclusions of silica in the dolomitic limestone rocks, possibly in combination with a dedolomitization process [2.1] [2.2]. They may have different characteristics and reaction mechanisms but both lead to expansion and cracking of concrete. Actually, AAR may only be considered as ASR with various forms [2.3]. In the following, when the general Alkali-Aggregate Reaction term and AAR acronym are used, they will mostly reference to Alkali Silica Reaction (ASR), unless otherwise specifically indicated.

In ASR, the alkaline pore solution of the cement pastes, containing almost entirely sodium, potassium and hydroxide ions with very low concentration of other ions such calcium, diffuse from cement paste into reactive (poorly crystallised) siliceous aggregates, causing the dissolution of the reactive silica and producing an amorphous calcium alkali silica gel (CaO-Na_2O/K_2O-SiO_2-H_2O) containing small amounts of calcium. The main silica-reactive minerals and rocks in concrete aggregates from around the world can be found in the RILEM Petrographic Atlas [2.4]. Reaction rims around aggregate particles are initially formed, followed by the deposition within the particles of the gel, which imbibes water and swells (relative humidity content above about 80% is required), inducing stress and micro-cracking the aggregate. This, in turn, causes expansion and micro-cracking also in the surrounding cement paste, with alkali silica gel flowing into the cracks and pores. Near the aggregate the gel contains less calcium and swell to a greater extent. As the gel moves through concrete, it picks up more calcium, which reduces its swelling potential [2.5]. Temperature increases the rate of expansion.

In smaller structures, a supply of moisture extraneous to the concrete itself is necessary, but in larger structures like dams sufficient moisture is probably retained after hydration to initiate, and in many cases maintain, reaction and facilitate on-going expansion, even after 50 years of service life [2.6].

Other aggravating factors that will influence the severity of ASR-induced damage include the synergistic effects of freezing and thawing or other deteriorative mechanisms (paragraph 2.5).

Initially the aggregate expansion is absorbed by the creeping of the cement paste and no macroscopic expansion occurs. This phenomenon leads to the initial quiescent period (initiation phase), where in a new structure no signs of expansion are detected. The observed concrete macroscopic expansion is instead related to the filling of the existing porosity by the swelling gel, to the internal pressure build up and the consequent micro-cracking formation within the concrete, with a reduced material tensile strength. Generally, the level of expansion strain of concrete before the appearance of visible surface cracks is considered about 0.04% [2.7]. However even lower levels of expansion can lead to significant effects in a dam [2.8]. For example, 0.02 - 0.03% expansion over 100 m of a dam concrete represent 20 - 30 mm of movement, probably more than can be tolerated.

The crack pattern is strongly influenced by the presence of confining effects and stress fields, with expansion mainly in the direction of low compressive stresses. For example, less restrained conditions on the concrete surface, compared to the core concrete, normally leads to conspicuous cracks opening on the surface, the “map” or “pattern” cracking, that is often an obvious indicator of ASR. Besides, frequently alkalis are leached from the surface regions, and the differential expansion compared to the core concrete further promote the cracks opening on the surface. The absence of said surface cracking is however not an indicator that the expansion does not occurs. Typically, the surface cracking is observed on slim structural parts while the main dam body often do not show any crack at the surface. Map-cracking can also result from other causes such as drying shrinkage.

ASR is a destructive long term reaction, because of the slowness of water and ion diffusion into the aggregate particles and typically only begins after decades from dam construction. As it progresses over a period of time, either the reactive aggregate is run out or the alkali hydroxides in the concrete are consumed, thereby ending the reaction. Naturally, if the supply of moisture is interrupted, swelling of the alkali-silica gel will cease, even if the reaction between the alkalis and reactive aggregate continues. However, as anticipated, this is not the case for concrete in dams.

Furthermore, even if consumed by the ASR development, the alkali hydroxides in the concrete pore solution are continuously regenerated by another reaction between the formed alkali silica gel and the portlandite of the pore solution. In fact there is now evidence that, with passing time, a portion of alkalis entrapped into the gel exchange for calcium of portlandite and alkalis are released back into the pore solution, presumably fuelling further reaction with the aggregate, providing reactive silica is still available [2.9]. This occurs in the massive concrete and probably not in the case of laboratory specimens where alkali leaching eventually reduces the alkali concentration to a level below the threshold level, that is necessary to sustain ASR. According to this interpretation, as long as portlandite, or the siliceous aggregates, has not become completely exhausted, the ASR reaction in the concrete will probably continue.

However ASR expansion typically ceases while concrete material retains significant reserves of alkali hydroxide and reactive aggregate, if a sort of “chemical or physical equilibrium” [2.10] is reached: for example when the metal alkali concentration or hydroxide ions fall below the threshold level specific of the reactive aggregate (Chapter 3) or the already formed reaction products become dense enough to hinder further permeation of these ions to the reacting silica into the aggregate (reaction rate controlled by the transport rate of the hydroxyl ions trough the gel product layers). This means that expansive reaction could recommence if any environmental or other conditions were to change. In any case, it is difficult to estimate the time when the reaction may cease, if at all [2.11].

The above described behaviour is usually represented through the typical S-shaped curve of ASR expansion versus time, introduced by [2.12] for modelling expansion, as detailed in the following Chapters 3 and 7. It is commonly verified in laboratory tests, where a maximum expansion is reached after few years or less, but it has been ascribed to the fact that alkalis are slowly washed out of small laboratory specimens. On the contrary, there are a number of cases of ASR in concrete dams, even with very low alkali content cements, where expansions are continuing after 40 years or more [2.7] and the long term expansion curve does not flatten out but it rather continues to grow as upward-positive slope linear at the extended age [2.13].

Different reasons may be invoked to explain this long-term behaviour of ASR affected concrete structures, related not only to the above described alkali “recycling” from the alkali silica gel but also to the phenomenon of alkali release from certain aggregates. In fact, enhancement of the alkali concentration in concrete may arise during the service life of concrete dams when aggregates with alkali-bearing minerals (i.e., volcanic glasses, feldspars, micas, clay minerals, nepheline, and zeolites), release their alkalis into the concrete pore solution, over a long time [2.14] [2.15]. The concrete alkali enrichment is a consequence of the alkaline cations release from aggregates. In order to maintain the electro-neutrality of the pore solution, it is assumed that the hydroxide ion concentration increases as well, thus triggering and/or accelerating ASR expansion development, even if the initial alkali content of the concrete is insufficient to promote ASR expansion development [2.16]. This chemical mechanism is generally accepted to explain the observed cases of discrepancy between measured and expected alkali contents in field concrete dams showing late or progressive ASR-induced distress, in consideration of their very high content of aggregates and their required long service life. In concrete dams containing alkali-reactive aggregates, deleterious ASR expansion may occur long time after construction even if low-alkali Portland cement is used as an ASR preventive measure~~.~~

The presence of blended cements, especially pozzolanic cements, or of Supplementary Cementing Materials (SCM) like fly ash or good quality pozzolans in the concrete dams is generally considered as an effective measure for minimizing the risk of deleterious ASR expansion, thanks to the ability of their hydration products to incorporate most of the alkalis initially available in the pore solution of the cementitious matrix. As a result, the alkalinity of the pore solution (OH^- ion concentration) will be significantly reduced. According to the recent RILEM guidelines [2.8], no case of damage by ASR has been to date reported for concrete dams containing high levels of good fly ash. For example no deleterious expansion has been observed at a Canadian dam (Lower Notch dam – Northern Ontario) built with a reactive argillite and 20% fly ash after more than 20 years and some Britain dams built more than 50 years ago with reactive aggregates and SCM are still in excellent conditions [2.17] [2.18].

However, in order to forecast the effectiveness of pozzolanic cements or SCM, even in the very long-term prevention of ASR, it is essential to verify if they are able to incorporate, besides the alkali already present and available in the concrete from the beginning, also those that may become available over time, after decades, due to the slow internal alkali supply. This eventual residual capability of alkali binding by pozzolanic cements or SCM approaching apparent ultimate hydration in old concrete dams (apparently, complete cement hydration) is still uncertain. Nevertheless, the existence of a residual capability of alkali binding by long-term hydrated pozzolanic cements has recently been experimentally verified, at least at laboratory levels and on cement pastes [2.19].

2.2 Sulphate deterioration

The other group of reactions leading to expansion of hardened concrete is caused by sulphates coming from the environment (External Sulphate Attack - ESA) or already present into the concrete mix components (Internal Sulphate Attack – ISA). The expansion is typically caused by the formation of calcium sulphate, as reaction product of sulphate ions with calcium hydroxide of the hardened cement matrix, followed by the formation of ettringite, as reaction product of sulphate with hydrated calcium aluminates. This expansive ettringite is usually termed as secondary ettringite, to be distinguished from the primary ettringite that is a normal by-product of early Portland cement hydration, beneficial in the control of cement setting and non-destructive. Generally, the expansion is described as due to the fact that the volume of ettringite is greater than the reacting. However, this is not correct. When all the reacting species, including water are considered the formation of ettringite leads to an overall decrease in volume. If the water needed for the reaction has to come from outside the system then it can be considered that there is a volume increase, but this alone is not sufficient to cause expansion. If this was the case the formation of other hydrates, such as Calcium Silicate Hydrates (C-S-H) in low water to cement ratio pastes, where water comes from outside would also give expansion, which is not observed. Furthermore, all cementitious materials contain a considerable amount of porosity, in which the ettringite can form harmlessly. For expansion to occur, ettringite must form under conditions of super-saturation and in a region where its growth is confined by other phases

[2.20]. This ettringite takes the form of massive aggregates or bands at the aggregate–cement paste interface, causing de-bonding, or filling cracks and voids. The majority of the individual ettringite crystals in the aggregates cannot be distinguished. Cracking may be intense, and, sometimes, massive secondary ettringite occurs together with Alkali-Silica Reaction. Indeed, it is important to note that primary ettringite may slowly re-crystallize into large crystals in pores over time, but not associated with any cracking. Thus the observation of large secondary ettringite crystals is a common and normal feature of old concrete exposed to water.

2.2.1 *External Sulphate Attack (ESA)*

The most common form of sulphate attack occurs on concretes in contact with water or soil containing high concentrations of sulphate ions. The relatively high concentrations of sulphate ions needed to produce external sulphate attack are unlikely to be found in dams, except possibly in foundations or appurtenant structures as for example power plants, diversion works, concrete lined galleries etc. However, even in such cases, this form of degradation is a progressive phenomenon, starting from the surface, with successive layers being degraded and disintegrating. So it would not lead to massive expansion of the whole structure. In considering the aggressivity of sulphates to concrete it is insufficient to consider only the sulphate ion concentration. The cation or metal ion involved have to be considered as well. The principal sulphate liable to be encountered naturally are calcium, sodium and magnesium sulphates. As concrete is not directly attacked by solid sulphates, but rather by their solutions in water, their solubility (1.2 g SO_3/l, 200 g SO_3/l and 150 g SO_3/l for calcium, sodium and magnesium sulphate respectively) is also of direct interest. It should be noted that although calcium sulphate is relatively insoluble in water, it may form in the concrete through reactions with other soluble sulphates.

Thaumasite formation may sometimes occur as the final stage of external sulphate attack [2.21] [2.22]. Once all the available aluminate has reacted to form ettringite, the sulphate ions, together with carbonate ions, may lead to the formation of Thaumasite. Due to the presence of silicate in its structure in place of the aluminate in ettringite, Thaumasite formation leads to decompositions of the main binding phase, C-S-H, and the concrete disintegrates into a “mush”. Thaumasite formation is very slow, although it seems to be favoured by low temperatures. The only serious cases of Thaumasite formation have occurred in very poor-quality concrete with water/cement ratios in the range 0.8 and above. Thaumasite formation is the final stage of degradation in sulphate attack and as such is a progressive surface phenomenon, which will not produce massive expansion [2.23].

Following the above arguments “External” Sulphate Attack (ESA) can be effectively ruled out as a cause of massive expansion in structures such as dams.

2.2.2 *Internal Sulphate Attack (ISA)*

Internal Sulphate Attack (ISA) occurs in a free sulphate environment and can arise from the late sulphates provided by the sulphate contaminated aggregates or by the thermal decomposition of primary ettringite, due to curing at elevated temperatures (>70-80°C) during initial hydration or to the excessive heat of hydration developed in massive structures like dams.

In the case of thermal decomposition, the terminology of Delayed Ettringite Formation or DEF is usually adopted and it is separately discussed below.

The other form of internal sulphate attack is due to release of sulphate from rock containing iron sulphides. With the right amount of humidity and oxygen, ironmonosulphide (pyrrhotite, FeS) and irondisulphide (pyrite, FeS_2) may both oxidise to give hydroxide and sulphates of iron as well as sulphuric acid, in a primary reaction. These sulphate ions may then react, in a secondary reaction, with the compound of hydrated cement paste: in particular with calcium hydroxide to form gypsum and with tricalcium aluminate (C_3A) to give secondary ettringite. Large deposits of ettringite may certainly be found in affected concrete, however it is not yet clear, whether expansion is related to ettringite formation as the degree of supersaturation and restraint are likely to be small or to the formation of iron oxides within composite rocks, which also entails an increase in volume and cracking of the aggregates. Aggregates containing iron sulphide show rusty deposits on the surface, although rusty deposits do not always come from ironsulphides.

2.2.2.1 *Delayed Ettringite Formation (DEF)*

The solubility of ettringite increases with temperature and with the concentration of alkalis in the pores solution. Therefore, instead of ettringite forming as a primary hydration product, monosulphate is formed and higher than normal levels of sulphate are absorbed on the C-S-H phase. When the concrete returns to lower temperatures, sulphate is released by the C-S-H phase and reacts with the monosulphate to form ettringite. Water or moisture is required for the reaction to occur, the availability of which will affect both the rate and the extent of expansion. This is referred to as Delayed Ettringite Formation or DEF. In some situations this ettringite formation may be expansive. The principal effects of DEF are visible displacement and cracking. DEF can also increase the risk of secondary forms of deterioration such as freeze/thaw attack or AAR. To prevent DEF the maximum temperature should be limited. Some standards require a maximum temperature below 60°C, but this is certainly very much on the safe side. Experience shows that very few cements show problems at temperatures below 80°C. In modern dams' measures are taken to restrict maximum temperatures to avoid problems of thermal cracking so temperatures high enough to produce problems of temperature inducted DEF should be unlikely. In recent years, field cases of heat induced delayed ettringite formation in in-situ concrete have arisen only when concrete with a high cement content (>400 kg/m^3) was cast in large masses in summer or in hot climates, under high relative humidity. However, some dams built in the early 1900's may have been subject to quite high temperatures and may be affected by DEF. It is well known that DEF is also often associated with AAR (see paragraph 2.7).

2.3 Concretes whose cement contains free lime or magnesium oxide

Soundness is a physical property of cement paste, which determines the ability of the cement paste to retain its volume after setting is completed. Uncombined or free lime (CaO) and magnesium oxide or free magnesia (MgO) are hydratable cement constituents with expansive behaviour and then represent a source of cement unsoundness.

The presence of free lime in the cement can be due to a defective dosage or a lack of homogeneity of the raw cement materials. A defective clinkerisation process and a too slow cooling of the clinker are other possible causes. Free lime can be, depending on its origin, primary or secondary. Primary free lime was never combined. Secondary free lime comes from the decomposition of tricalcium silicate that occurs when the clinker is slowly cooled down. Both must be distinguished, since the primary is found in the clinker forming isolated and localised agglomerations of relatively coarse grains, and the secondary is uniformly distributed in the clinker, in a fine granular form. Cement free lime hydration is a slow and expansive process. Its effects are also different depending on the free lime type. For primary free lime, hydration is more intense and localised: when incorporated into concrete, problems due to expansion usually occur in the form of pop-outs. For secondary free lime, it is more disperse and extensive. For this reason, the primary free lime is more dangerous than the secondary one. High free lime content may also influence mechanical and physical properties of cement, including compressive strength, setting time and mixing consistency.

Magnesium oxide in cement usually comes from dolomite present as impurity in limestones or from magnesium clay. Except for small magnesium amounts that can be found in solid solution with other clinker components, the rest is in free state: either in vitreous form when the clinker is rapidly cooled, or in crystalline form (periclase) when the clinker is slowly cooled. Free magnesium oxide hydration is even slower than the free lime hydration. It is particularly expansive when found in periclase form and in a lesser extent when it is in vitreous form.

Cement unsoundness is usually detected and assessed by exposing set cement to curing conditions that cause hydration in a short term. In particular, the autoclave test is suitable to provide an index of potential delayed expansion caused by hydration of both free CaO and to MgO, while Le Chatelier test only accounts for the unsoundness due to free CaO.

So far as is known, no deterioration of concrete in dams has occurred due to free lime and magnesium oxide when the cements have been normally subjected to specific soundness tests, within the regular testing procedures aimed at satisfying the cement physical, chemical and durability requirements.

If suitably limited and controlled, the expansive properties of calcium and magnesium oxide when mixed with water, can be effectively used in the shrinkage-compensating cements and concretes, in order to reduce the tensile stresses and the risk of concrete cracking. This has been the case of the MgO concrete developed and applied in the construction of about 30 concrete dams in China in the past decades: the controlled expansion of concrete allowed to limit the thermal tensile stresses, and thus the risk of concrete while the concrete temperature after cement hydration was dropping [2.24]. Recently (2018) a specific Technical Committee has been appointed by RILEM on this item (RILEM TC CEC: “Controlled expansion of concrete by adding MgO-based expansive agents taking the combined influence of composition and size of concrete elements into consideration”).

2.4 Expansion due to water absorption

When concrete absorbs water, it increases its volume and experiences small expansion because of its contact with water. This volume increase normally ranges between 0,001 and 0,002% and reaches values ten times higher only when the concrete is previously dried [2.25]. Expansion is caused by the cement paste as well as by aggregate water absorption. In particular, some aggregates have laminar structure components that can absorb large water quantities between their layers (moisture sensitive aggregate), producing expansions but limits are generally specified by national standards on aggregates to avoid their use in concrete.

With the advance of the absorbed water, the concrete volume slowly and progressively develops following an asymptotic pattern and causes the growing of internal stresses in the structure. It might take several decades before the air pore system of the concrete is fully saturated with water [2.26]. However, given the usual small expansion values, these stresses are easily resisted by the concrete without the appearance of cracks and deleterious massive expansions are not generally observed in concrete dams due to concrete water absorption.

The only known case of concrete dam in which the expansion was attributed, at least initially, to water absorption by concrete is Mequinenza dam, a 79 m high gravity dam built in Spain between 1957 and 1964 . Not recoverable deformations in the central block, with elevation of the top of the dam, displacements towards the downstream direction, closing of vertical joints and cracks formations were observed. The diagnosis process, based on concrete analyses together with a bi-dimensional and tri-dimensional model of the moisture progress and its stress effects on the structure, led to the preliminary conclusions that the expansions were due to water absorption by the concrete 10 to 100 times bigger than normal, until 0.275 %. However, more recently, it was realized that this diagnosis was not correct and a new diagnosis was proposed based not to a massive concrete swelling but to a general foundation swelling, due to lignite water absorption, coupled with local construction effects and crack openings. Signs of Alkali-Aggregate Reactions were found but only in few samples of 2 cores drilled in the dam out of a total of 19 cores and secondary Ettringite and Thaumasite were identified only in the tunnel concrete coatings [2.27] [2.28].

In the case of first years of operation of the Sayano-Shushenskaya concrete dam, a penetration of the moisture into the concrete to a depth of 4-6 m, and the maximum change in the moisture content of the concrete of the upstream face up to 1.5% was measured. The moisture-induced swelling strains at a depth of 0.5 m from the upstream face reached a value of about 0.005% and at a distance of 4.5–5 m a value of 0.0006%. The only calculated effect was the occurrence of compressive stresses, which reached about 1.0 –1.5 MPa on the upstream face, below the seasonal drawdown of the reservoir and up to 2 m in the zone of the variable water level [2.29].

2.5 Freeze - Thaw

Hardened concrete subjected to frost action (freezing and thawing cycles) can expand due to a physical phenomenon and be damaged either by scaling on the surface or by internal deterioration with reduced compressive and tensile strength of concrete. As already stated in the ICOLD Bulletin n° 145 (“The physical properties of hardened conventional concrete in dams”), the deterioration by frost action on a saturated concrete is related to the transition phase from water to ice, inside the concrete, and to the correspondent increase of volume (around 9%). If the degree of saturation (% of water filled pores) is less than 80 to 90% this increase of volume could be sustained by the concrete without any damage. On the contrary if the degree of saturation reaches these threshold percentages, the ice formation is able to induce an internal tensile stress state that can lead to concrete deterioration, with cracking and spalling.

Usually the most affected zone is on the upstream face, along the water line, for a thickness which may reach a few tens of centimetres, depending on the temperature conditions. Fig. 2.1 shows the degree of saturation of a 55-year old concrete dam versus the distance from the upstream dam surface. Measures were carried out on concrete cylinders taken from the upstream face at four different vertical locations: above the normal water level (-1 m), at the normal water level (0 m) and at depths of 10.5 and 18.5 m. The results showed that an air-entrained concrete in the upstream face is more or less saturated at depths of 10.5 and 18.5 m. At the normal water level (0 m), the saturation levels in the outermost 200 mm of the upstream face are above capillary saturation (0.79). Above the normal water level (-1 m), the saturation levels are below capillary saturation. At a distance of 150–200 mm from the upstream face, the two latter profiles converge at a saturation level of approximately 0.80 [2.30]. These degree of saturations profiles in a concrete dam are of particular interests also for the other expansion phenomena above considered, in particular AAR and sulphate attacks.

As for the chemical External Sulphate Attack (ESA), the physical freezing and thawing action can be effectively ruled out as a cause of massive expansion in concrete dams.

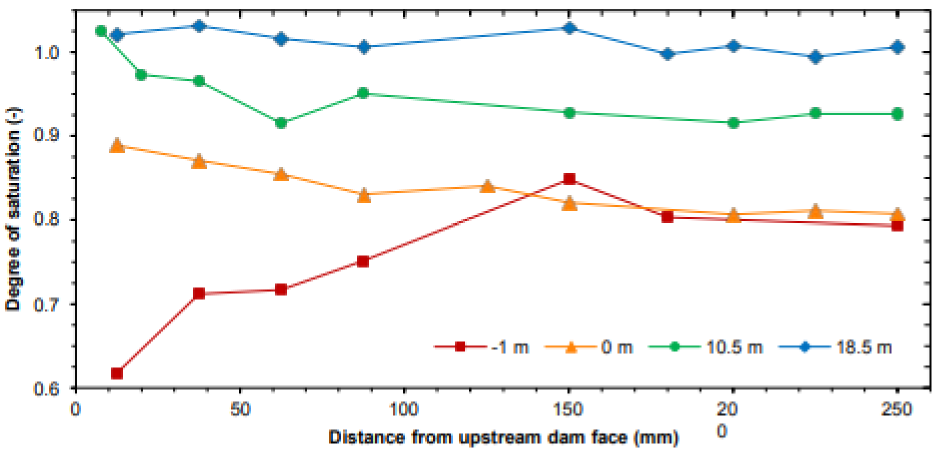


Fig. 2.1 – Water saturation levels in the concrete from the upstream face of a concrete dam [2.30]

2.6 Comparisons

From the above discussion, it is clear that by far the most common form of expansive process affecting dams is Alkali-Aggregate Reaction, and in particular Alkali-Silica Reaction. Apart from this, a much smaller number of cases may be attributed to ironsulphide containing rocks (Internal Sulphate

Attack). Heat induced Delayed Ettringite Formation should not occur if adequate precautions have been taken to avoid thermal cracking although there are a limited number of cases where DEF is occurring with considerable effects. External Sulphate Attack will not give large scale expansions and is not a realistic possibility in dams other than perhaps foundations and appurtenant structures. Concrete affected by Freezing and Thawing in dams is mainly in the water line zone of the upstream face and in some appurtenant works such as the spillways. Water absorption as cause of massive expansive phenomena in concrete dams has never been ascertained and it is thought not to be significant.

2.7 Synergistic effects

Concrete dams as other concrete structures are often affected simultaneously by a wide range of deterioration phenomena and interactions between two or more forms of damage are likely to accelerate degradation, producing an effect greater than the sum of their individual effects (synergistic effect) and further reducing their service life.

For example, there is evidence that other mechanisms that produce expansive forces in concrete, such as freezing and thawing, Internal Sulphate Attack (ISA) and Delayed Ettringite Formation (DEF), can have a synergistic effect with Alkali-Aggregate Reactions; each making the other worse. Avoidance of these effects, important in itself, should therefore be seen as an integral part of avoiding long-term alkali-aggregate damage in very long-service structures like dams. Thus, strenuous endeavours to avoid AAR, whilst obviously laudable, should not cause all of the other potential threats to concrete durability to be overlooked or considered to be of secondary importance.

The issue of determining the primary cause of damage to concrete elements in the presence of a combination of Alkali-Aggregate Reaction and DEF is very controversial: in fact, although the microscopic causes are very different, the macroscopic effects appear very similar. Moreover, it exists mechanical couplings (the degradation due to a reaction weakens the material and promotes the other one) as well as chemical couplings between the reactions leading to a possible AAR initiator effect on DEF or vice versa.

According to some authors ASR, the considerably more widespread and significant type of AAR, is to be considered as the primary cause of cracking and deterioration, and the subsequent precipitation of ettringite into the cracks is just a secondary damaging effect [2.31]. Other researchers have suggested that the chemical changes resulting from ASR, by reducing the pore solution alkalinity, may promote the formation of ettringite, expediting the release of sulphates entrapped by the hydrates during elevated-temperature curing.

In contrast, it has been postulated that the formation of ettringite leads to increased pore solution alkalinity, thereby enhancing the chances of ASR [2.32]. In fact the reaction of sulphate with calcium aluminates consumes calcium from calcium hydroxide, thereby releasing more hydroxyl ions to the pore solution [2.33].

For submerged and partially submerged exposure, in the outer 10 cm of the concrete upstream face, water leaching is able to reduce the alkali content, thus locally reducing the risk of ASR and creating differential expansion zones [2.34].

For its part, ASR can reduce the resistance of concrete to cyclic freezing and thawing, even if the concrete is adequately air-entrained. If cracks induced by ASR become saturated the freezing water will propagate and widen the cracks.

AAR may not be the main cause of concrete deterioration in many concrete structures but it may promote other forms of deteriorations such as freezing-thawing and sulphate attack, through opening the required pathways for aggressive substances to enter concrete and causes deterioration.

2.8 References

[2.1] Katayama, T., “The so-called alkali-carbonate reaction (ACR) — Its mineralogical and geochemical details, with special reference to ASR”, Cement and Concrete Research, Vol. 40, Issue 4, pp. 643-675, 2010.

[2.2] Leemans, A., Katayama, T., Fernandes, I., Broekmans,M. “Types of alkali–aggregate reactions and the products formed”, Construction Materials,Vol. 169, Issue CM3, 2016.

[2.3] Sims, I., Poole, A.B., “Poole Alkali-Aggregate Reaction in Concrete: A World Review”, CRC Press, 2017.

[2.4] RILEM TC 219-ACS Recommended Guidance AAR-1.2, for Use with the RILEM AAR-1.1, Petrographic Atlas: Characterization of Aggregates Regarding Potential Reactivity to Alkalis, Springer, 2016.

[2.5] RILEM State-of-the-art Report, “Guide to diagnosis and appraisal of AAR damage to concrete in structures – Part 1 Diagnosis (AAR 6.1), Springer, Godard, B., de Rooij, M., Wood, J.G.M. Eds., vol. 12, p. 87, 2013.

[2.6] Stark, D., "The moisture condition of field concrete exhibiting alkali-silica reactivity", Proc. Second International Conference on Durability of Concrete (ACI SP-126), Montreal, Canada, Vol. 2, pp 973-987 1991.

[2.7] Charlwood, R.; Scrivener, K., “Expanding Concrete in Dams: Long-Term Challenges”, Proc. International Symposium on Dams and Reservoirs under Changing Challenges, 79th ICOLD Annual Meeting, Lucerne, Switzerland, pp. 179–186, 1 June 2011.

[2.8] RILEM, “Recommended Specification: AAR-7.3 Preliminary international specification to minimize damage from Alkali-Aggregate Reactions in concrete – Part 3: Concrete dams and other hydro structures”, in RILEM Recommendations for the prevention of damage by Alkali-Aggregate Reactions in new concrete structures, RILEM State-of-the-art Report, Springer, Nixon, P.J. and Sims, I. Eds., vol. 17, pp. 155-168, 2016.

[2.9] Thomas, M.D.A., Fournier, B., Folliard, K., "Alkali-Aggregate Reactivity (AAR) Facts Book", U. S. Federal Highway Administration Report n° FHWA-HIF-13-019, 2013.

[2.10] Hobbs, D.W., "Alkali-Silica Reaction in concrete", Thomas Telford Ltd, 1988.

[2.11] Brueckner, R., Ndugga, N., Meri, T.C., “Effects of alkali-silica reaction on a hydropower structure after 50 years of ongoing deterioration”, MATEC Web of Conferences 199, 03008, ICCRRR 2018.

[2.12] Larive, C., Joly, M., and Coussy, O.,“Heterogeneity and anisotropy in ASR-affected concrete - Consequences for structural assessment”, Proc. 11th International Conference on AAR in concrete, Quebec, Canada, pp. 919-928, 2000.

[2.13] Latifee, E. R, Kabir, M.R., “Predicting ASR long term expansion through existing test method”, Malaysian Journal of Civil Engineering Vol. 27, n. 2, pp.180-192, 2015.

[2.14] Bérubé, M.A., Duchense, J., Dorion, J.F., Rivest, M., “Laboratory assessment of alkali contribution by aggregates to concrete and application to concrete structures affected by alkali-silica reactivity”, Cement and Concrete Research, Vol. 32, pp 1215-1227, 2002.

[2.15] Constaniner, D., Diamond, S., Alkali release from feldspars into pore solutions, Cement and Concrete Research, Vol. 33, n. 4, pp 549-554, 2003.

[2.16] Berra, M., Mangialardi, T., Paolini A.E., “Alkali release from aggregates in long-service concrete strucures: laboratory test evaluation and ASR prediction”, Materials, 11, 2018.

[2.17] Thomas, M.D.A., "Field studies of fly ash concrete structures containing reactive aggregates." Magazine of Concrete Research, Vol. 48, n.177, pp 265-279, 1996.

[2.18] Thomas, M.D.A., Hooton, R.D., Rogers, C. and Fournier, B. 2012. “50 years and still going strong: fly ash puts paid to ASR.” Concrete International, January, pp. 35-40.

[2.19] Berra, M., Mangialardi, T., Paolini A.E., “Residual capability of alkali binding by hydrated pozzolanic cements in long-service concrete structures”, Advances in Materials Science and Engineering, 2018.

[2.20] Yu, C., Scrivener, K., “Mechanism of expansion of mortars immersed in sodium sulphate solution”, Cement and Concrete Research, Vol. 43, n. 5, pp 105-111, 2013.

[2.21] Baronio, G., Berra, M., “Concrete deterioration with the formation of Thaumasite”, Il Cemento, Vol. 83, n. 3, pp 169-184, July –September 1986.

[2.22] Mingyu, H., Fumei, L., Mingshu, T., “The thaumasite form of sulphate attack in concrete of Yongan Dam”, Cement and Concrete Research, Vol. 36, pp 2006-2008, 2006.

[2.23] Crammond, N., “The occurrence of thaumasite in modern construction – A review”, Cement and Concrete composites, Vol. 24, pp 393-402, 2002.

[2.24] Chongjiang, D., “A review of magnesium oxide in concrete - A serendipitous discovery leads to new concrete for dam”, Concrete International, pp 45-50, December 2005.

[2.25] Fernández Cánovas, M., “Entumecimiento higrométrico del hormigón”.Materiales de Construcción”, IETcc. Nº 383. pp.: 27-41, 1980.

[2.26] Eriksson, D., “Numerical models for degradation of concrete in hydraulic structures due to long-term contact with water”, Licentiate Thesis KTH Royal Institute of Technology Department of Civil and Architectural Engineering Division of Concrete Structures Stockholm, Sweden, 2018.

[2.27] Buil, J. M., Río, F., Aguado, A., Agulló, L., Araujo, G., López, C. M. y Agulló, L. “Mequinenza dam: revision of an earlier diagnosis”. 23th ICOLD Congress, Q.91-R52, Brasilia (Brasil), 25-29 May 2009.

[2.28] Buil, J. M., Río, F., Campos, A., Aguado, A., López, C. M., Chinchón, S. “Problemática y actuaciones recientes en la presa de Mequinenza”, Proc. IX Jornadas Espanolas de Presas, Comité Espanol de Grandes Presas, Valladolid (Spain), 25-27 March 2010.

[2.29] Sokolov, I.B., Rasskazchikov, V.A., “Consideration of moisture effects on the upstream face of concrete dams”, Hydrotechnical Construction, Vol. 20, n. 3, March 1986.

[2.30] Rosenqvist, M., “Frost-induced deterioration of concrete in hydraulic structures: interactions between water absorption, leaching and frost action”, Lund University, 2016.

[2.31] Thomas, M. D. A., “Delayed ettringite formation in concrete - Recent developments and future directions, University of Toronto, 1998.

[2.32] Pettifer, K,, Nixon, P.J., “Alkali metal sulphate — a factor common to both alkali aggregate reaction and sulphate attack on concrete”, Cement and Concrete Research, Vol. 10, pp.173-181, 1980.

[2.33] Bauer, S., Cornell, B., Figurski, D., Ley T., Miralles, J., Folliard, K., “Alkali-Silica Reaction and Delayed Ettringite Formation in concrete: a literature review”, Report No. FHWA/TX-06/0-4085-1, February 2006.

[2.34] Havvagul V., “Alkali – Silica Reactions in Concrete Dams: Alkali Leaching in the Votna Dam”, Department of Structural Engineering Faculty of Engineering Science and Technology, NTNU-Norwegian University of Science and Technology, Master Thesis, 2016.

3 BEHAVIOUR OF CONCRETE DAMS AND HYDROELECTIC PROJECTS AFFECTED BY EXPANSIVE REACTIONS

3.1 Development in time of the expansive reactions

3.1.1 *Alkali-Aggregate Reaction*

Some of the early cases of ASR that were reported in the USA (with aggregate types shown in brackets), such as Stewart Mountain (rhyolitic, andesitic volcanics), Gene Wash (andesite, rhyolite) and Copper Basin (andesite, rhyolite) dams showed rapid initial expansion and then a slowing and ending of expansion after about 25-30 years [3.1]. This limited period of expansion was interpreted to be due to the supply of alkalis being readily available from the cement and once this is used up, the reaction and related expansion ceases. However, as discussed in Chapter 2, when alkali silica gels come into contact with cement pastes, they will gradually react with calcium ions to give the normal Calcium Silicate Hydrate (C-S-H) binding phase and in so doing the alkalis will be released.

Moreover, there have been an increasing number of large dam cases of AAR, where SCMs were not used, and expansion is continuing after 40 years or more years, such as at Fontana (greywacke – USA), Hiwassee (greywacke – USA) and Roanoke Rapids (granitic-gneiss – USA), R.H. Saunders (limestone – Canada) and Mactaquac (greywacke – Canada), Chambon (gneiss – France), Pian Telessio (gneiss – Italy), Kariba (granitic gneiss – Zambia/Zimbabwe) and Cahora Bassa (granite, porphryoblastic gneiss – Mozambique) dams. There have also been some cases exhibiting features of both AAR and of ISA such as San Esteban (granite, diabase, gneiss and shale with some pyrite – Spain) and Isola and Ilsee (Switzerland). As explained in Chapter 2, the role of new alkali supply from concrete aggregates can play in the long-term behaviour of dams and clearly warrants serious attention in terms of the duration of remaining expansion, perhaps effectively indefinitely, and whether or not such a supply can overpower the preventative value of using SCMs to reduce the initial alkali loading.

For the purposes of illustration, the S-shaped expansion versus time curves, introduced in Chapter 2, is used herein to facilitate discussion of the AAR expansive process in concrete dams over time. Here, it is divided in three main stages, the initiation phase, the propagation phase and the stabilization phase as shown in Fig. 3.1.

In the initiation phase the behaviour of dams affected by extremely slow expansive phenomena is generally satisfactory since the reaction is stared but the swelling is at its initial state of progress and small cracks do typically develop in those dams [3.2]. Furthermore, the measured displacements of dams generally show an initial period of normal behaviour without signs of concrete swelling. The acquired experience shows that the first appearance of expansive behaviour (Initiation time) in concrete dams, can vary considerably from case to case, due to the different aggregate reactivities and different alkaline conditions inside the concrete pore solutions. The Fig. 3.2 shows the initiation times of several concrete dams around the word, taken from the literature [3.3] [3.4], passing from few years to over half a century.

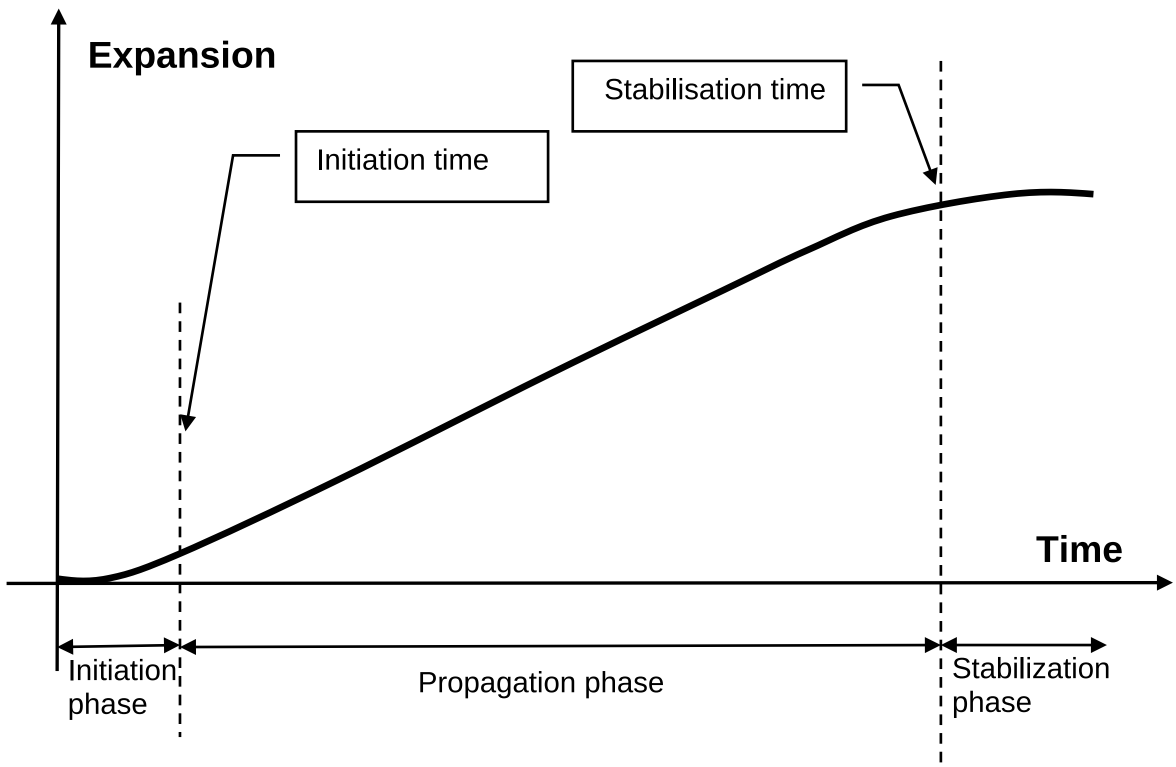


Fig. 3.1 - Definition of expansion phases in concrete dams affected by AAR

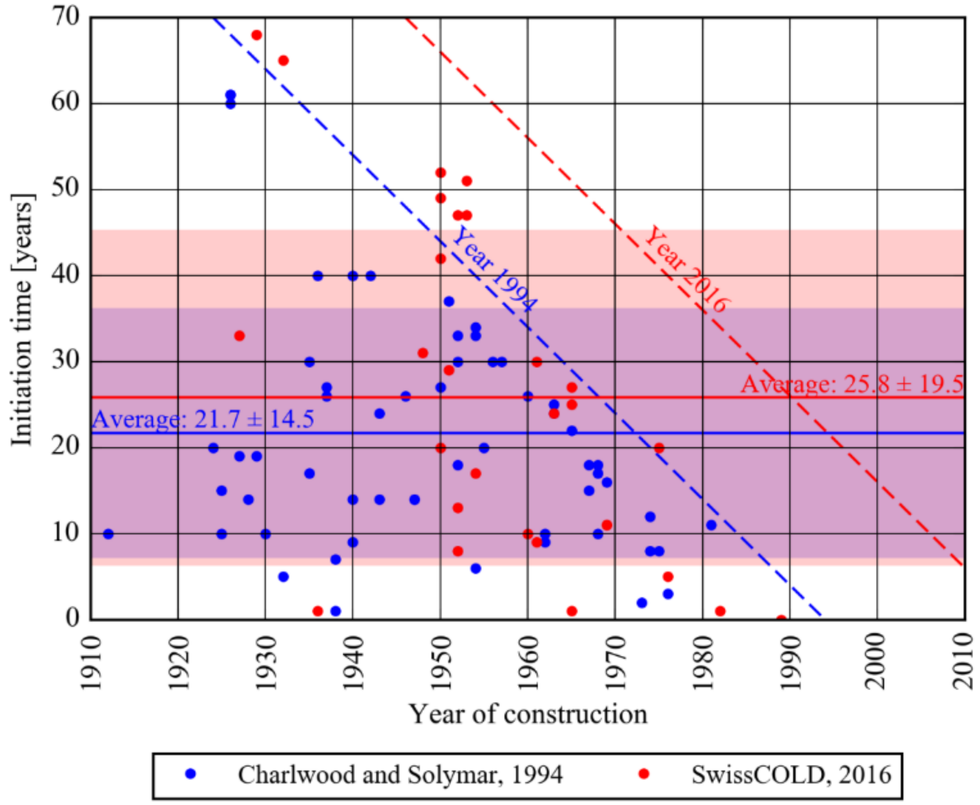


Fig. 3.2 - Initiation times of AAR in concrete dams [3.3] [3.4]

In this phase rising of dam height and horizontal permanent displacements (often upstream drift) are clearly identified, usually with greater expansion in the upper part of the dam. Important cracks may also occur. Some of them are induced directly by differential swelling while other cracks are induced indirectly by the structural response to the concrete expansion. Cracks produced directly by differential swelling are visible in particular in the inspection galleries [3.2]. Such type of cracks is present in various dams: e.g. Pian Telessio dam (Italy), Isola dam (Switzerland) [3.5] or Portodemouros dam [3.6]. These cracks are characterized by a relative continuity along the gallery, but they are often not visible or less visible on both dam faces. Structural cracks, produced indirectly by expansion, appear typically along structural discontinuities, as for example at the transition between a straight gravity and a curved part like at Illsee dam (Switzerland) [3.5], or along the foundation as peripheral cracks on the downstream face of arch dams, e.g. Serra dam (Switzerland) [3.7].

Even if the typical diffuse map cracking at the surface of structures affected by swelling may not be visible in many dams, the expansion can cause consistent micro-cracking in the aggregates and the cement paste, with a reduction of concrete quality (paragraph 3.3)

At some time in the future, the expansion should slow to a low rate and theoretically come to an end ("Stabilization phase"), as seems to be happened in about 30 years in the case of Gene Wash and Copper Basin dams (Fig. 3.3). The total expansion in this period of time has been evaluated, based on vertical movements, between 2000 and 2500 με (0.2-0.25%). Petrographic examination of cores in the later years confirmed that AAR came to a conclusion in these dams. Reaction products have solidity filled the micro-cracks caused by the earlier expansive forces [3.8]. Such cases are however uncommon and the reason of the stabilization not completely known. Apparently the reactive elements have been consumed. The influence of progressive restraint effects is also sometimes recalled but no experimental evidence has been really produced [3.9].

For many cases of large dam structures, as documented above, the propagation phase has been shown to last for very long time, with no sign of abatement, suggesting that for practical purposes the expansion should be assumed to continue at a constant rate for the foreseeable future. In Fig. 3.4 [3.10], Fig. 3.5 [3.11], Fig. 3.6 [3.12] and Fig. 3.7 [3.13] some schematic examples of undiminished expansion rate in the propagation phase are shown, in USA and Europe.

However, in the stabilization phase, absorption of water by the gel will continue as long as residual water remains from the hydration process or is penetrating into the structure. A condition of fully saturated condition in the concrete may require several decades (paragraph 2.4). In this way a very slowly progressive expansion could be considered as continuing indefinitely, even if most often the consequences are aesthetic and not functional or structural.

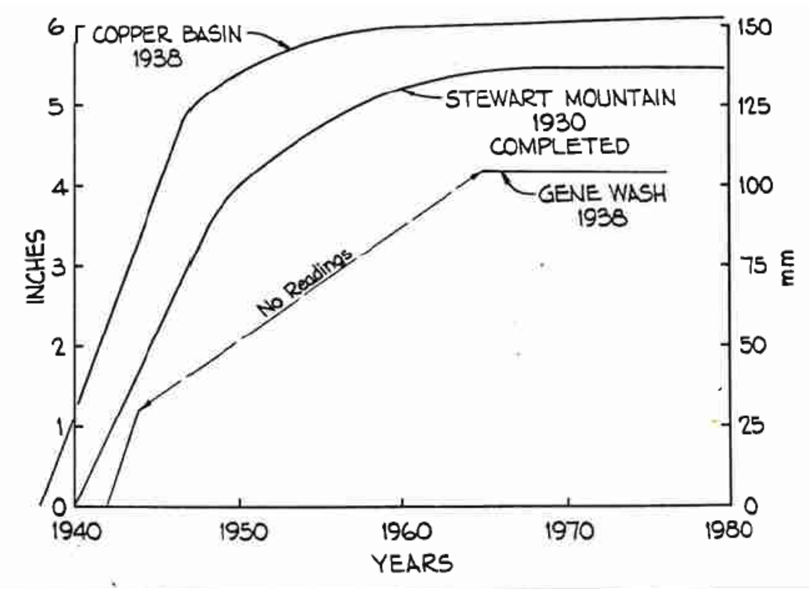


Fig. 3.3 – Upstream movements of crest of three arch dams that appear to have stabilized (USA) [3.8]

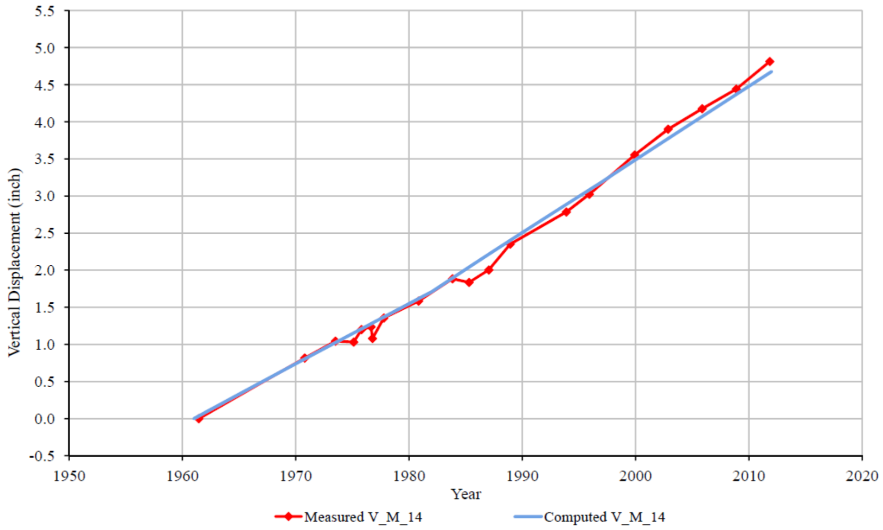


Fig. 3.4 – Example of undiminished expansion rates: measured and computed displacements at block 14, near centre of Fontana Dam (USA) [3.10]

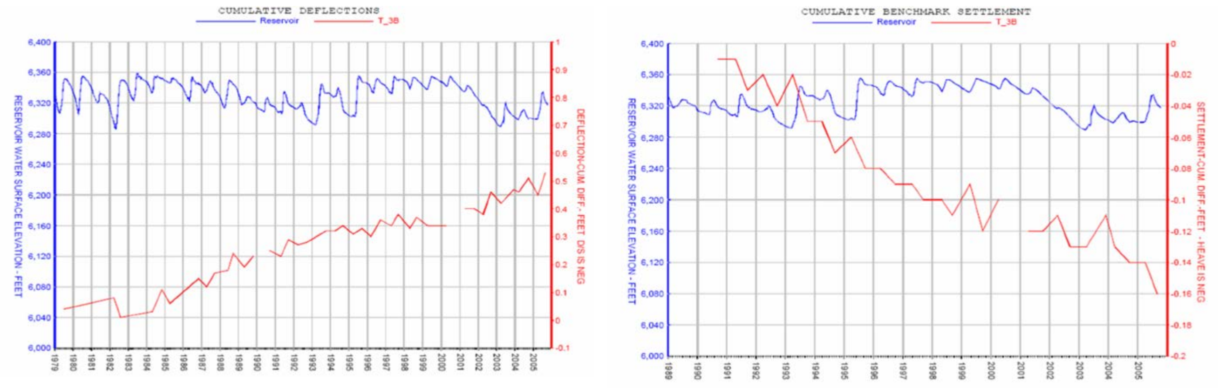


Fig. 3.5 – Example of undiminished expansion rates: movement upstream and vertical expansion at Seminoe dam (USA [3.11]

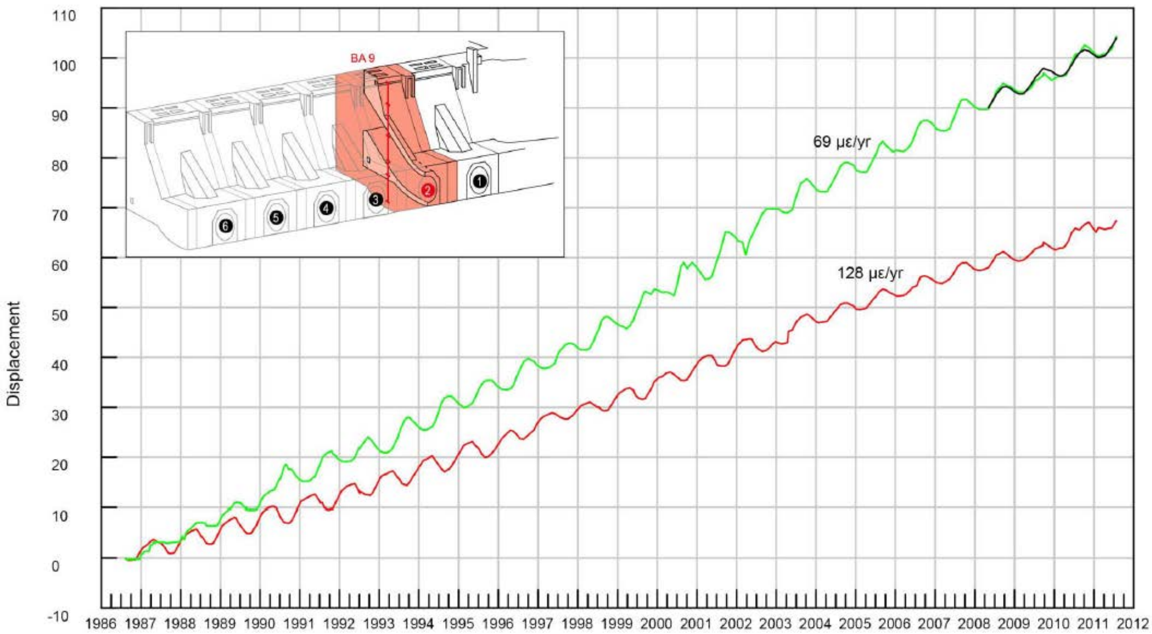


Fig. 3.6 – Example of undiminished expansion rates: Measurements from Borehole Extensometer BA-09 at Unit 2 at Mactaquac Generating Station (Canada) [3.12]

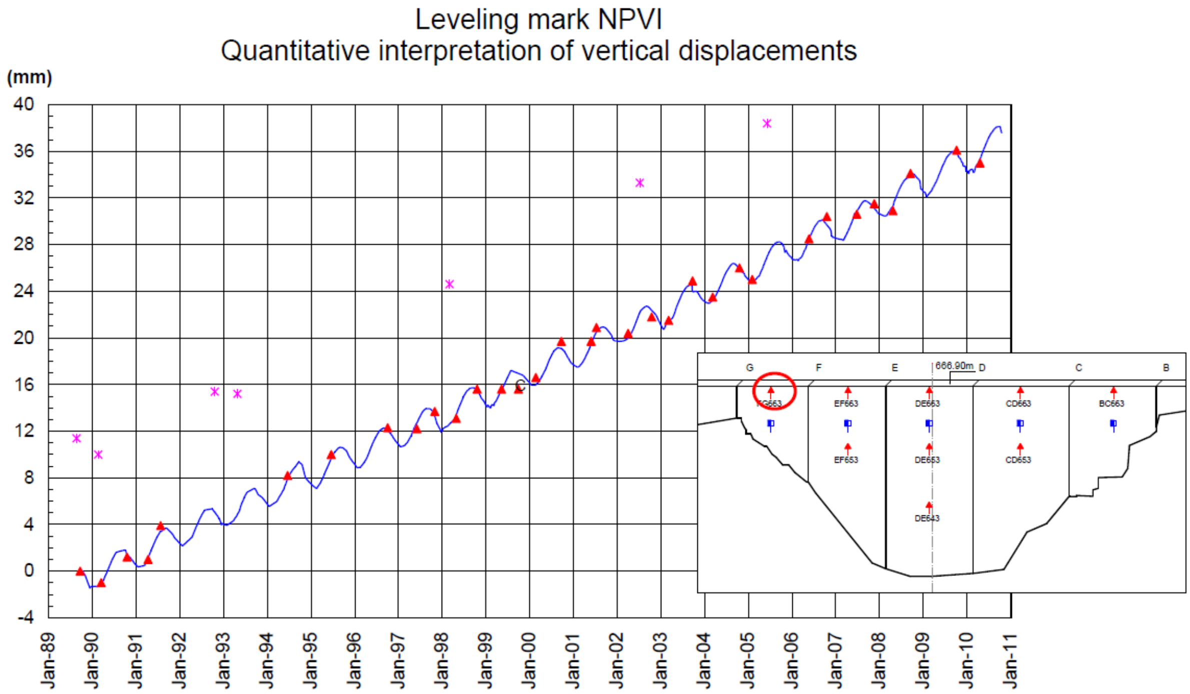


Fig. 3.7 – Example of undiminished expansion rates: vertical displacements at Alto Ceira dam (Portugal) [3.13]

3.1.2 *Internal Sulphate Attack (ISA) and Delayed Ettringite Formation (DEF)*

Both forms of internal sulphate attacks presented in Chapter 2 have been found to cause expansion in concrete dams, The documented cases concern different countries as, for example Portugal (Piracana, Cabril, Caia, Fratel and Fagilde dam) [3.14], Spain (San Esteban, Torán, Graus and Tavascán dam) [3.15] [3.16] [3.17], Brasil (Rio Descoberto dam) [3.18], Switzerland (Isola and Ilsee dams) [3.5] and France (Bimont dam) [3.19]. In some of these dams sulphate attack and AAR are found together, as for example Fagilde and Isola dams.

As with the AAR, so with the internal sulphate attack the expansion curve covers three distinct phases and could be considered to have a S-shape. In the first phase the expansion begins slowly at roughly constant rates while in the second one it accelerates and finally, the expansion may become in some way stabilized. When all the potentially reactive minerals are consumed, the oxidation reactions and the expansion tend to stop [3.14]. An example is shown for the Tavascan dam (Fig. 3.8). Also the formation of ettringite can slow and stop, as it happened in the case of Bimont dam. In this last case some concrete blocks on the right shoulder were particularly affected by DEF, those in which the thermal heat generated from cement hydration was not sufficiently dissipated during construction, causing concrete temperature increase, in the summer season, up to 80°C [3.19]. The high humidity environment conditions also promoted the expansive ettringite formation for about 30 years (Fig. 3.9 and Fig. 3.10). Cracks on the involved blocks occurred in the sixties of last century.

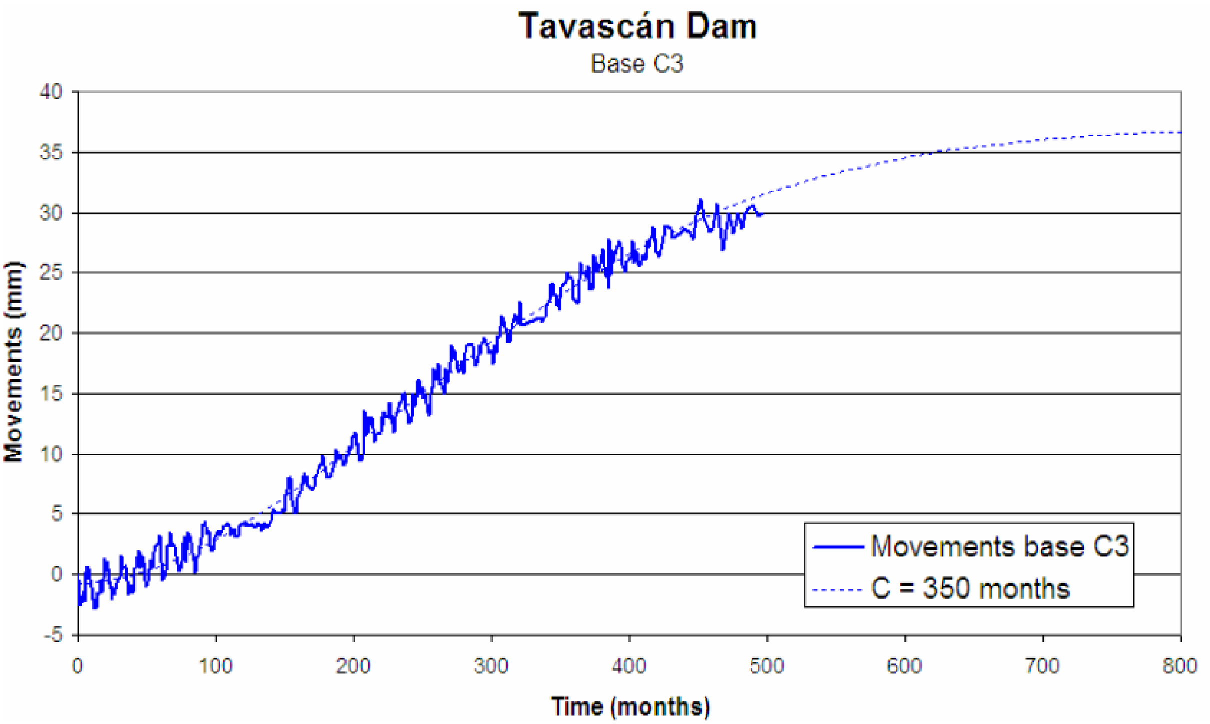


Fig. 3.8 – Movements at Tavascan dam, due to iron sulphide in the concrete aggregates: measurements (solid lines) and calculated curves at 350 months (dashed lines) [3.16] [3.17]

Fig. 3.9 – Bimont dam affected by DEF [3.19]

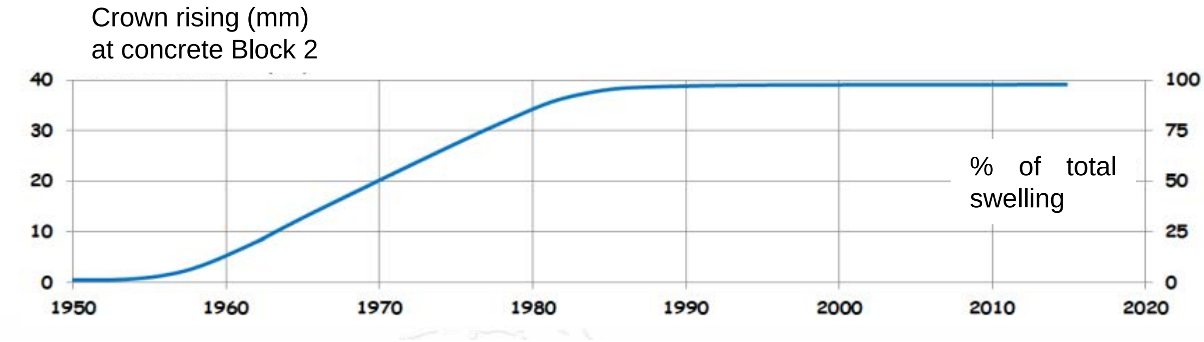


Fig. 3.10 – Expansion time developing at Bimont dam due to DEF: [3.19]

3.2 Spatial distribution of expansion

The distribution of the AAR expansion within a dam, over the dam height as well as across the wall thickness, is usually not necessarily uniform due to anisotropic expansion behaviour of concrete, to the compressive stress or restraint conditions and to other several factors that affect the expansion rate. The main are: temperature, alkalinity of the concrete pore solution (related to the aggregate mineralogy) and humidity conditions. These aspects will be discussed in the following paragraphs. Some differences between AAR and DEF are pointed out. Generally speaking, AAR can occur in the whole dam structure, due to the use of a reactive aggregate, while this is usually not the case for DEF, where the required temperature increase is generally restricted to the core massive part or to particular areas [3.20].

3.2.1 *Anisotropic Expansion Behaviour*

It is known that in a sound concrete specimen the tensile strength along the direction parallel to the casting direction (usually vertical) is lower than in the perpendicular direction (usually horizontal). This is because of distribution of flat and elongated aggregates, possible segregation phenomena in concrete and also possible discontinuities in planes perpendicularly to casting direction that, basically, represents the weakest direction. This intrinsic concrete anisotropic behaviour is found also in the swelling process of concrete affected by AAR. In fact, it was observed that, in free expansion conditions, a concrete specimen swells in the direction parallel to the casting direction more than in the perpendicular directions (from 1.3 to 2.8 times) [3.21].

The chemical reaction and formation of expansive gels can be considered an isotropic phenomenon before micro-cracking occurs. Afterwards the gel expansion will induce the propagation of the cracks preferentially in the weakest direction (perpendicular to the casting direction), which will influence the further swelling which will tend to be anisotropic.

If expansions in restrained conditions are taken into consideration, again the gel expansion induces the crack propagation preferentially in the direction with lower restraint, i.e. in the direction which requires less energy dissipation. When the compressive stresses are applied in the lateral direction the gel tends to expand along the longitudinal one. This anisotropic behaviour induced by the stress state is more relevant than the intrinsic one, particularly with the behaviour of AAR-affected concrete dams, where the concrete is generally constrained and often in biaxial or triaxial conditions. Concrete in AAR-affected dams is then considerably influenced by the coupling between chemical and mechanical loading and restraint (Chapter 6).

Experience gained at Mactaquac Generating Station (Canada), severely suffering from AAR, provided clear evidence, particularly during slot cutting in the intake structure, of the large impact of the stress state on concrete expansion, and of the anisotropic and not volumetric concrete behaviour. Due to the slot cutting, the stress conditions inside the structure changed considerably and the concrete expansion response considerably changed in the two directions, in close connection with the compressive stress changes [3.22].

Another source of anisotropy in the expansion behaviour of a dam is the possible physical variability of concrete inside the structure. For example in the construction of the intake dam and powerhouse of Fig. 3.11, the type of cement changed during construction, passing from a low heat Portland cement (mean alkali content of about 0.4% - 0.7% Na_2O eq) to a local high alkali cement (2.5% Na_2O eq), causing an increase of the alkali content of concrete up to about 7.5 Na_2O eq/m^3. This was considerably higher than the threshold alkali level of the aggregate used, producing considerable expansion of the concrete elements made with this specific local cement. On the contrary, concrete with low alkali cement was considered as not susceptible to AAR, provided that alkalis are not released from the aggregate in excess of the AAR threshold [3.23].

Using different types of aggregates or different concrete composition in different parts of the dam is also source of anisotropy. For example concretes composition for foundations has often different cement content compared to concrete used in the dam elevation parts, as well as facing concrete is different compared to concrete used for the dam main body.

Even if the ISA/DEF mechanism of reaction is different, compared to AAR, it is considered to induce isotropic expansion [3.20], at least at the beginning, before micro-cracking occurs. After that anisotropy arises for different reasons, similar to those discusses for AAR. In particular, in the case of ISA due to the oxidation of iron sulphides, anisotropy of expansion may be related not only to the spatial distribution of potentially expansive minerals but also to the availability of oxidizing agents [3.17].

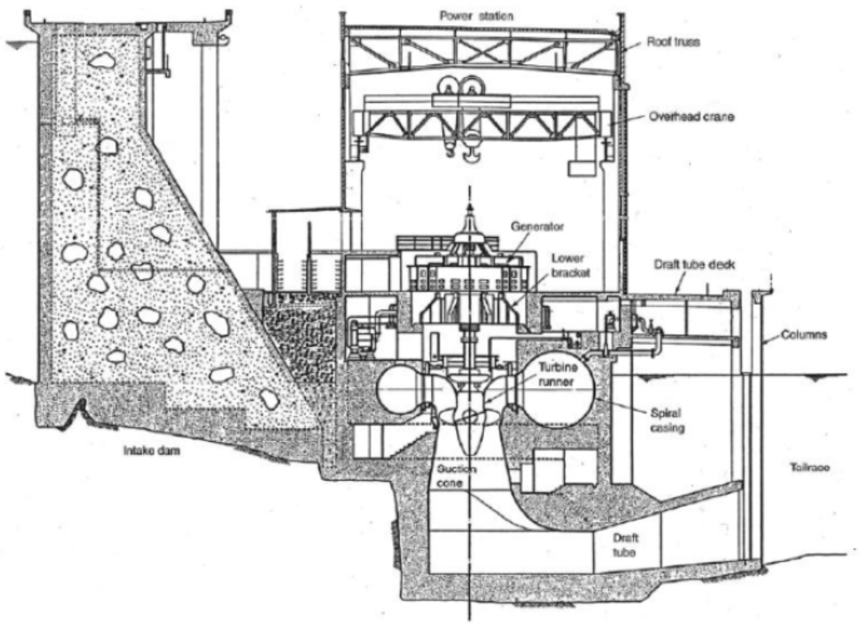


Fig. 3.11 – Concrete gravity intake dam and power house suffering from AAR, where cements with different alkali contents were used during construction [3.23]

3.2.2 *Compressive stress*

One AAR related effect is that the expansion is stress dependent. As the compressive stresses increase, the expansion decreases. At a critical amount of compression, the expansion will cease, and no further build-up of stress will occur. This effect is well documented in [3.24] [3.25] for Mactaquac power plant concrete structures (USA), where the critical stress level has been estimated from back calculation to be 5.5 to 8.3 MPa. This self-limiting effect has been very beneficial at Mactaquac as the build-up in stresses due to AAR and the limited compressive stresses in the massive nature of the structures, has not led to compression failures in the structures. In the converse, concrete in tension will tend to enhance growth and this is believed to have reduced the extent of tensile cracking within the structures.

The unrestrained expansion rate at Mactaquac has not been specifically measured, but deduced from instrumentation and back calculation to be in the order of 150 με/year (strain x 10^{-6}/year) that is 0.15 mm/m/year or 0.015%/year. Actual expansion within the structures is generally less due to the stress-dependent nature of the reaction and can vary from 0 to 145 με /year although, at some isolated locations, rates as high as 200 με/year have been measured. This means that the concrete at Mactaquac has experienced a total expansion of 2000 to 4000 με, a significant amount for structures of this size. For example, the deck of the intake has risen more than 10 cm since initial construction.

In gravity dams, the mitigating effect of compressive stress on the expansion can clearly be observed. In a straight gravity structure like Portodemouros spillway, for example, with a length of 130 m and a height of 10 m, the expansion rate in vertical direction is around 150 με/year while in horizontal direction it measures only 10 με/year. The horizontal expansion is clearly hindered by the structure and therefore an increase of compressive stress is expected. If the horizontal expansion were the same as the vertical one, the horizontal compressive stress should increase yearly by some 4 MPa (with a typical Young's modulus of 30 GPa). The compressive strength should be reached after around 10 years. In 1983 in-situ stress measurements have been performed by means of over-coring testing method. The results indicated compressive stress up to 5-6 MPa, thus clearly lower than the potential one. In arch dams it is generally more difficult to give evidence for the effect of compressive stress, since the thin and curved structure is nearly free to expand upstream, in particular in periods of reduced reservoir level [3.2]. For high arch dams like Cahora Bassa dam (170 m high in Mozambique), the behaviour analysis of the dam over time carried out showed changes in the expansion strain-rate due to an increase in compressive stresses with height above the foundation [3.26]. Since the chemical expansion is a slow load condition, creep is included in the structural behaviour, where it plays an important role. In fact, similar results can be obtained using a model with stress dependency and a model without, but with creep [3.2].

Compressive stresses decrease also DEF expansion in the direction subjected to restraint and lead to cracks parallel to the restraint. Thus restraint causes a decrease of the volumetric expansion and DEF expansion under restraint is anisotropic. The differences of laboratory expansion between stress-free and restrained directions have been quantified by a coefficient of anisotropy of 0.6 and 0.45 for compressive stresses of 1.6 and 4.2 MPa respectively [3.27].

3.2.3 *Temperature*

The reaction rate, as for any other chemical reaction, is influenced by the temperature: the higher is the temperature the higher is the reaction rate. The relationship of reaction rate with temperature has been shown to follow the Arrhenius relationship (Rate $=R_{ref} * \exp(-E_a/RT)$), as further detailed in Chapter 6. In this relationship, usually considered valid for temperatures up to around 38-50°C, E_a is the activation energy, i.e. the minimum energy required for an AAR reaction to proceed. For mass concrete, values of about 40-50 kJ/mol are typically reported for this parameter [3.28]. As a very rough approximation, it may be assumed that the reaction rate doubles every 10°C increase in temperature and consequently a temperature rise from 10 to 40°C can increase the AAR rate 10 fold. This is also a reason for occasional very severe AAR damage in tropical area dams [3.2].

However, high temperatures (60°C and above), as used in some laboratory tests, may lead to additional effects. Indeed, while at ambient temperature the alkalis ions (Na^+ and K^+) in solution are effectively balanced by OH^- ions, so that there is a direct relation between alkali content and pH, when the temperature is raised above 50-60°C, there is an increase of concentration of sulphate ions, which partially balance the alkali ions, leading to a decrease of pH for the same alkali content. This could explain why lower expansions may be seen in high temperature testing (although the rapid leaching of alkalis at high temperature is also a factor) [3.28].

For expanding concrete dams the temperature represents a key issue as its distribution and annual variation can considerably affect the spatial distribution of expansion. Warm and slender parts in dams swells significantly more than the colder and more massive ones. This is clearly evident in the example of arch dam of Fig. 3.12 [3.4], even if the compressive state effects is also simultaneously present, to strengthen the temperature effect

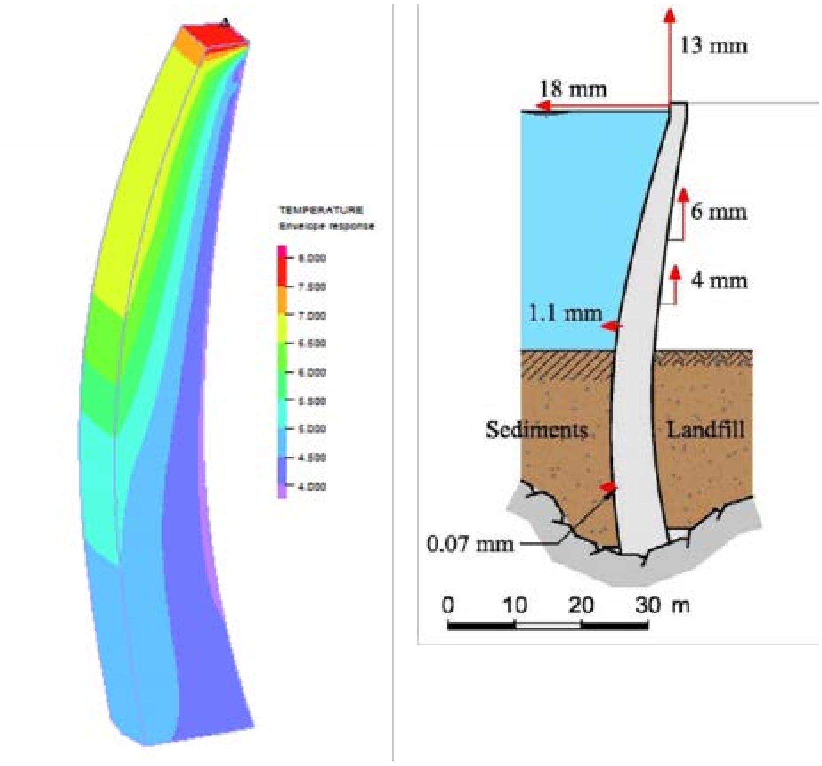


Fig. 3.12 – Temperature distribution and measured displacements in an arch dam [3.4]

According to the experience gained in managing expansive dams in Switzerland, the distribution of structural cracks also suggests that the sunlit, near-surface dam areas react much faster than those in shaded areas or within the dam structure. Concrete temperature on sunlit areas are between 4 to 6 °C warmer than the ambient temperature and can produce internal restraining stresses, schematically shown in two real examples of gravity-arch dams in Switzerland (Fig. 3.13) [3.4]. For those dam structures with the downstream face exposed to the sun, structural cracks were found within the dam (none on the downstream face), while structures with their upstream face oriented towards the sun revealed horizontal structural cracks on the downstream face [3.2].

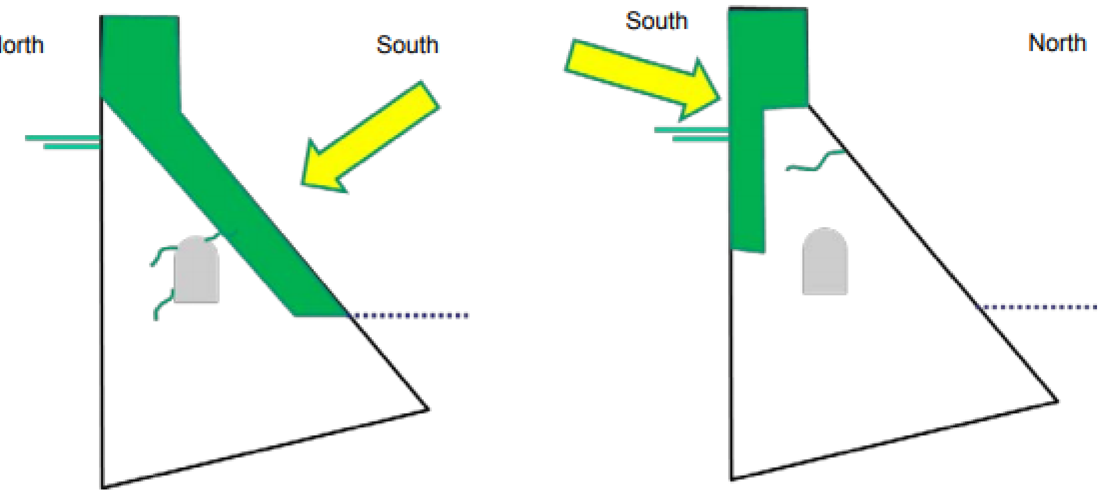


Fig. 3.13 – Sun-facing concrete areas in gravity-arch dams differently facing the sun [3.4]

The presence of internal cracks (Fig. 3.13), observed in the inspection galleries but which were not found at the faces, clearly indicate a greater expansion near the surface than in the internal part of a dam. The most probable reason, apart the confirmation that moisture cannot be relevant, is that the reaction at the faces is accelerated by the higher temperatures. Many dams located in the south facing slopes are strongly subjected to solar radiation. The average yearly temperature downstream can be, as said, 4-6 °C higher than at the upstream face and the maximum temperature in summer might be even more than 10°C higher, due to impounding [3.2].

The orientation of the dam is of particular importance also for expansion phenomena due to the oxidation of iron sulphides. If the downstream face of the dam, where more intense oxidation can occur, is exposed to the south and then exposed to higher temperatures, the expansive reaction process is accelerated (paragraph 3.2.5 on moisture).

As for the expansion phenomena due to DEF, the precipitation of ettringite is strongly influenced by the temperature reached during the hydration in the dam construction phase and its duration of application. Laboratory work has shown that temperature exceeding 60 - 80 °C, in the presence of other key parameters (Chapter 2), generally results in the DEF formation. Concerning the temperature during the concrete service life, a heating implies an increase of ettringite solubility, decreasing the expansion [3.29]. However, a temperature increase implies an increase of the kinetics which can be explained by enhanced ionic exchanges at higher temperatures [3.30].

3.2.4 *Alkali content of concrete and aggregate reactivity*

Under established exposure conditions, the severity of concrete degradation due to AAR is strongly related to the alkali-reactivity of the aggregates used for the concrete structure and the hydroxide ions of the concrete pore solution (high pH), being, at ambient temperature, almost completely balanced by sodium (Na^+) and potassium (k^+) alkali metals. These last are usually quantified though the alkali oxides (Na_2O and K_2O) coming from of all the concrete components: cements and SCMs but also water, admixtures and, in the long term evaluations, also from alkali releasable from some rock minerals (e.g. feldspars) in the aggregates.

In particular, the available alkali content in the concrete pore solution can be calculated by considering a 100 % alkali release from Portland cement, water and admixtures while, for the SCMs the available alkali could be determined by suitable methods such as ASTM C311 [3.31] [3.32]. Different laboratory test methods have also been proposed in the literature for estimating the long-term alkali contribution by aggregates to concrete structures [3.33] [3.34]. The total available alkali content of the concrete is expressed as kg of Na_2O equivalent (Na_2O + 0.658 K_2O) per cubic meter of concrete (kg Na_2Oeq/m^3).

Much early work focused on the existence of an “alkali threshold” defined as the maximum level of available alkali content in concrete that does not produce deleterious AAR expansion. This was initially estimated to be in the range 3-4 kg Na_2Oeq/m^3 and considered as a general safety limit. However, the Threshold Alkali Level (TAL) required to initiate damaging expansion in the concrete varies considerably between aggregates and it has been therefore used as reactivity parameter for assessing the alkali-reactivity of concrete aggregates. This is, for example, a proposed classification [3.35]:

TAL > 7.4 kg Na_2Oeq/m^3 = non reactive aggregate.

5.5 < TAL ≤ 7.4 kg Na_2Oeq/m^3 = slowly reactive aggregate

2.8 < TAL ≤ 5.5 kg Na_2Oeq/m^3 = moderately reactive aggregate

TAL ≤ 2.8 kgNa_2Oeq/m^3 = quickly (highly) reactive aggregate

The more the aggregate is reactive, the lower is the TAL. Broadly speaking the reactivity of a mineral increases as its crystal structure becomes more disorganize allowing easier penetration of the alkaline pore solution into the mineral structure [3.28].

The Threshold Alkali Level (TAL) of aggregates can be evaluated by means of short term laboratory tests, in particular the RILEM AAR-3 method (expansion test in concrete at 38 _C and 100 % RH after 1 year) [3.33] [3.35]. However longer-term tests, involving exposure of large blocks and field studies indicate that this “threshold” is lower than values derived from short term laboratory testing [3.28]. This could be due to the presence of slowly reactive aggregate as well as to the fact that alkali leaching has a great influence on the small slim specimens used in the laboratory short term tests while has a negligible effect on larger concrete blocks and in the field test. Tests of long enough duration, possibly avoiding loss of alkalis through leaching, are therefore recommended.

If the Level of the available alkali content of a concrete remains lower than the Threshold Alkali Level of the reactive aggregate used, during all the dam service life, deleterious expansion does not develop in the concrete. However, as above discussed, enhancement of the alkali concentration in concrete may arise during the service life of concrete dam, when the aggregate with alkali-bearing release their alkalis into the concrete pore solution, over a long time [3.34]. This fact should have been taken into account in the design stage. Otherwise concrete may transform itself into an expansive concrete, with a very long initiation phase.

The questions of alkali content is further complicated by the binding of alkali in the ASR gel and also in the cement hydrates and by the “recycling” of alkalis as ASR gel transforms to C-S-H when it comes into contact with the paste, so releasing alkalis.

As for DEF is concerned, the presence of alkalis strongly decreases the stability of ettringite, especially when temperature increases and can have different effects on the sulphate reaction process, depending on whether the concrete is taken into consideration at the fresh or at the hardened state. The presence of alkalis at early age is able to enhance future DEF growth. In fact, in the pore solution of fresh concrete, during the heat development necessary for DEF, the alkalis will tend to lower the formation of non-deleterious primary ettringite and thus promote the release of sulphates either in the pore solution or adsorbed by the cement CSH. These sulphates will then provide reactants to form subsequent deleterious secondary ettringite in the hardened material, after cooling. The presence of alkalis in hardened concrete, on the contrary will tend to hinder or even avoid the DEF to occur, since the desorption of the sulphates bound with the CSH is slowed down. [3.20].

3.2.5 *Moisture*

The high internal humidity in the concrete dam, usually close to 100%, can supply the necessary moisture to sustain numerous deterioration mechanisms, AAR and ISR/DEF included. A confirmation of the high moisture conditions in concrete dams is the case of the Illsee gravity dam in Switzerland, where an impervious membrane was installed on the upstream dam face in order to reduce the moisture within the dam and so to prevent the further expansion. Until 10 years after rehabilitation no slowdown of the drift could be observed [3.5].

However, towards the downstream faces, a condition of equilibrium with the environment is reached, so that a layer thickness of about 1 m of concrete is only partly saturated [3.36], as shown in Fig. 3.14 [3.37]. Even if these humidity conditions are usually reached after several years, some factors can speed up the process such as the presence of cracks, construction joints, drainage systems etc. . Nevertheless, in the downstream dam faces exposed to air, in the crest, and in the uppermost part of the upstream face AAR is favoured by wetting/drying cycles, humidity gradients, freezing/thawing cycles that may cause migration and concentration of alkalis in concrete [3.38].

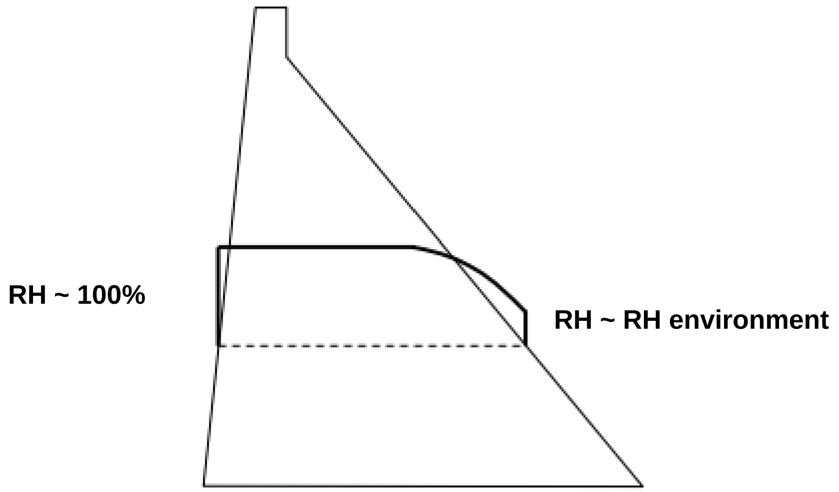


Fig. 3.14 – Humidity distribution within a dam body [3.37]

Water is a reactive medium essential to producing the ettringite formation, being involved both in the transfer process as in the actual formation of reaction products. DEF primarily affects the parts of structures either in contact with water (submerged zone, tidal zone) or subjected to water ingress (exposure to bad weather, waterproofing defects, absence of drainage, etc.), or perhaps exposed to a high moisture level.

While for AAR development the humidity threshold is generally considered to be about 80-85 % of Relative Humidity (RH) [3.20], for DEF higher values are reported and 90-92% or 93-95%, in any case generally lower than the humidity conditions inside the concrete dams (close to 100%).

As for the ISA expansion phenomena due to the oxidation of iron sulphides, in the upstream face of the dam, which is usually submerged for long periods of time, the availability of oxygen is much lower than in downstream face and, therefore, the ISA reaction is here slowed down or considerably reduced in its effects. On the contrary, the reaction is accelerating on the downstream face where oxygen is easily available and, at the same time, ambient humidity and rainwater are sufficient source of moisture to start and drive oxidation. This acceleration is further enhanced if the downstream face is oriented south and can then be affected by higher temperatures. Fig. 3.15 shows a scheme of the non-linear distribution of expansion of a typical concrete gravity dam section. The inwards expansions from the downstream face tend to decrease and stabilize with time as the availability of oxygen decreases from the most exposed areas of the dam towards the interior [3.17].

As the downstream face is exposed to more favourable conditions for developing ISA expansions, compared to the upstream face, gravity dams are expected to move upstream.

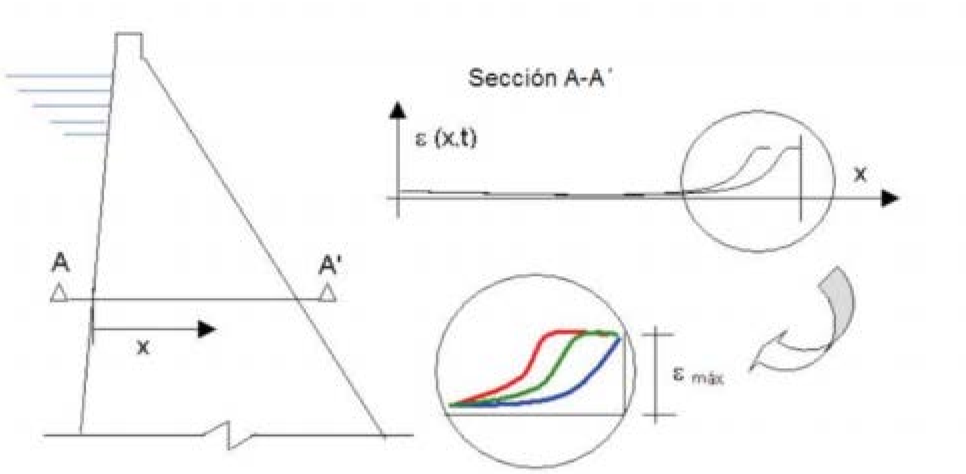


Fig. 3.15 – Non linear distribution of ISA expansions in a representative section of a dam body [3.17]

3.3 Effects of expansion on concrete physical properties

3.3.1 *Overview*

The measurable consequence of the micro-cracking forming inside the concrete, as a consequence of expansive reactions, is a reduction of concrete quality, in particular strength and stiffness. The extent of such reduction essentially depends on the reaction driving force (difference between the total alkali content of the concrete and the Threshold Alkali Level of the aggregate) and the consequent total expansion achieved. Although a concrete quality reduction can be observed in laboratory tests, for the dam safety this aspect has been found in some cases limited or of secondary importance, in relation to the amount of expansion developed and eventually to the expansion rate.

Small amounts of expansion can actually enhance the compressive strength and other properties of concrete by offsetting hydration shrinkage effects and making a less permeable product. However, if the expansion is large enough, the deterioration due to the volume change leads to excessive micro- and macro-cracking and will degrade the material properties. Cracking of the cement paste, and even aggregates, reduces the concrete’s modulus of elasticity, the tensile strength and the ability to resist weathering effects such as freeze-thaw. Compressive strength may also decrease. Creep increases considerably [3.39] and this can be beneficial in the case of AAR-affected structures.

Based on results of laboratory mechanical tests performed on AAR damaged concretes, different behaviours are to be expected for concretes with rapid reactive aggregates compared to concretes with slow rate reactive aggregates. In the first case, the concrete is generally characterized by high expansion rates, no strength gain and significant reduction in the elastic properties. In the second case, on the contrary, the concrete is characterized by low expansion rates, compressive strength increase, comparable to that of the reference concrete, and only limited reduction in the elastic properties) [3.40].

Expansion rates in dam concretes are generally found to be low/moderate, usually in the range of 10 to 50 με/year, corresponding to a total expansion values, over a forty years of propagation phase, of about 400 to 2000 με. Only a limited number of reported cases presents concrete expansion rates above this range, up to 200 με/year [3.2]. For example, the concrete of Chambon dam is reported to develop a maximum of 70-80 με/year at its crest and values as low as 10 με/year at the dam's toe. Maximum values as high as about 100 and 150-200 με/year are known for the concrete of the dam of Moxoto, (Brazil) and the generation plant of Mactaquac (Canada), respectively (paragraph 3.2.2 on compressive stresses).

For AAR affected concrete dams, as for other concrete structures is sometimes difficult to assess the real degree of damage due to AAR. In fact the concrete properties are determined on cores and no independent reliable reference values can always be found. Historical values, if available, are to be considered or, alternatively, values of the concrete properties referred to cores taken from different part of the structure, with different degree of damage. Furthermore cores may fall apart along major cracks during coring or sample preparation, so that the intact part of the cores often overestimate the quality of the concrete of the dam body [3.41]. On the other hand the concrete in a reacted concrete dam is usually under some degree of restraint perpendicular to the direction of loading in the form of the surrounding concrete or applied load. This is why concrete properties obtained from cores removed from a dam structure may underestimate the true properties of the in-situ concrete [3.42].

More information on the changes of mechanical properties are discussed further in this chapter. In particular, most of information of the AAR effects on the physical properties of concrete in dams are provided through the analysis of testing results, mainly derived from the Bureau of Reclamation experience in USA. They are particularly valuable and representative of the large variability of concrete proprieties inside the dam body and over the time. In fact they come from a rich database of a special “Ageing Concrete Information System” (ACIS) [3.43], specifically developed to collect records of concrete materials properties from core tests of several large dams.

In particular, the materials properties of concrete incorporated into the ACIS database allow comparative modelling of the expected performance of the USBR (Unites States Bureau of Reclamation) concrete dams, comparing AAR deteriorated dams to dams of similar age, but not suffering from AAR phenomena. In this way trends were established for compressive strength, splitting and direct tensile strength, and elastic properties of ageing and non-ageing dams. The strength and elastic properties of ageing mass concrete differed significantly from those of comparable non-ASR concretes. Both spatial variations within a structure and long-term changes in the concrete properties were identified. They are briefly summarised in the following paragraphs.

3.3.2 *Compressive Strength*

The concrete compressive strength generally decreases as the AAR mechanical damage increases. However compressive strength is relatively little influenced by concrete damage and micro-cracking due to AAR and certainly less than tensile strength and modulus of elasticity. In fact concrete can sustain a considerable amount of compressive loading despite the presence of a large number of micro-cracks and may still have a compressive strength that is fairly close to the original design strength, even for moderate levels of expansion [3.44]. Therefore compressive strength is not to be considered a strong indicator AAR.

Furthermore, compressive strength is seriously affected by restraint perpendicular to the direction of loading. If laboratory tests are carried out on short specimens, the lateral restraints produced by the friction on the loading platens can hide and balance the loss of compressive strength in concrete with AAR.

Consistent reduction of compressive strength (up to about 40%) have been found only at very high unrestrained expansion levels (about 5 mm/m, i.e. 0.5% or 5000 με) [3.45], not common for dam concretes. Similar trend is reported in the UK ISE document [3.46], summarised in Fig. 3.16, that indicate lower bounds to a large number of data of residual mechanical properties (compressive and tensile strength, elastic modulus) of unrestrained concrete, for different AAR expansion levels, as percentages of the actual properties of unaffected concrete. Obviously, being representative of lower bounds to the concrete properties, the reduction values of Table 3.1 generally tend to overestimate the deterioration in concrete with AAR expansion. However the prediction of deterioration in elastic modulus at low expansion level (0.5 mm/m) was considered not sufficiently reliable, since this relationship tends to overestimate the elastic modulus in this expansion range [3.47].

Table 3.1 - Percentage of concrete strengths and elastic modulus as compared with AAR unaffected concrete for different free expansion levels [3.46]

Property	Concrete expansion level			
	0.5 mm/m	1.0 mm/m	2.5 mm/m	5.0 mm/m
Cube compressive strength	100	85	80	75
Uniaxial compressive strength	95	80	60	60
Tensile splitting strength	85	75	55	40
Elastic modulus	100	70	50	35

These values are based on laboratory tests on cast cubes and cylinders as well as on cores extracted from structures but they are not specifically referred to concrete in dams. Compressive strength reduction has been not always observed in the dam engineering experience, also after a relevant expansion. For example, at Serra dam (Switzerland), with an estimated total expansion of almost 1100 με (1.1 mm/m or 0.11%) the concrete maintained excellent properties [3.48]. Unchanged concrete properties have also been observed at the Pian Telessio dam (Italy), which required a rehabilitation [3.2].

Anyway, there are cases in which an appreciable reduction of compression strength due to AAR development has been measured, as for example at the USBR Parker Dam (USA), as shown in Fig. 3.16. In this figure the concrete compressive strength over the time of two USBR dams with comparable concrete mixtures and built at about the same time frame, one with reactive aggregate and affected by AAR (Parker dam) and one with not reactive aggregate and unaffected by AAR (Hoover dam) are compared, to better highlight the change in the compressive strength [3.49] [3.43].

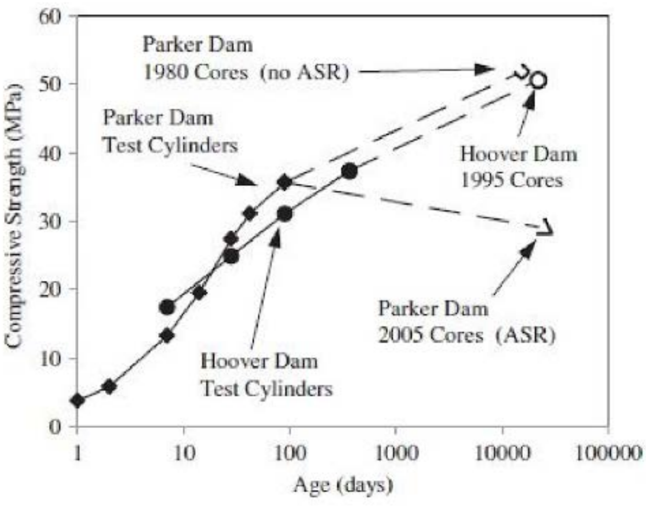


Fig. 3.16 – Effect of AAR on compressive strength of cores in Parker dam [3.49] [3.43]

The trend of Fig. 3.16 was produced by [3.49] using data of [3.43], fully reported in Fig. 3.17. They demonstrate how, just a few years after construction, the AAR affected Parker dam had compressive strengths significantly lower (about 10 MPa) than the reference not AAR affected Hoover dam. It is interesting to note the high compressive results in Fig. 3.17, at 42 years (15,330 days) age, that were identified as coming from mass concrete placed in the lower portion of Parker Dam. This concrete was occasionally prepared with a low alkali cement that was randomly delivered early in the construction of dam. These results have been taken as the compressive strength of the non reactive concrete cores of Parker dam reported in Fig. 3.16. The results of Fig. 3.17 also show the large scattering of compressive strength results coming from a concrete in a large dam.

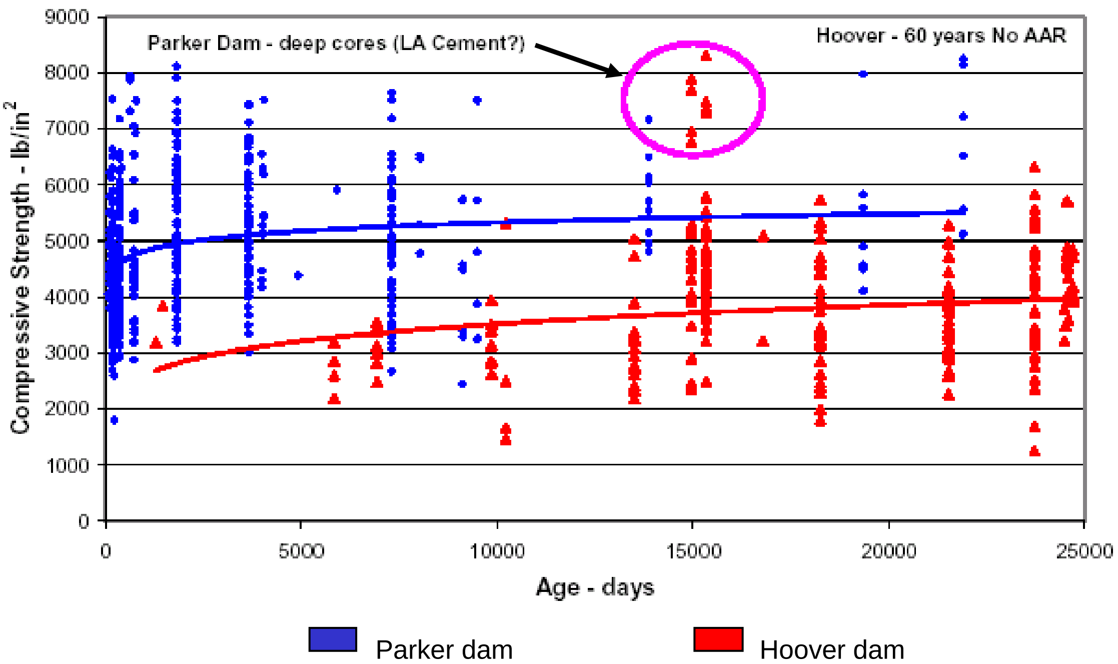


Fig. 3.17 – Compressive strength development over time for mass concrete dams with and without AAR, Parker dam and Hoover dams respectively [3.43]

Unlike AAR that is believed to cause only limited decrease in concrete compressive strength, DEF has been reported to produce a more severe decrease, at least in laboratory. This fact has been explained by DEF higher and growing faster expansions compared to AAR, that usually develops much more slowly. Due to DEF expansions higher and growing faster, the ability to mitigate, at least initially, the concrete deterioration through crack filling by new formation products, a sort of "healing process", could not sufficiently and efficiently develop in DEF affected concretes, in comparison to AAR concretes, thus leading to a more severe decrease of compressive strength [3.20].

3.3.3 *Tensile strength of concrete, lift bond and shear in construction joints*

Measuring the tensile strength of concrete is not an easy task, since different test procedures do not give similar results and experimental data are affected by a large dispersion. Care should be taken in the interpretation of tensile strength values obtained using splitting tests, especially those performed on cores smaller than 150 mm diameter, since they tend to overestimate the tensile strength [3.50]. Whenever possible direct tensile strength tests should be performed.

It is generally thought that tensile strength is much more susceptible to AAR deterioration than compressive strength, which has also been confirmed by measurements on concrete structures in service [3.39] [3.51]. Furthermore, tensile strength might become more critical for the stability of concrete gravity dams, possibly affecting their potential failure modes (PFMs) [3.52].

For maximum expansions that are likely to occur in concrete dams (e.g. 1.0 -2.0 mm/m) a tensile strength reduction of about 25-50%, compared to that of an unaffected concrete is expected [3.46] (Table 3.1). Loss of tensile strength up to 30% has also been reported [3.45]. These values become important in the structural analysis of concrete dams, particularly for dynamic analysis due to earthquakes.

The results of direct and splitting tensile strength of good quality concrete and AAR affected concrete provided by the US Bureau of Reclamation [3.43] and are shown in Fig. 3.18.

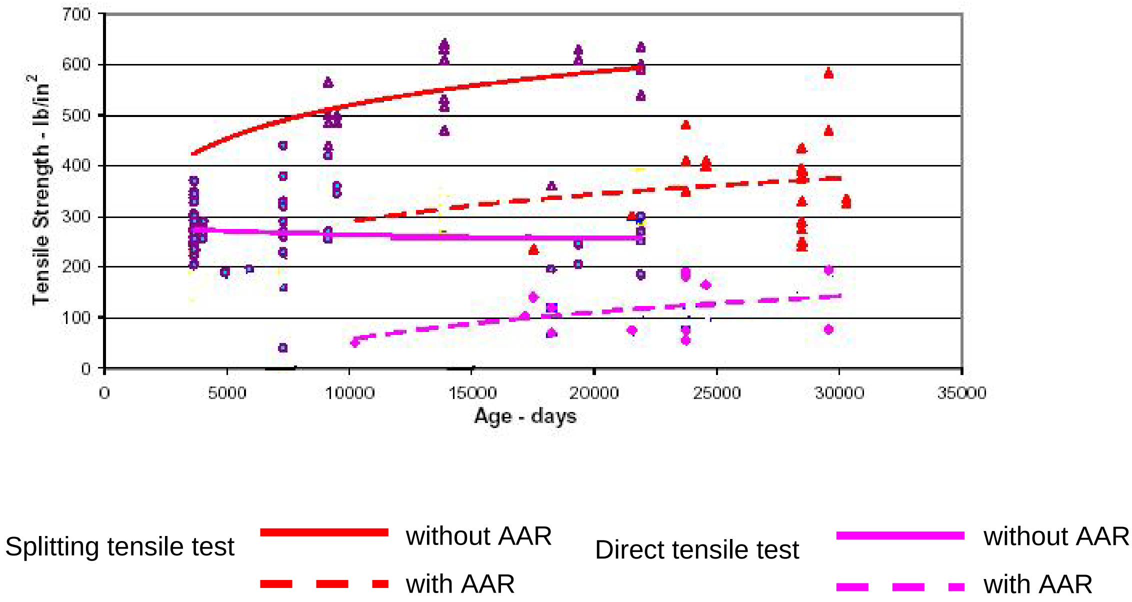


Fig. 3.18 – Comparison of the effects of AAR on tensile strength of mass concrete dams. Adapted from [3.43]

From Fig. 3.18 it is evident that the concrete tensile strength of USBR AAR affected dams' averages about half of the tensile strength of reference dams, without AAR degradation, for both direct and splitting tensile strength. Tensile testing of cores at Mactaquac [3.24] has indicated a significant loss of tensile strength and in some cases this reduction occurred even if no compressive strength loss was evident.

A relevant loss of the tensile strength, due the AAR internal concrete micro-cracking, particularly if associated to a reduced fracture energy (paragraph 3.3.5), may significantly favour initiation and propagation of the structural cracks in dams, inducing dam potential failure modes. Structural cracks are not the direct consequence of the expansion phenomena, but a secondary effect induced by the structure and determined by the permanent displacements or by the non-uniform distribution of the expansion within the massive dam. They appear typically along discontinuities, for example at the transition between a straight gravity dam and a curved part, or along the foundation as peripheral cracks on the downstream face of arch dams. In fact, the upstream drift in arch dams produces tensile stresses at the downstream dam toe in case of low water level in the reservoir.

Within the activities of the above mentioned “Ageing Concrete Information System” (ACIS), the average direct tension and shear properties of lift lines in concrete dams built between 1925 and 1938 by USBR in USA were estimated for mass concrete suffering from AAR in comparison to dams constructed during the same period of time but not suffering from AAR. The lift lines bond in AAR affected dams was about 30% of comparable not affected dams. Instead, the average direct tensile strength of AAR-affected parent concrete was found about 40% of the not affected concrete. As for the shear and sliding friction properties of lift lines, in the AAR affected dams the cohesion was about 70% of the comparable reference concrete while the internal friction angle was reduced by about 10% [3.53].

3.3.4 *Modulus of elasticity*

The decrease in modulus of elasticity is more apparent than the loss of compressive strength in AAR affected concrete dams for both early and long term ages. Reduction of this parameter is expected also for expansions less than 0.5 mm/m. However at larger expansion levels (1- 2 mm/m), the reduction of the elasticity modulus can reach values of about 30-50 % (Table 3.1 [3.46]). This can lead to apparent “low stresses” using conventional linear elastic structural analysis. However, these analyses should be used with caution as the behaviour may be best represented using nonlinear analysis.

One possibility to assess how much AAR has influenced the mechanical properties is a comparison between the modulus of elasticity and the compressive strength. Micro-cracks increase strains at the relatively low stress levels used for testing the modulus while they have an insignificant effect at the high stress level at which the concrete fails in the compressive strength testing. Therefore the comparison between modulus and compressive strength may indicate the level of damage [3.41].

This is clearly shown by core testing results provided by USBR ACIS database [3.43] reported in Fig. 3.19. This figure provides the relationship between compressive strength and modulus of elasticity for all USBR dam concrete cores, with and without ASR. Although the correlation coefficient for the equations is poor, the trend lines clearly highlight the demarcation between the two classes of concretes. A considerable reduction of the elasticity modulus for these dams is marked, in accordance with the previsions in the case of high expansion levels, above 1 mm/m. Usually relationships between concrete strength and modulus of elasticity cannot be adopted for concretes affected by AAR.

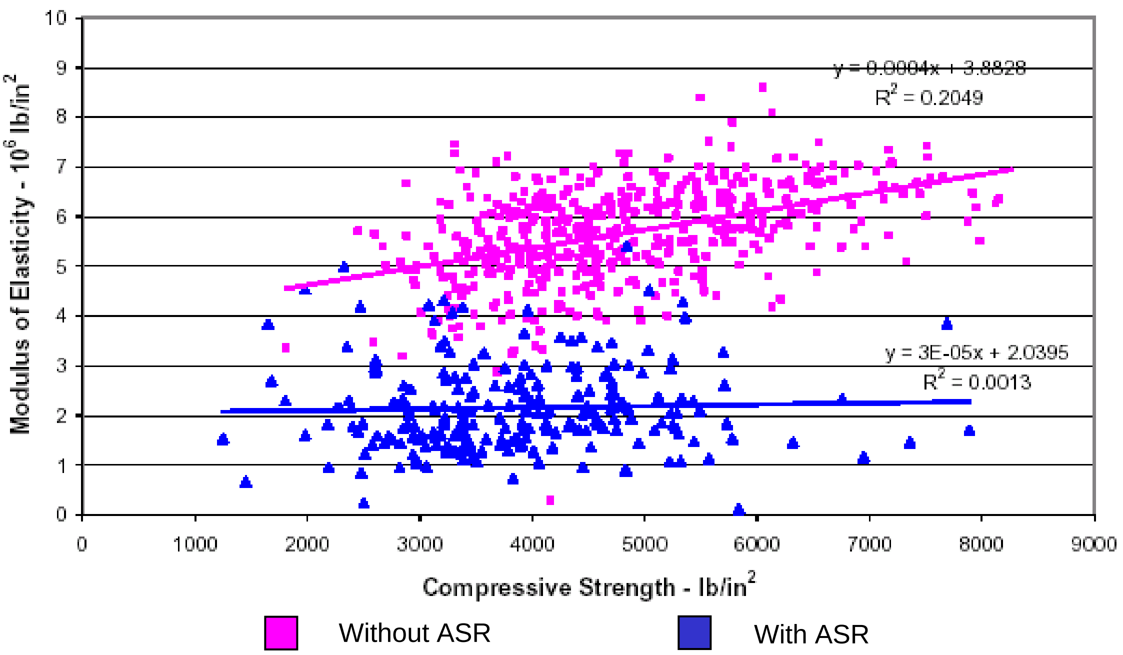


Fig. 3.19 – Compressive strength versus modulus of elasticity for USBR mass concrete cores [3.43]

Individual correlations between compressive strength and modulus of elasticity are normally much better for individual dams using the same aggregate types. The more the distinction between reference values and actual values of the modulus is apparent, the more the concrete is supposed to be damaged by the expansive reaction. If AAR is suspected in dams, the compressive strength to modulus of elasticity ratios and spatial orientation may provide the best supporting documentation for evaluation purposes. An alternative approach to assess the degree of damage is that of the stiffness damage test further discussed in the Chapter 4 on diagnosis.

Also in the case of ISR/DEF affected concrete, the modulus of elasticity is greatly affected by the presence of microcracks resulting from the sulphate expansion and can be reduced significantly even by modest expansion levels [3.44].

The determination of the concrete modulus of elasticity with back calculation of long term compressive stress development in structural analyses is impracticable, as it is not possible to distinguish between creep effect of actual reduction in elastic modulus. Elastic modulus reduction can only be assessed when analysing the displacements due to water level variations.

3.3.5 *Creep*

A very important property with respect to concrete dams suffering from AAR is the effect that this reaction has on creep of the concrete. It is reported as significantly increased in the presence of AAR and, with this increase, it has a remarkable effect of relaxing the build-up of stresses in the structure. Literature data shows that concrete affected by AAR has a creep coefficient 2-4 times greater than that of the reference not expansive concrete [3.39]. Creep coefficient is the ratio of the ultimate creep strain to the elastic strain at the age of loading.

Another beneficial effect of creep is that it tends to reduce the stress applied to steel embedded or affixed to the concrete. Creep is very important to realise in the analysis of AAR-affected structures where stresses in both the concrete as well as embedded steel could be overestimated several times if creep is ignored. Simplified methods used to take into account the creep effects are presented in Chapter 6. In the context of the very slow loading process produced by AAR in dams, creep may explain why some concretes subjected to important swelling are still in good conditions [3.54].

3.3.6 *Fracture energy*

The most important input parameter for nonlinear fracture mechanics calculations of concrete is undoubtedly the stress-crack opening relationship and the specific fracture energy G_F is a derivative parameter of this relation. This parameter is reported to decrease as the AAR process develops. Few unpublished data from Tschegg E.K. and referred to in [3.39] suggest a loss of G_F of approximately 25% for expansion levels of 2 mm/m. Fig. 3.20 shows the shape of the curve between applied force and crack opening, providing a qualitative information about the AAR affected concrete ductility. No appreciable changes on the specific fracture energy, G_F, were instead measured due to the induced damage in concrete by ettringite formation, testing concrete specimen at different expansion levels [3.55]. However, these results on the effect of expansion phenomena on concrete fracture properties still need to be confirmed by future advanced research, particularly on dam concrete,

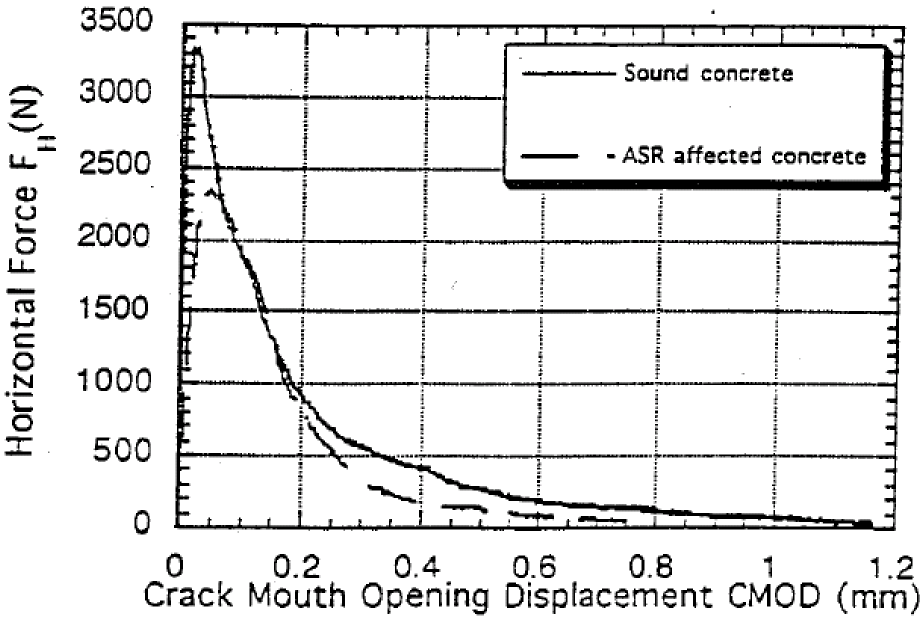


Fig. 3.20 – Load – Crack opening curve for sound and ASR affected concrete [3.39]

3.4 Effects of expansion on reinforcement bars and bond strength

The presence of reinforcing bars, acting as restraints inside concrete suffering from expansive phenomena, like for example in powerhouse concrete structures, is able to reduce the concrete expansion in the direction of bars, causing compression into the concrete (chemical pre-stressing effect). However, it can also induce the buildup of additional stresses into the bars themselves. A moderate concrete expansion of about 0.1% (1000 με) may bring into the steel bars additional stresses of about 50 – 100 MPa, depending on the structural element and size and on the steel reinforcement ratio, so that a critical stress condition may arise in the long term [3.56] [3.57] [3.58], taking also into account reasonably foreseeable reduced concrete properties (previous paragraph 3.3). Larger expansions can even yield reinforcements, but this may be not detrimental, provided they are well anchored and ductile [3.59].

The expansion of the concrete and restraint provided by internal reinforcement can also induce a change in the bond stresses at the concrete – reinforcing bar interface [3.44]. In case of insufficient thickness of cover concrete for reinforcements (under 3-4 the bar diameter), there is evidence of delamination and loss of anchorage developing with severe AAR. The bond strength of bars not restrained by links and with cover of the order of 1.5 times the bar diameter is reduced by up to 50%, proportional to the reduction in splitting tensile strength of the AAR affected concrete [3.46] [3.60] [3.61].

Based on both experimental testing and numerical modeling, it was found [3.62] [3.63] that the concrete – reinforcing bond initially increases with the increase of AAR effects, but decreases when AAR deterioration reaches a certain level (Fig. 3.21). The bond between concrete and reinforcement has crucial effects on behaviour of reinforced concrete structures, affecting their load carrying capacity and safety.

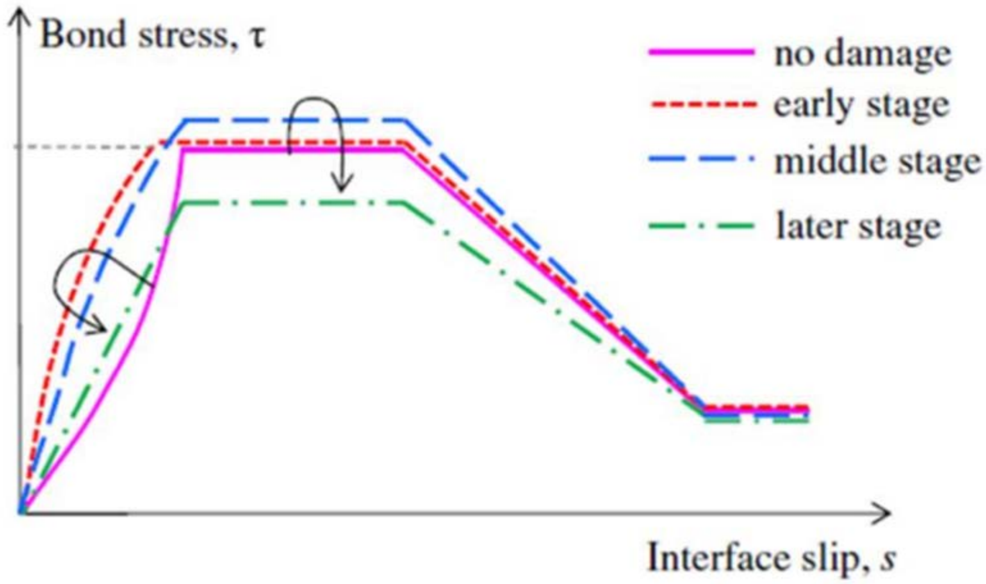


Fig. 3.21 - Changes in bond behaviour at different stage of AAR ([3.62], reported by [3.63])

Both bending moment capacity and shear capacity of reinforced concrete structural elements are reported to increase at the earlier expansions, probably due to the beneficial chemical pre-stressing effect. However, with long-term expansions, a reduction in bending and shear capacities can be expected (Fig. 3.22), even if the long term behaviour still need to be further studied [3.64].

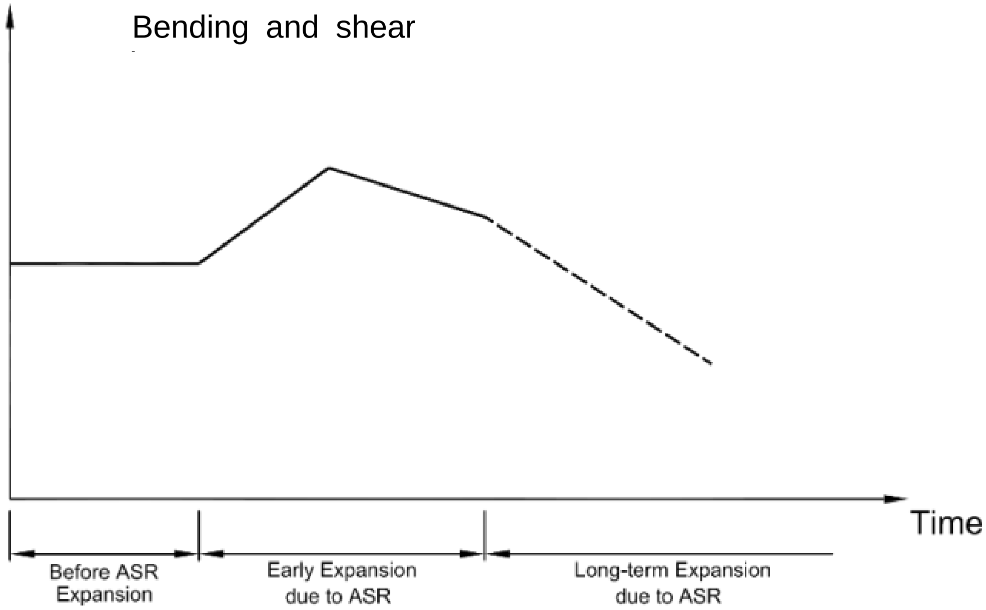


Fig. 3.22- Effects of different degree of Alkali-Aggregate Reaction deterioration on bending and shear capacity of reinforced concrete structural elements (adapted from [3.62])

3.5 References

[3.1] USCOLD, Second International Conference on Alkali-Aggregate Reactions in Hydroelectric Plants and Dams, Chattanooga, Tennessee, USA, 22 - 27 October, 1995.

[3.2] Amberg, F., “Performance of dams affected by expanding concrete”, Proc. Dams and reservoirs under changing challenges, 79 Annual meeting of ICOLD, Swiss Committee on dams, Lucerne, CRC Press, pp. 115-122, June 2011.

[3.3] Charlwood, R.G., Solymar, Z. V., “A review of alkali–aggregate reactions in dams”, Dam Engineering, Vol. 5, n. 2, pp. 31–62,1994.

[3.4] Swiss Committee on Dams, “Concrete swelling of dams In Switzerland”, Report of the Swiss Committee on Dams on the state of concrete swelling in Swiss Dams, AAR Working Group, May 2017.

[3.5] Otto, B., “AAR and ISA at two dams in Switzerland”, ICOLD Workshop of chemical expansion on concrete in dams and hydro projects, Granada, 18-19 October 2007.

[3.6] Pérez Rodríguez, D.A., Pazo Garcìa, I., Alonso Munoz, J.M., Gonzalo Carracedo, A., “La rehabilitacion del aliviaderodel la presa de Portodemouros”, Proc. 2nd International Congress on Dam maintenance and rehabilitation, Zaragoza, 22-23 November 2010.

[3.7] Leroy, R., Micoulet, G. & Tognola, F., “Rehabilitation of Serra dam (Switzerland) affected by ASR”, Proc. 2nd International Congress on Dam maintenance and rehabilitation, Zaragoza, 22-23 November 2010.

[3.8] Tuthill, L.H., “Alkali-Silica Reaction, 40 years later”, Concrete International, pp. 32-36, April 1982.

[3.9] Bourdarot E., “Analyse du comportement à long terme des barrages-voûtes”, EdF Centre d’Ingénierie Hydraulique, Colloque Tecnique CFGB, Aix en Provence, 10-11 May, 2001.

[3.10] Dodson, A., Curtis, D., “Fontana and Hiwassee dams – Managing Alkali-Aggregate Reactions”, International. Workshop on Alkali Aggregate Reactions, 81th ICOLD Annual Meeting, 20 Aug 2013.

[3.11] Fiedler, W.R., “Bureau of Reclamation experience with AAR”, International. Workshop on Alkali Aggregate Reactions, 81th ICOLD Annual Meeting, 20 Aug 2013.

[3.12] Fletcher, J., Curtis, D. “Managing AAR at Mactaquac GS”, International. Workshop on Alkali Aggregate Reactions, 81th ICOLD Annual Meeting, 20 Aug 2013.

[3.13] Batista, A.L., Gomes, J.P. “The Portuguese large concrete dams affected by swelling processes”, International. Workshop on Alkali Aggregate Reactions, 81th ICOLD Annual Meeting, 20 Aug 2013.

[3.14] Pina, C., “Expansive reactions in concrete dams in Portugal - the old and the new ones”, Presentation at annual Symposium of the French Committee on Dams and Reservoirs, Chambéry (France), 26 January 2017.

[3.15] Charlwood, R.G., Sims, I., “A review of the effectiveness of strategies to manage expansive chemical reactions in dams and hydro projects”, Proc. Symposium on the Dam Swelling Concrete, DSC 2017, Chambéry, France, pp. 3.-39, 13-15 June 2017.

[3.16] Aguado, A., Serafim G., Chinchon, S., Lopez, C.M., “Internal Sulphate Attack in dams”, Special Workshop “Chemical expansion of concrete in dams and Hydro-electric projects”, Granada, 18 October 2007.

[3.17] Aruado, G.S., Chinchon, S., Aguado, A., “Evaluation of the behaviour of concrete gravity dams suffering from internal sulphate attack”, Ibracon structures and materials journal, Vol.1, n.1, pp. 84 - 112 March 2008.

[3.18] Andriolo F.R., “Expansion effects of phyrite in the rock matrix of the Rio Descoberto dam”, ICOLD Workshop of chemical expansion on concrete in dams and hydro projects, Granada, 18-19 October 2007.

[3.19] Kert, M.C., “Rapport sur l’amélioration de la sécurité des barrages et ouvrages hydrauliques”, Session extraordinaire du Sénat Français, Annexe au procès-verbal de la séance du 9 juillet 2008.

[3.20] Martin, R.P., Bazin, C., Toutlemonde, F., “Alkali aggregate reaction and delayed ettringite formation: common features and differences”, Proc. 14th International Conference on Alkali-Aggregate Reaction, Austin, Texas, USA, 2012.

[3.21] Larive, C., “Apports combinés de l’expérimentation et de la modélisation à la compréhension de l’alcali-réaction et de ses effets mécaniques”, PhD thesis, Paris: LCPC. 1998.

[3.22] Curtis, D., Feng, L.F., Sooch, G.S., Zheng, J.T., Fletcher, J., “Practical analysis and assessment of AAR-affected dams and hydroelectric plants, Proc. 36th Annual USSD Conference, Denver, Colorado, pp. 675-694, April 11-16 2016.

[3.23] Brueckener, R., Ndugga, N., Meri, T.C., “Effects of alkali-silica reaction on a hydropower structure after 50 years of ongoing deterioration”, MATEC Web of Conferences 199, 03008, ICCRRR 2018 (2018)

[3.24] Charlwood, R.G., Solymar, S.V., Curtis, D.D., “A Review of Alkali Aggregate Reaction in Hydroelectric Plants and Dams”, CEA and CANCOLD, Fredericton, pp 1-29, September 1992.

[3.25] Thompson, G.A., Charlwood, R.G., Steele, R.R., Curtis, D.D.,” Mactaquac Generating Station Intake and Spillway Remedial Measures”, Proc. 18th International Congress on Large Dams, Durban, South Africa, Vol. 1, Q: 68, R: 248, pp. 342-368, November, 1994.

[3.26] Batista, A.L., Carvalho, E.F., Tembe, I.M., “Comprehensive assessment of the Cahora Bassa dam´s concrete swelling process”, Proc. 15th ICAAR - International Conference on Alkali Aggregate Reaction in Concrete, Sao Paulo, Brasil, 3-7 July 2016.

[3.27] Bouzabata, H., Multon, S., Sellier, A., Houari, H., “Effects of restraint on expansion due to delayed ettringite formation”, Cement and Concrete Research, Vol. 42, n. 7, pp.1024- 1031, 2012.

[3.28] Charlwood, R.G., Scrivener, K., “Expanding concrete in dams –long term challenges”, Proc. Dams and reservoirs under changing challenges, 79 Annual meeting of ICOLD, Swiss Committee on dams, Lucerne, CRC Press, pp. 179-186, June 2011

[3.29] Scherer, G.W., “Factors affecting crystallisation pressure”, Proc. International RILEM TC 186-ISA, workshop on ISA and DEF, Villars (Switzerland), pp. 139-154, September 4 – 6, 2002

[3.30] Baghdadi, N., Seignol, J.-F., Martin, R.-P., Renaud, J.-C., Toutlemonde, F.: “Effect of early age thermal history on the expansion due to delayed ettringite formation: experimental study and model calibration”, Proc. Euro Mediterranean symposium on advances in geomaterials, Hammamet (Tunisia), pp. 661-666, 2-7 May, 2008

[3.31] Duchense, J., Bérubé, M.A., “Available alkalies from supplementary cementing materials”, ACI Materials Journal, Vol. 91, n. 3, pp. 289-299, May 1994.

[3.32] American Society for testing and Materials, ASTM C311-18 Test Method, “Standard Test Methods for Sampling and Testing Fly Ash or Natural Pozzolans for Use in Portland-Cement Concrete”, Philadelphia, 2018.

[3.33] RILEM, “Recommended Test Method: AAR-3 Detection of potential alkali-reactivity – 38°C test method for aggregate combinations using concrete prisms” – Application 2 – AAR3.2: Method for determining the critical alkali threshold for an aggregate combination:”, in RILEM Recommendations for the prevention of damage by Alkali-Aggregate Reactions in new concrete structures, , RILEM State-of-the-art Report, Springer, Nixon, P.J. and Sims, I. Eds., vol. 17, pp. 79-97, 2016.

[3.34] Berra, M., Mangialardi, T., Paolini, A.E., “Alkali Release from aggregates in long-service concrete structures: laboratory test evaluation and ASR prediction”, Materials, 11, 1393; 2018.

[3.35] Berra, M, Mangialardi, T., Paolini, A.E., “Alkali–silica reactivity criteria for concrete aggregates”, Materials and Structures, Vol. 38, n. 3, pp. 373-380, April 2005.

[3.36] Bazant, Z. P.; Wittman, F. H., “Creep and shrinkage in concrete structures”, New York: John Wiley and Ltd, 374 p., 1982.

[3.37] Campos de Moura, A., “Análisis numérico de presas de hormigón bajo acciones expansivas”, Tesis doctoral, Universitat Politècnica de Catalunya, Departament d´Enginyeria de la Construcció, Barcelona (Spain), November 2012.

[3.38] Nixon, P.J., Collins, R.J. and Rayment, P.L., ‘The concentration of alkalies by moisture migration in concrete-A factor influencing alkali-aggregate reaction’, Cement and Concrete Research, Vol. 9, n. 4, pp. 417-423, 1979.

[3.39] Rotter H., M., “Alkali-Aggregate Reaction: from basic principles to structural behaviour – A literature review, Rapport n° EMP/GSC 1995-11, Ecole Polytechnique Montréal, pp. 166, 1995.

[3.40] Giaccio, G., Zerbino, R., Ponce, J.M., Batic, O.R., “Mechanical behaviour of concretes damaged by alkali-silica reaction”, Cement and Concrete Research, Vol. 38, pp. 993-1004, 2008.

[3.41] Leeman, A., Griffa M., “Diagnosis of alkali-aggregate reaction in dams”, SFOE-Project SI/500863-01, EMPA Swiss Federal Laboratories for Material Science and Technology, 2013.

[3.42] Jones, A.E.K., Clark, L.A., “The effects of ASR on the properties of concrete and the implications for assessment”, Engineering Structures, Vol. 20, pp. 785-791, 1998.

[3.43] Dolen, T., “Materials properties model of aging concrete. Technical Report”. Report DSO-05-05, U.S. Department of the Interior, Bureau of Reclamation, Materials Engineering and Research Laboratory, 86-68180, 2005.

[3.44] Noël, M., Sanchez, L., Tawil, D., “Structural implications of internal swelling reactions in concrete: review & research needs”, Magazine of Concrete Research, Vol. 70, n. 20, pp. 1052-1063, October 2018.

[3.45] Swamy, R. N., Al-Asali, M. M., “Influence of alkali-silica reaction on the engineering properties of concrete”, Alkalies in concrete, ASTM STP 930, Ed. V.H. Dodson, American Society for testing and Materials, Philadelphia, pp. 69-86, 1986.

[3.46] ISE (Institution of Structural Engineers), “Structural effects of alkali-silica reaction. Technical guidance on the appraisal of existing structures”, 1992.

[3.47] Jones, A.E.K., Clark, L.A., “A review of the Institution of Structural Engineers Report “Structural effects of Alkali-Silica Reaction (1992)””, Proc. 10th International Conference on AAR, Melbourne, Australia, pp. 394-401, 1996.

[3.48] Amberg, F., “Recent experiences with expanding concrete in the Alpine region”, ICOLD Workshop on "Chemical expansion of concrete dams & hydroelectric projects, Granada Spain, 18–19/10/2007.

[3.49] "Alkali-Aggregate Reaction in Concrete: A World Review", Editors Sims, I., Poole A.B., CRC Press, 1st Edition, 2017.

[3.50] RILEM State-of-the-art Report, "Guide to diagnosis and appraisal of AAR damage to concrete in structures – Part 1 Diagnosis (AAR 6.1), Springer, Godard, B., de Rooij, M., Wood, J.G.M. Eds., vol. 12, p. 87, 2013.

[3.51] Swamy, R. N., M.M., Al-Asali, "Engineering properties of concrete affected by alkali silica reaction", ACI Material Journal, pp. 367-374, 1988.

[3.52] Amberg, F., "Stability of concrete dams - Some basic requirements", 13th ICOLD International Benchmark Workshop on Numerical Analysis of Dams, Lausanne, Switzerland, 9-11 September 2015.

[3.53] Dolen, T.P., "Selecting strength input parameters for structural analysis of aging concrete dams", Dam maintenance and rehabilitation II - Romeo Garcìa et al. Editors, Taylor & Francis Group, London, , pp. 240-251, 2011.

[3.54] Lupiem, R., "Mass concrete swelling", Proc. Canadian Dam Association, 3rd Annual Conference, Regina, Saskatchewan, Canada, pp. 240-251, 16-21 September 2000.

[3.55] Rocco, C., Giangrasso, F., Bergol, L., Di Pace, G., Planas, J. "Fracture properties of concrete exposure to delayed ettringite formation", Proc. FraMCoS-5 (Fracture Mechanics for Concrete and Concrete Structures), Vail (USA), 2004.

[3.56] Swamy, R.N., "Effects of Alkali-Aggregate Reactivity on material stability and structural integrity", CANMET/ACI International Workshop on AAR in Concrete, Dartmouth, Nouvelle-Écosse, pp. 293-309, 1995.

[3.57] Fournier,B., Sanchez, L., Beauchemin, S., "Outils d'investigation de la réactivité alcalis-granulats dans les infrastructures en béton", Final Report, Université Laval, Canada, August 2015.

[3.58] Godard, B., Wood, J.G.M., " Appraising structures affected by the alkali–aggregate reaction", Construction Materials, Paper 1500068, Proceedings of the Institution of Civil Engineers, 2016.

[3.59] Chana, P.S., "Bond strength of reinforcement in concrete affected by Alkali Silica Reaction", British Cement Association, Digest of Contractor Report 141, Transport and Road Research Laboratory, Department of Transport, 1989.

[3.60] ISE (Institution of Structural Engineers), "Structural effects of alkali-silica reaction. Technical guidance on the appraisal of existing structures", Addendum, April 2010.

[3.61] Huang, Q., Gardoni, P., Trejo, D. & Pagnotta, A., "Probabilistic model for steel–concrete bond behaviour in bridge columns affected by Alkali Silica Reactions", Engineering Structures, vol. 71, pp. 1–11, 2014.

[3.62] Gowripalan, N., Cao, J., "Effect of Alkali Silica Reaction on bond strength and load capacity of reinforced concrete structures", Proc. 5th International fib Congress, Melbourne, Australia, 7-11 October, 2018.

[3.63] Hansen, S. G., Barbosa, R. A. & Hoang, L. C., "Prestressing of reinforcing bars in concrete slabs due to concrete expansion induced by Alkali-Silica Reaction", Proc. fib Symposium on Performance-based approaches for concrete structures, 2016.

4 DIAGNOSIS IN EXISTING DAMS AND HYDROELECTIC PROJECTS

4.1 Introduction

When dealing with a deteriorated concrete dam the first step is to identify the cause of the damage and to assess the role that expansive phenomena like alkali-aggregate or sulphate reactions have played in generating the observed dam behaviour, together or with the contribution of other possible external or internal actions. Based on a broad investigation of materials and methods used in the dam construction, a reliable diagnosis can only be made through a detailed visual inspection of the dam (paragraph 4.1), combined with an accurate interpretation of the monitoring data throughout its service life (paragraph 4.2). A comprehensive analysis of both laboratory (paragraph 4.3) and field investigation (paragraph 4.4) results is also required, associated with the damage assessment of the dam (paragraph 4.5), able to provide a quantitative measure of the expansive damage. This diagnostic process may also be helpful when forecasting the future of the dam (paragraph 4.6), before or even after the repair interventions adopted to safety manage the structure.

The various stages of this process are basically dealt with in this chapter but more detailed recommendations can be found in some general guides specifically devoted to this aim, as for example the RILEM "Guide to diagnosis and appraisal of AAR damage to concrete structures" [4.1], the British Cement Association report on "The Diagnosis of alkali-silica reaction" [4.2], and the Canadian Document from Laval University "Outils d'investigation de la réactivité alcalis-granulats dans les infrastructures en béton" [4.3].

4.2 Visual inspections and interpretation of monitoring data

Generally, the first symptoms that can induce operators to suspect the presence of an expansive phenomenon on the concrete dam are anomalous or unexpected inelastic movements of the structures, evidenced by the installed monitoring systems, by limitations of the gate operation or by misalignment of rotating machinery, even before the appearance of concrete deterioration or the formation of cracks in the dam.

The difference between predicted values obtained through an idealized mathematical model of the dam, under known loading and thermal conditions, and the actual monitoring data is, indeed, the true criteria to correctly identify anomalies or unexpected behaviours. They may concern the global movements of the concrete dam body, both in vertical and horizontal direction, generally upstream, differential movements between parts of the dam (for example misalignments in the crest or distortion of embedded elements), progressively closing of vertical expansion joints, due to horizontal compressive stresses, loosing of horizontal lift joints with water leakages etc. The closure of expansion joints may be accompanied by the extrusion of the sealing materials and sometimes by the spalling of concrete near the joints. Types and severity of these effects also depend on the structural geometry.

Fig. 4.1 and Fig. 4.2 show an example of upstream displacements of an arch concrete dam while Fig. 4.3 and Fig. 4.4 show an example of downstream displacements of an hallow gravity concrete dam detected over a period of more than 30 years through suitable pendulums installed in the structure. Vertical movements of the top of the same hallow gravity concrete dam (about 1 mm/year), measured through a leveling system are shown in Fig. 4.5 and Fig. 4.6. Fig. 4.7 shows the concrete spalling along a vertical joint of a gravity dam suffering Alkali-Silica Reaction.

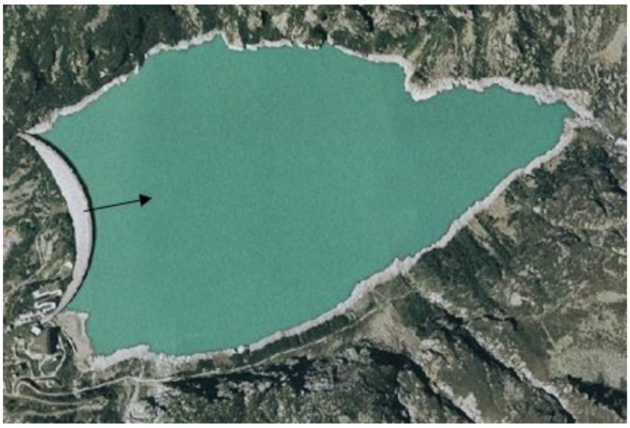


Fig. 4.1 - Horizontal upstream movements of a concrete arch dam.

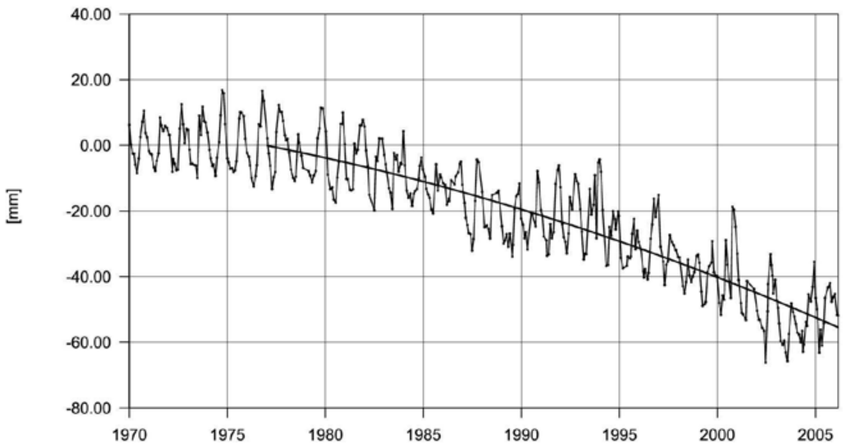


Fig. 4.2 - Downstream radial crest displacement at an arch dam central section [4.4]

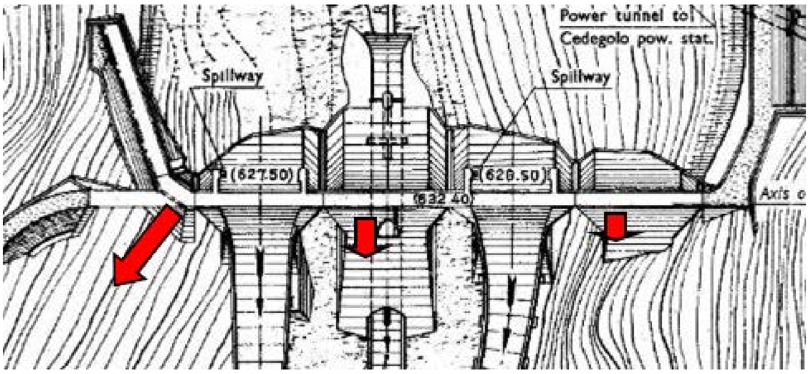


Fig. 4.3 - Horizontal downstream movements of different concrete blocks of a hallow gravity concrete dam.

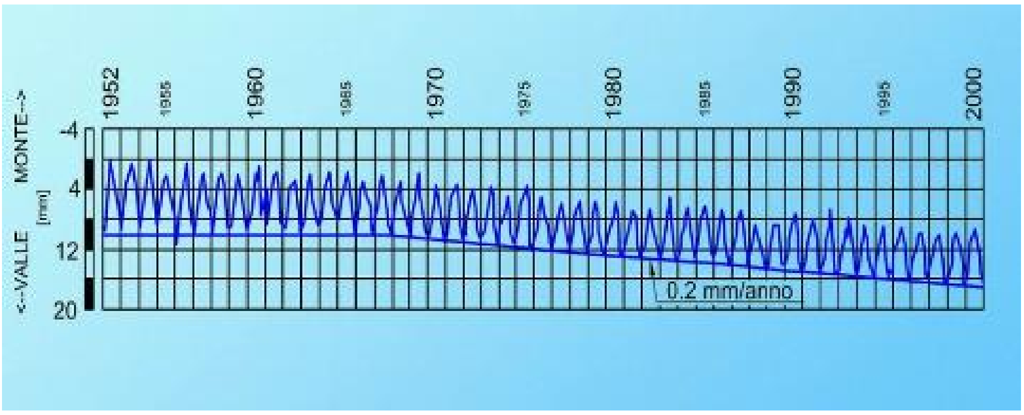


Fig. 4.4 - Downstream drift trend of the main block of the hallow gravity concrete dam of Fig. 4.3 (0.2 mm/year) [4.5]

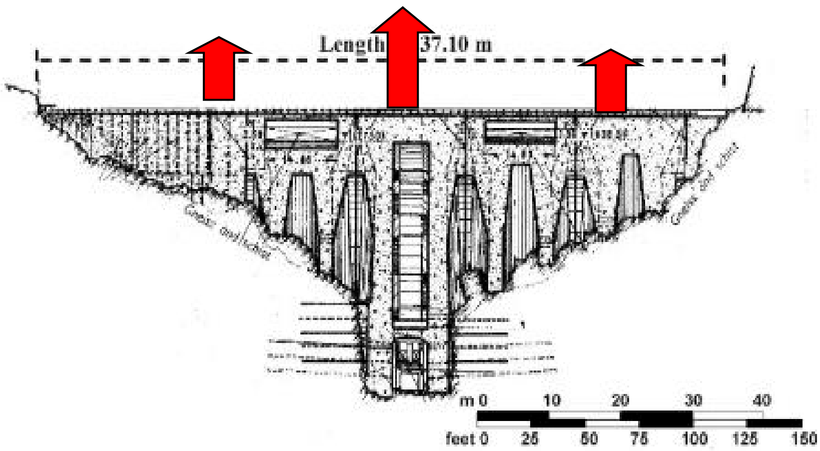


Fig. 4.5 - Vertical movements of different concrete blocks of a hallow gravity concrete dam.

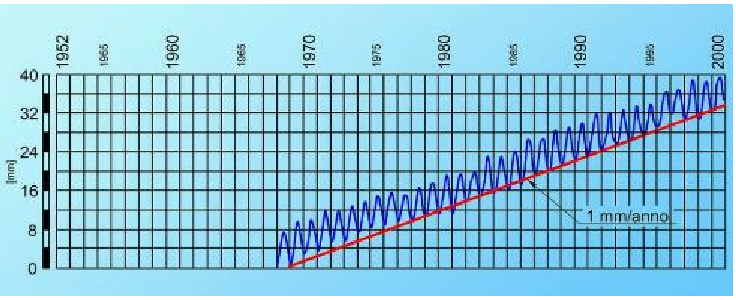


Fig. 4.6 - Vertical drift trend of the main block of the hollow gravity concrete dam (1 mm/year) Fig. 4.5 [4.5].

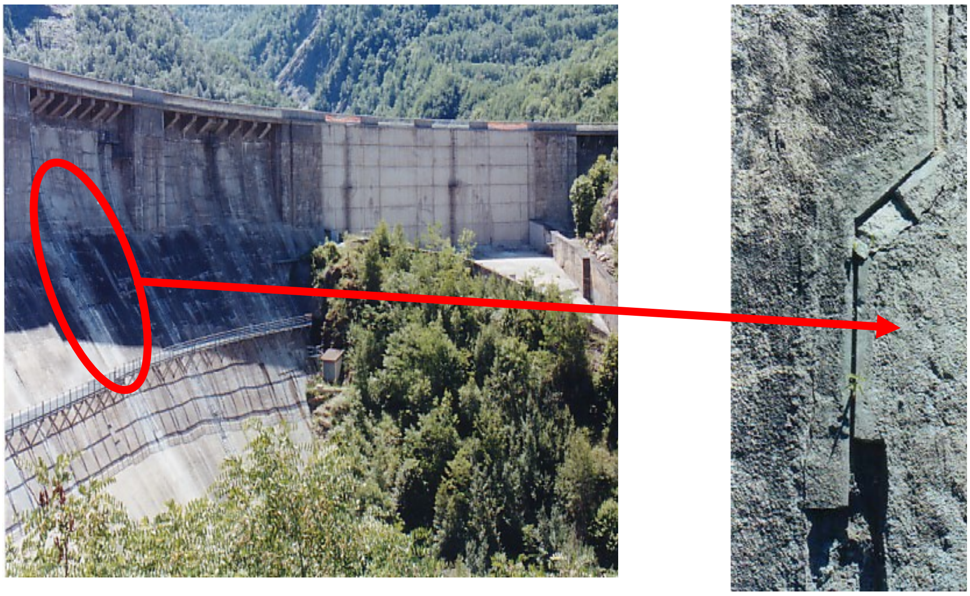


Fig. 4.7 - Gravity dam suffering from Alkali-Silica Reaction with spalling of the concrete near the vertical joints (particularly on the right)

As already discussed in the previous chapters, AAR in concrete develops slowly, at the beginning with no sign of swelling (initiation phase), and then gradually progresses up to reach concrete expansion levels able to evidence the first symptoms in the dam, about 20 - 30 years or even more after the dam construction (propagation phase). These harmful levels of expansion (about 0.01 – 0.02%) are generally much lower than those usually assumed for normal structural concrete (0.04 - 0.05%), taking into account the massive nature of the dam structure. For dams, such values (0.04 - 0.05%) are already indicator of large expansions [4.6]. Naturally the tolerable expansion limit, in a dam will vary, depending on the dimensions, configuration and related equipment [4.7].

The expansion rates of concrete during the propagation phase may vary considerably, depending on the aggregate reactivity, its Threshold Alkali Level (TAL) and the pore solution alkalinity in the concrete, this last being related to the mix design and material used during construction and to the alkali supply during the dam long service life (released from the aggregates or recycled from the gels). As discussed in Chapter 3, in most cases averages values of concrete unrestrained (free) expansion rates lie within a range of 10 to 50 με/year (0.01 to 0.05 mm/m/year or 0.001 to 0.005 %/year), with cases of extremely high free expansion rates, up to 100 με/year (0.1 mm/m/year or 0.01%/year) or more. Restrained conditions or compressive stress of only few MPa (e.g. 5 to 8 MPa) are able to slow down the expansion and eventually to stop it.

The progressive development of concrete swelling and consequent movements of the dam cause build-up of tensile stresses with cracks formation, sometimes together with concrete deterioration (spalling and pop outs), gel exudations and surface staining or discoloration.

On the unrestrained concrete of upstream and downstream dam faces, the cracks tend to present a random distribution, characterized by a network of fine cracks joined up in polygonal shapes and confined by larger cracks in a pattern usually named “map cracking” (Fig. 4.8). This form of cracking is not typical of expansive reactions only but can also result from other phenomena such as abnormal shrinkage. The time of first appearance is, however, quite different, as cracking due the concrete shrinkage usually appears early in the life of a structure.

Because the least restraint occurs in a direction perpendicular to the surface, these cracks tend to align sub-parallel to the surface and usually do not go to much deeper from the exposed surface (Fig. 4.9). Naturally other form of concrete deterioration can take advantage from this crack pattern, like for example freezing and thawing. Furthermore, due to the dam self weight and water hydraulic pressure, in the upper part of the dams the concrete expansion is generally less restrained, and here cracking is more easily found.

Surface cracking does not usually pose serious structural problems but sometimes other types of cracks can be observed in dams affected by expansive reactions, affecting the structural integrity, for example within the dam body, in the internal cavities, in correspondence of joints or other structural discontinuities. For example, expansive upstream crest movements, coupled with foundation restraints, can lead to increasing vertical compression on the upstream face and horizontal fissure opening downstream (Fig. 4.10 and Fig. 4.11). Depending on the geometry of the dam, deformations can also induce stress concentrations able to develop even more complex crack patterns (Fig. 4.12 and Fig. 4.13).

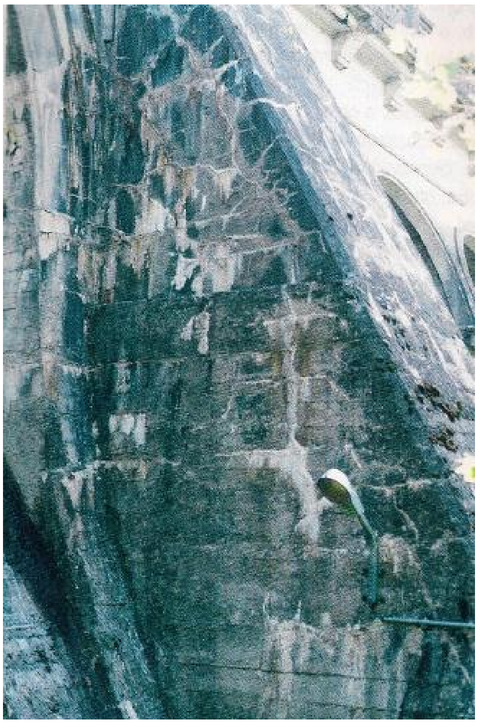


Fig. 4.8 - Surface map cracking on a dam concrete buttress with gel exudation and staining

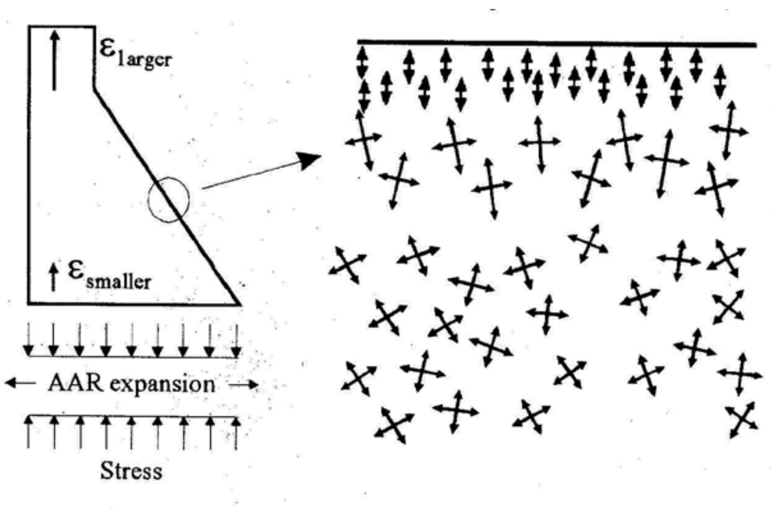


Fig. 4.9 - Sketch of cracking on the downstream face of a gravity dam affected by expansive reactions

Crack

Fig. 4.10 - Structural crack in an arch gravity dam affected by expansive reactions

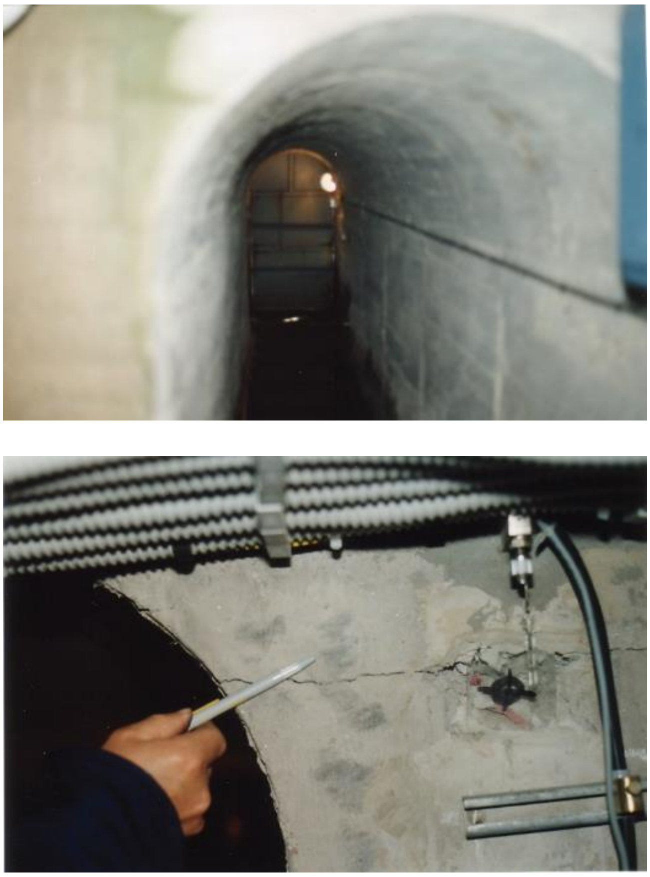


Fig. 4.11 - Particular of the structural crack in the upper gallery

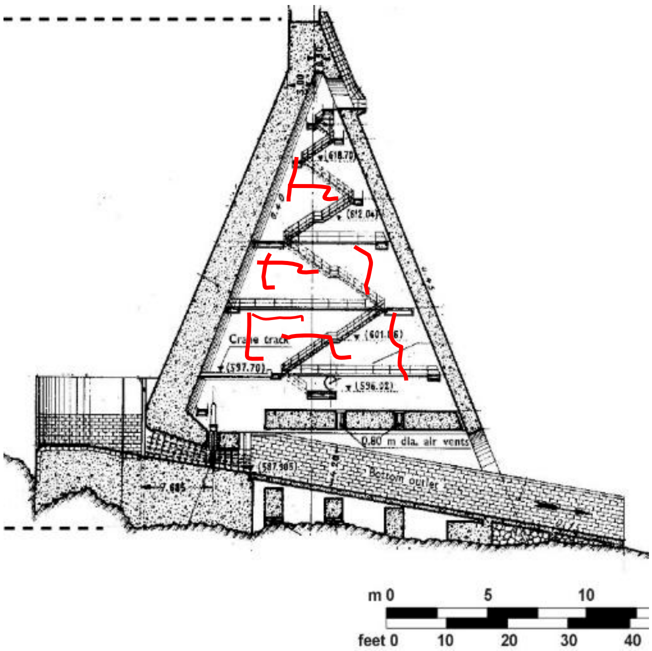


Fig. 4.12 - Vertical cross section along the main buttress of a hallow gravity dam and the structural cracks inside the dam cavity

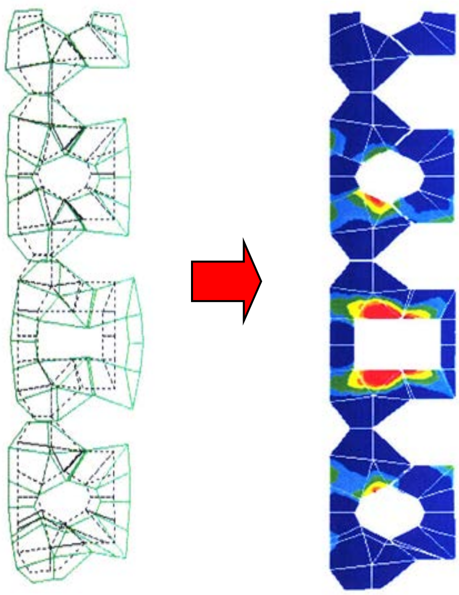


Fig. 4.13 - Horizontal cross section of an hallow gravity dam with deformation and stresses induced by expansive reactions

The diagnostic activity of Alkali-Silica Reaction should not rely on the presence of silica gel exudation from cracks in the concrete. In fact its presence reflects the fact that there is enough moisture to transport it to the surface. Furthermore, even if present, it can carbonate on exposure air, becoming almost indistinguishable from the normal calcium carbonate deposits that are common in water retaining structures subjected to seepage. The color of these deposits depends on their composition, mainly white if formed of calcite and transparent or slightly darker in the presence of silica. However darkening can be caused by biological or organic inclusions. The silica gels can appear with different consistency, from viscous to waxy, rubbery or hard texture. In presence of alkali-carbonate reaction or sulphate attacks other types of deposits are usually found as for example brucite (magnesium hydroxide) in the first case and gypsum, ettringite and thaumasite in the second one. Samples of all deposits should be collected for chemical analysis in the laboratory.

Also the so called “pop-outs”, that are sometimes reported as sign of Alkali-Silica Reaction, may be entirely absent. In fact they are usually caused by very fast and strongly reactive aggregates, as for example chert, near the concrete surface, situation that is not found everywhere. Furthermore, even if present, pop-outs can be also ascribed to other causes as freezing and thawing.

The evidence given by the visual inspections should be considered and evaluated to provide a preliminary assessment and to highlight areas requiring more exhaustive examination and analysis, both through sampling and laboratory testing and through field investigations.

4.3 Laboratory Investigations

The number of samples required for diagnosis and their distribution depend on the object of the investigations. An initially limited number of concrete cores, drilled where the damage is most evident, together with fragments and exudations, may be sufficient to investigate if the causes of expansive phenomena actually are the alkaline or sulphate reactions. Only on the basis of the first indications, a more detailed sampling and laboratory investigation program should be planned to establish the extent and severity of the problem.

Typical symptoms of Alkali-Silica Reactions are aggregate particles that are recognizably reactive or potentially reactive and evident signs of reactions in the particles, in the cementitious matrix and at the interfaces, with silica gels in cracks, voids or around the aggregate particle edges (dark rims) (Fig. 4.14, Fig. 4.15, Fig. 4.16 and Fig. 4.17). Brucite can be found in presence of Alkali-Carbonate Reactions while gypsum, ettringite or thaumasite are typical of sulphate attacks.

A petrographic examination with a polarized light microscope carried out by qualified and experienced petrographers on concrete thin sections taken from cores or fragments is the most reliable and rapid method to identify the cause of the expansive phenomena. The use of fluorescent dyes in the epoxy resins of thin sections and their study in the fluorescent light mode make the cracks and deposits within aggregates, along the interface and in the cementitious matrix, easier to be recognized. The amount of reaction products present is not to be taken as indication for the severity of concrete deterioration, that should be assessed separately, based on the extent of cracking (paragraph 4.5 on damage assessment). In dams with reactive phenomena at their initial stage cracks are significantly less frequent than in dams with well-developed reactions.

Fig. 4.14 - Dark rims surrounding reactive particles in a dam concrete core.

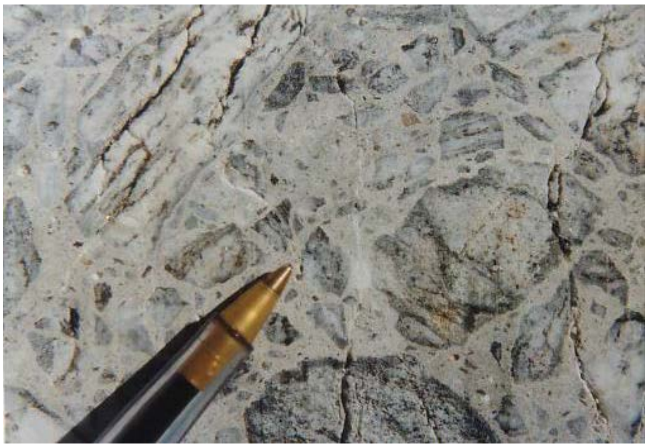


Fig. 4.15 - Cracks partially filled with silica gel in a dam concrete core.

Fig. 4.16 - Formation silica gel at the interface between aggregate particle and cementitious matrix.

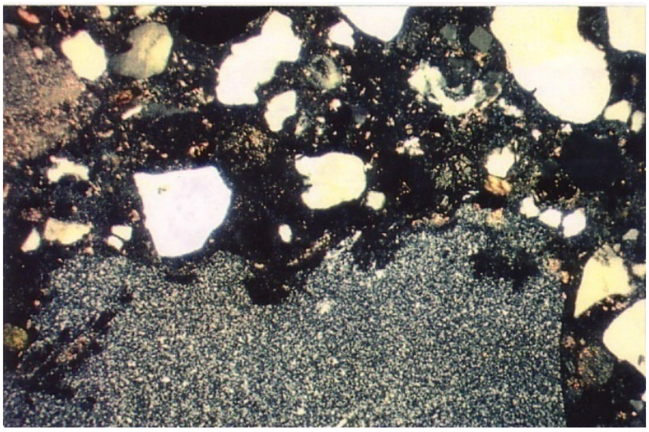


Fig. 4.17 - Flint particle with reacted boundaries (concrete thin section observed through a polarized light microscope using crossed polarizers).

In the case of Alkali-Carbonate Reaction a typical rim of calcite forms in the reacted dolomite particles, at the interface with the cementitious matrix, as a result of the dedolomitization process. A further external layer of micro-crystalline brucite is often visible (Fig. 4.18 and Fig. 4.19). Alkali-Carbonate is the hardest reaction to identify when investigating an affected structure because its by-product, brucite, is very difficult to find even by microscopic examination.

Some other laboratory investigations are needed to corroborate the initial diagnosis or to support it, by means of electronic microscopy and microanalysis techniques. In particular, Scanning Electron Microscope (SEM) observations with microanalysis by an Electron-Probe Micro-Analyzer (EPMA) and by X-ray (XRD), concrete chemical analyses (e.g. determination of soluble alkali content) can be very useful in order to verify the first diagnostic inputs from the petrographic examination, to clearly differentiate the Alkali-Silica Reaction from sulphate attack or other type of deterioration and to observe the damage in concrete at microscopical level.

Fig. 4.18 - Dark rims surrounding reactive dolomite particles in a dam concrete core.

Fig. 4.19 - Dolomite particle with a rim of calcite due to dedolomitization (concrete thin section observed through a polarized light microscope).

SEM analysis is especially useful when gel is scarcely found in concrete cracks but other laboratory tests (e.g. petrographic examination with a polarized light) evidenced the presence of reactive aggregates with reaction rims. It can be carried out both on concrete fracture surfaces and on polished sections, in order to identify the reaction products morphology and, if SEM is suitably equipped with by EPMA, also their chemical composition. In the case of Alkali-Silica Reaction the reaction product usually appears as a massive–textured amorphous gel or as a reticular aggregation of poorly crystalline gel (Fig. 4.20) or again as rosette-like crystals (Fig. 4.21). Its composition changes with time and varies widely depending on its location. Near the aggregate, the original gel spectrum usually shows large peaks due to silicon and alkali and only a very weak peak due to calcium. However, during its migration from the reacted particles into the cement paste, through the cracks, the gel comes in contact with the cement paste and loses part of Na and K, assuming approximately the same composition as the normal calcium silicate hydrate (C-S-H) of the cement paste.

Another method for detecting alkali-silica gel in concrete is the uranyl-acetate treatment procedure. With this petrographic tool, a freshly exposed concrete surface is sprayed with a solution of uranyl acetate, rinsed with water, and then viewed under ultraviolet light. Reacted particles and gel appear as bright yellow or green areas. However, the uranyl-acetate solution is slightly radioactive and concern has raised because the method can lead to false positive: some gels may not be detected and those that react to the uranyl solution are not always ASR gels (pozzolanic products for example). Another procedure for detecting gel in concrete is the Los Alamos staining method, which is used in the field as well as the laboratory. A solution of sodium cobaltinitrite is applied to a fresh concrete surface and viewed for yellow staining, which indicates gel containing potassium. A second reagent, rhodamine B, is added to provide contrast for the yellow stain and to identify calcium-rich ASR gel. However these methods must not be used alone to diagnose ASR.

Secondary Ettringite (Fig. 4.22 and Fig. 4.23) and Thaumasite crystals, gypsum plates, calcium and magnesium carbonates are other possible reaction products, depending on the type of expansive reaction involved (sulphate reaction or Alkali-Carbonate Reaction). Mapping of elements by EMPA on polished thin sections can provide additional useful information about the principal elements in concrete affected by expansion phenomena.

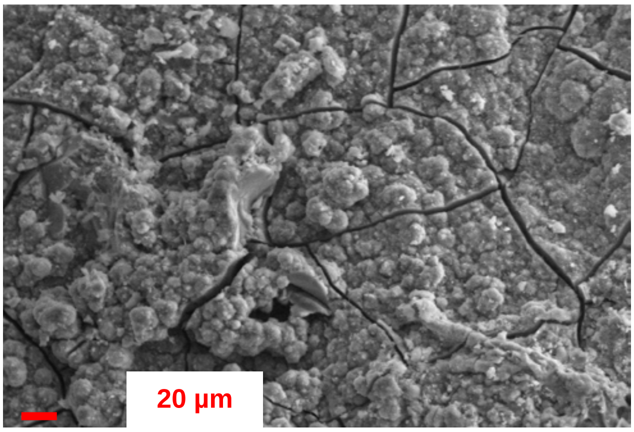


Fig. 4.20 - Massive–textured amorphous ASR gel

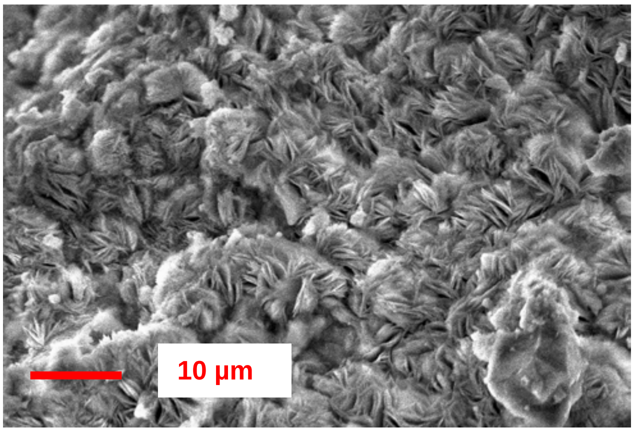


Fig. 4.21 - Rosette-like crystals in ASR gel

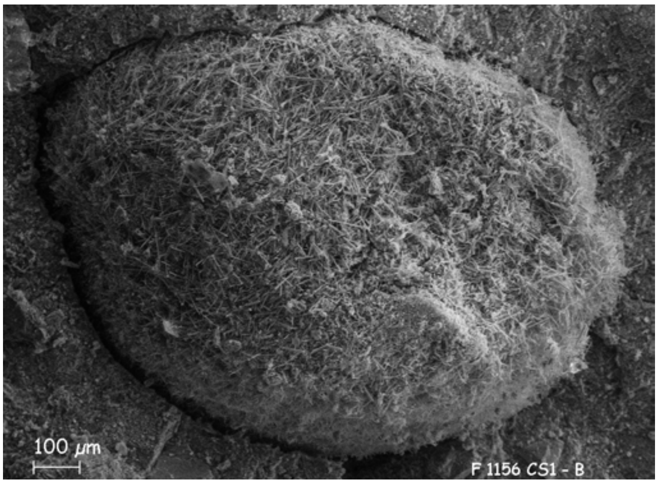


Fig. 4.22 - Massive–textured secondary ettringite crystals in a concrete pore

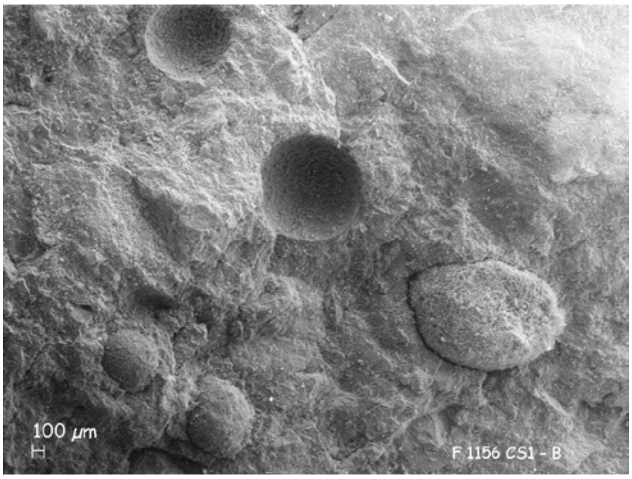


Fig. 4.23 – Concrete pores filled with secondary ettringite massive crystals

The XRD is a technique suitable to determine the crystalline phases present in concrete, mineralogical composition of the aggregate and crystalline reaction products, such as ettringite, thaumasite and brucite. However amorphous phases like those present in the Alkali-Silica Reaction gels cannot by identified by this technique as they result in very weak diffraction peaks.

Determination of soluble alkali contents in concrete may be useful in the diagnosis as well in the prognosis assessment of a concrete dam, as it provides information on the remaining effective alkali in the concrete. Several methods are available, based on different extraction methods: acid soluble, water soluble, hot or cold. However, different results are expected and therefore the extraction method is to be chosen depending on the specific concrete feature and on the aim of the analysis. For example the acid extraction method is able to extract considerable amount of alkalis from the aggregates and can considerably overestimate the soluble alkalis from the cement. Water soluble alkali provides indication of the remaining active available alkalis in the concrete and is therefore relevant and indicative not only for the diagnosis but also for the prognosis process.

Expansion on concrete cores taken from the dam may be used to confirm the diagnosis of aggregate reactivity (paragraph 4.6 - Evaluation of residual expansion rate of concrete). Standard reactivity tests on aggregates retrieved from quarry used for the dam construction or even extracted from concrete cores can further assist in the diagnosis process.

If the results of these preliminary analyses, carried out on a limited number of concrete cores or fragments, provide sufficient evidence of the presence of a specific reaction phenomenon in the concrete, then a more extensive laboratory program should be defined, as previously mentioned. It should be based on a larger number of tests, more representative of the concrete and widely distributed on the dam structure, not only for the complete characterization of concrete at the micro-level but also for a comprehensive investigation at the meso-levels (mechanical and physical proprieties: compressive and tensile strength, elastic modulus, creep, etc). This deeper investigation should be able to better identify the relevant factors that actually influence the ongoing reactions and the concrete swelling kinematics, as well as to obtain the suitable input parameters for the mathematical modeling of the material degradation as well as of the expansion at the macroscopic structural level (Chapter 6).

4.4 Damage assessment

Once AAR o Sulphate deterioration is established as the main or a contributive cause of the concrete deterioration in the dam, the degree of damage in the concrete at the micro-structural level should be assessed.

The crack pattern and the modulus of elasticity are often evaluated to have an understanding of current concrete condition (damage reached to-date) and the rate of expansion. However it should be kept I mind that crack formation may not be attributable solely to expansion phenomena.

The crack pattern can be considered both at the macroscopic and microscopic level. In the first case the individual surface crack widths are measured over a defined surface grid, suitably selected on the structure, and the total amount of cracking is quantified. For example, the method proposed by LCPC (Laboratoire Central de Ponts et Chaussées) [4.8] consists of measuring the widths of all cracks intersecting two perpendicular 1 m lines originating from the same point and their two diagonals 1.4 m long (Fig. 4.24). The total Crack Index (CI) is determined as a value in millimeters per meter.

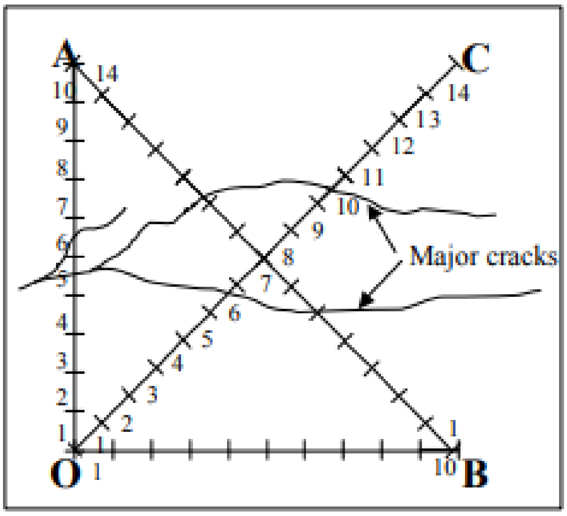


Fig. 4.24 - Determination of the crack-index along lines A-B, A-C, A-D and B-C: Side of the square = 1m [4.9]

The UK ISE (Institute of Structural Engineers) proposed a method for surface crack mapping based on measuring cracks widths along five parallel lines that are each 1 m long. Again, the total width of intersecting cracks along each line is summed and divided by the length over which they were measured [4.10]. In the 2010 Addendum, ISE stated that the crack summation procedures for estimating expansion to date work well in directions where there is little restraint from structural stress, reinforcement, or pre-stress. This suggests that in structures with higher restraint, like it may occur in concrete dams, this would not be the case.

The examinations are repeated at regular intervals and the results are compared over time, with a goal of establishing a rate of expansion progression.

Moreover, it should be recalled that for concrete in the field, the surface indications sometimes poorly correlate to the extent of expansion degradation within the concrete. Presence of surface phenomena affecting pH, such as leaching and carbonation, or reducing humidity during the drying periods, can lower the surface expansion development compared to the expansion conditions inside the concrete. The measurement of surface cracking may underestimate the true expansion attained in the core of the structure.

At the microscopic level, the Damage Rating Index (DRI) was initially developed in Canada as a method to determine the extent of internal damage in concrete affected by for the AAR in several dams and to compare the severity of AAR among structures. It was then implemented and worldwide used.

For the determination of DRI, polished sections of concrete cores taken from a structure are subjected to a petrographic examination, under a stereo-binocular microscope, in order to identify and count a number of features, indicative of AAR. These are, for example, the presence and distribution of reaction products, the existence of internal micro-cracking and the location of micro-cracking (within the aggregate or through the cement paste).

For concrete dams the investigated concrete area has to be increased, to get representative results: a regular grid on the polished section which includes about 200 grid squares 1 cm by 1 cm in size should be drawn, as shown in Fig. 4.25 [4.11].

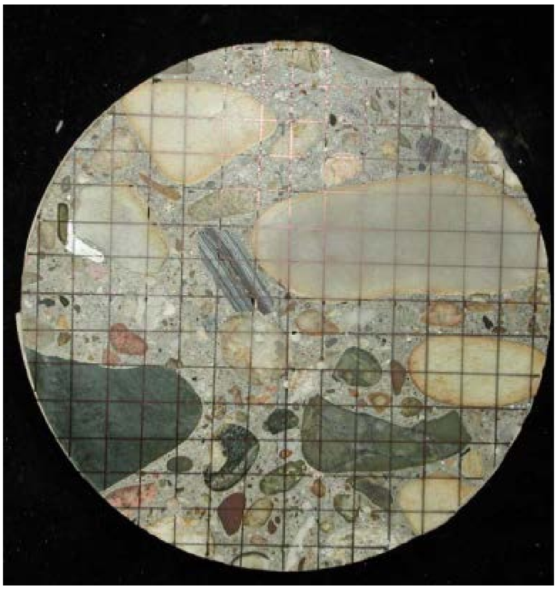


Fig. 4.25 - Surface of the polished core specimen for DRI evaluation [4.11]

To obtain the value of DRI, the weighting factors such as those of Table 4. 1 [4.12] have to be applied to the counted numbers of each damage feature, and the results normalized for an area of 100 cm^2.

Table 4. 1 - DRI features and weighting factors [4.12]

Damage feature measured	Factor
Coarse aggregate with cracks	0.25
Coarse aggregate with cracks & gel	2.0
Coarse aggregate with open cracks	4.0
Coarse aggregate debonded	3.0
Coarse aggregate with reaction rims	0.5
Cement paste with cracks	2.0
Cement paste with cracks and gel	4.0
Air voids with gel	0.5

An example of applying the DRI approach to cores taken from the intake and diversion sluiceway at Mactaquac generating station (Canada) is given in Fig. 4.26 [4.13]. It is interesting to note that, in this case, the damage is generally restricted to coarse aggregate with cracks and filled with gel. It is also important to know which concrete constituents are being damaged by AAR because this affects strength changes. Other examples of DRI for two USA dams (Center Hill [4.14] and Fontana [4.15] dams) are reported in Fig. 4.27 and Fig. 4.28 respectively. Unlike the case of Mactaquac, in the Center Hill concrete a predominance of cracks in the cement paste is observed, with a small amount of gel, only present in the coarse aggregate. Finally, the DRI representation of Fig. 4.28 (Fontana concrete) show a homogeneous distribution of cracks with gel in the coarse aggregate and in the cement paste. It is noted that all cores would show some damage and typically DRIs in the range of 10 to 25 would be considered non-reactive concrete. In ASR-affected concrete the DRIs tend to increase over time as more expansion and hence damage develops. Possible correlations between DRI and concrete expansions have been proposed, based on laboratory testing Fig. 4.29 [4.16], even if with a non-linear trend. DRI measures on dam concretes typically remain below 150, corresponding to expansion levels usually below 0.15-0.20%.

DRI weighting factors assigned to each defect may not universally apply to all types of reactive aggregates [4.17]. Site-specific criteria for severity ratings and weighting factors for AAR indications may need to be established, in accordance with the reactivity of the aggregate used on site. However, the DRI method can provide very useful relative information in identifying the progress of the expansive process (AAR or other alterations) that forms cracks when cores are extracted regularly from the concrete dam and examined preferably by the same petrographer, as there is currently no standard test procedure available.

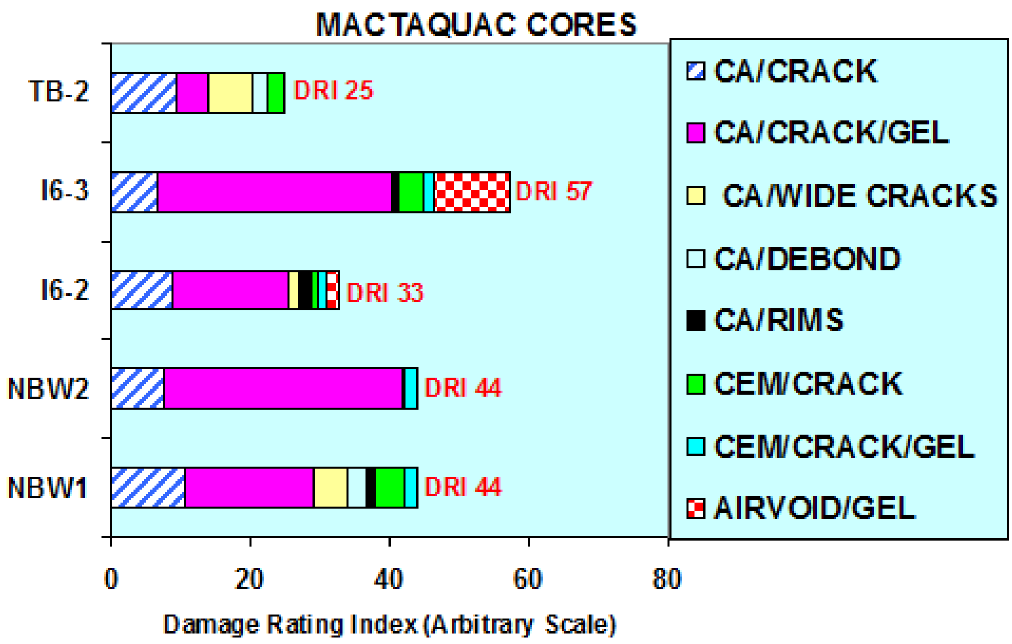


Fig. 4.26 - DRI values at Mactaquac cores taken from the intake and diversion sluiceway of Generating Station [4.13]]

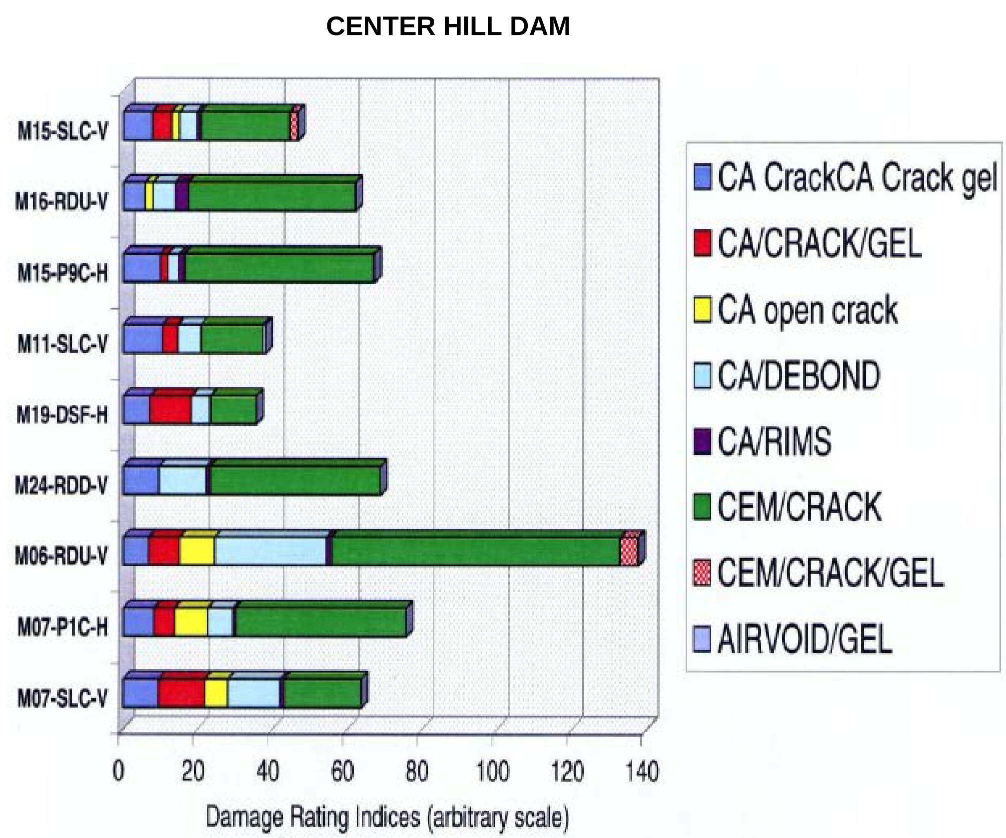


Fig. 4.27 - DRI values at cores taken from Center Hill dam [4.14]

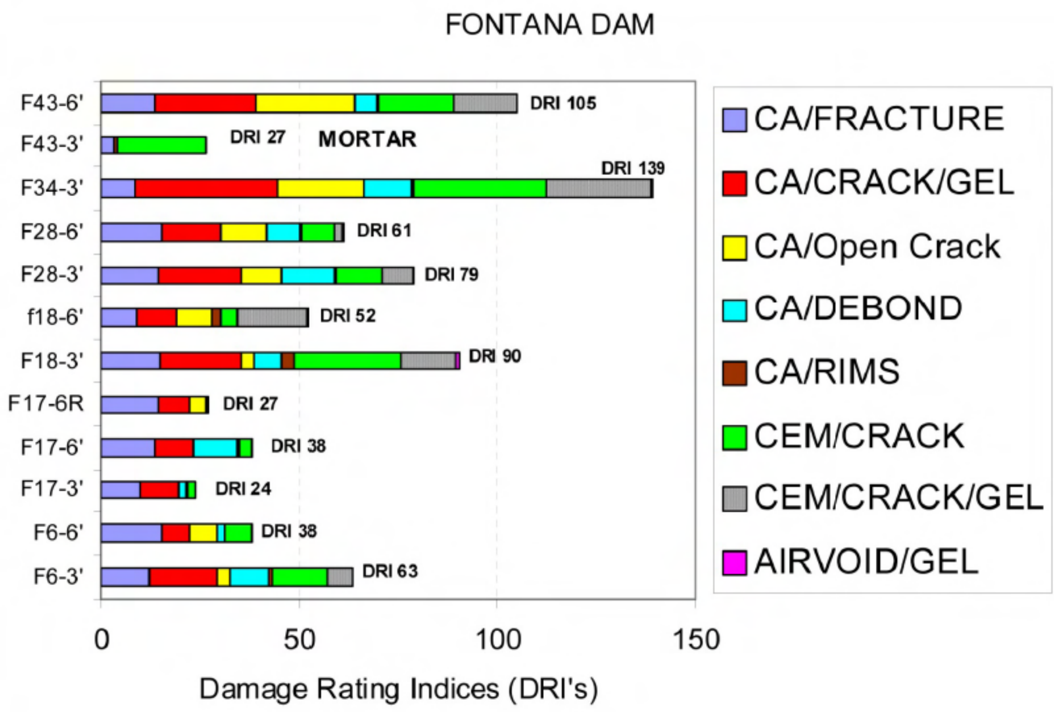


Fig. 4.28 - DRI values at cores taken from Fontana dam [4.15]

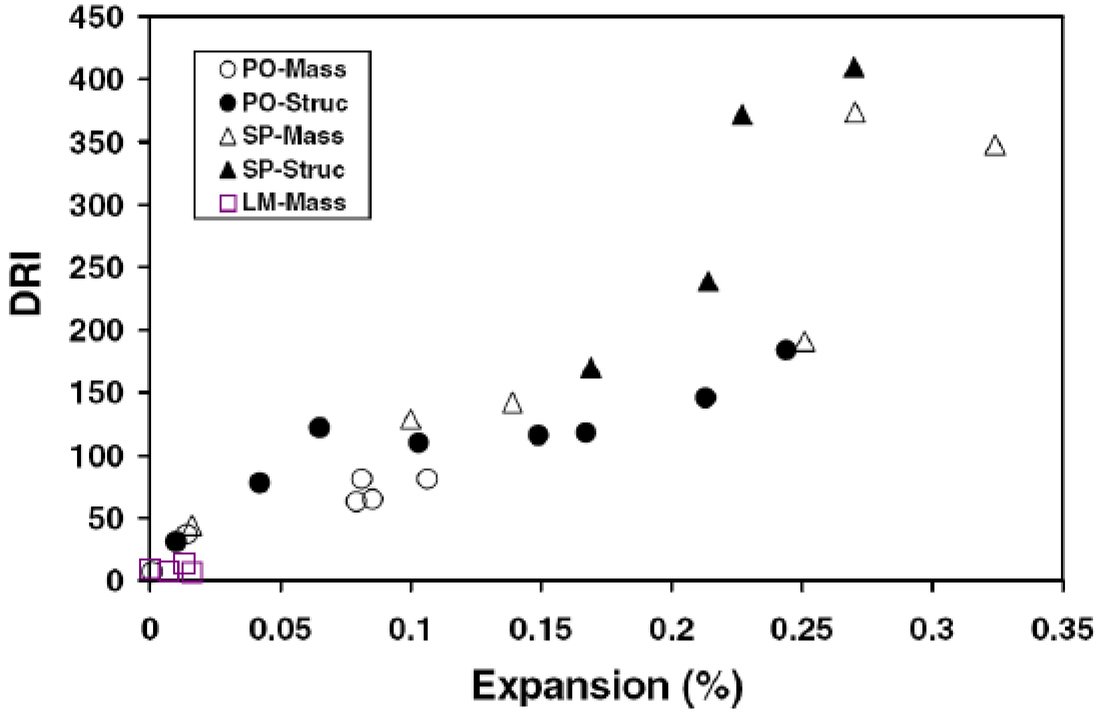


Fig. 4.29 - DRI and concrete expansion relationship from laboratory [4.16]

Several authors [4.18] suggested that severity of ASR damage can be evaluated by testing concrete cores for changes in modulus of elasticity and development of hysteresis (stiffness deterioration), through the so called Stiffness Damage Test (SDT). It was also adopted by ISE, the Institution of Structural Engineers in the UK [4.10]. The test consists in subjecting a set of concrete, cores extracted from the structure to be evaluated, to five cycles of uniaxial loading/unloading up to a maximum of about 40% of the concrete design strength. To quantify the degree of damage in the concrete, the following parameters have been proposed: (1) the energy dissipated (measurement of the surface area) during the first cycle (hysteresis loop) or during the last four cycles and (2) the accumulated plastic strain over the five load/unload cycles, this last being related to the closure of the existing cracks (Fig. 4.30). The above parameters were found to progressively increase with increasing internal micro-cracking and expansion in concrete affected by ASR. However, the energy dissipated parameter was found to best correlate with ASR expansions, and was suggested for assessing the expansion attained to date in concrete affected by ASR [4.19].

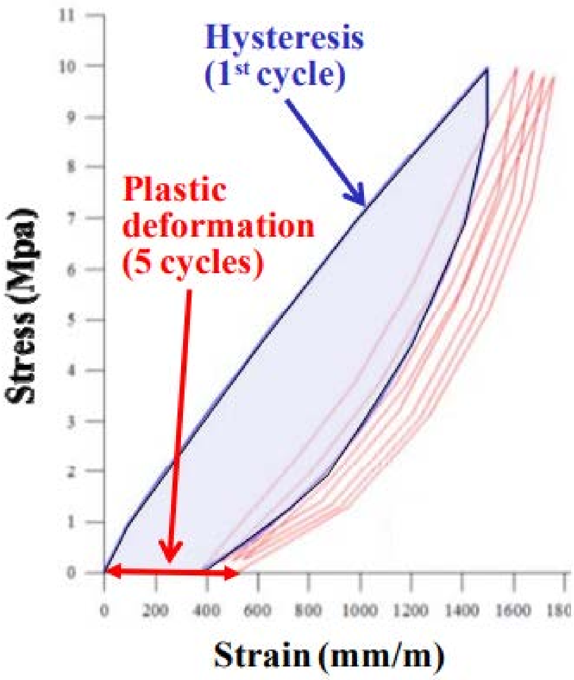


Fig. 4.30 - Main parameters used for evaluating concrete condition through SDT: hysteresis of the 1st loading/unloading cycle and plastic deformation over the 5 cycles. [4.19]

Changes of concrete mechanical properties in dams affected by expansive phenomena are also to be considered in the damage assessment process but they are often difficult to quantify, due to the variations of concrete quality in large dams and the lack of reference concrete properties. The effects of expansion on physical properties like compression and tensile strength, and modulus of elasticity have been discussed in Chapter 3.

4.5 Field investigations

The knowledge of the stress state within the dam body is frequently essential to confirm the reliability of the diagnosis, to verify the stress state calculated through numerical modeling or to update the stress calibration in Finite Element Analysis, particularly when the structural integrity or stability is controlled by stresses. To this aim field investigations can be carried out in different areas inside the concrete dam through suitable structural diagnostic techniques of a mechanical nature such as quasi-nondestructive experiments by flat jacks or pressure cells into the dam surface and in-depth in boreholes by overcoring tests, usually with extraction of specimens for the laboratory.

Such procedures have been found to be appropriate for estimating stress conditions at an instant in time. Measurements of stress changes over time have been attempted with stress cells. The use of stress cells that measure the stress directly may be appropriate if the stress cell is stable, without leaks if a fluid cell or creep or drift if mechanical. However, stress cells that measure strains and require conversion to stresses do not work in expanding concrete because the relationship between strain and stress is path dependent and unknown for the borehole where the gauge is installed. This issue was studied at Roanoke Dam [4.20].

In the flat jack method, a thin flat jack is introduced into a suitable opening or slot previously made into the concrete. With the aid of this device, pressure (compressive stress) is applied to the concrete. This causes a partial restoration of the initial displacement field, which at some point reach the previously measured values. The necessary pressure is be related to the compressive stress in the direction normal to the slot. Nevertheless, it is limited to a superficial measurement. The development of structural diagnosis based on flat-jack tests and inverse analysis has been also proposed to the engineering practice [4.21].

Overcoring is one of the most commonly used method that enables the determination of a comprehensive two- or three-dimensional stress tensor in one single point inside the concrete. It consists in measuring the strains that develop at the face or on the wall of a small diameter borehole (pilot hole) when this one is relieved from the surrounding in-situ stress field by overcoring. To this aim a suitable strain cell is put in place into the borehole through a special installation rod device. From the recorded strains the stresses can be calculated when the elastic parameters determined from biaxial or triaxial laboratory tests on the over-cored extracted concrete specimens are known. Fig. 4.31 shows the typical procedure using a stress cell to measure the strains on the borehole is used.

Overcoring field measurements have been frequently used, as comparison data, in order to verify the numerical modeling used to predict future structural behaviour (displacements, stresses and damage patterns) of an AAR-damaged dam [4.22] (paragraph 4.6 on residual expansion rate).

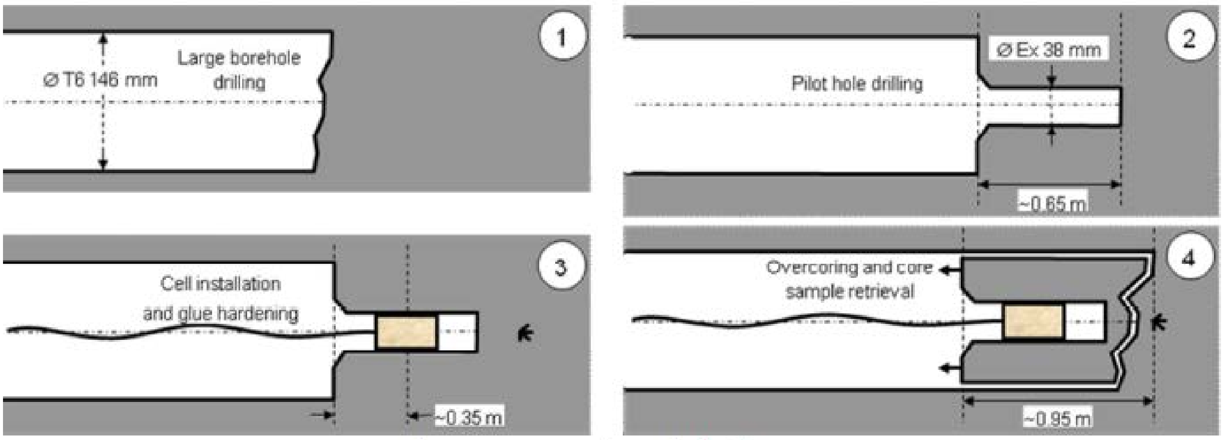


Fig. 4.31 -- Overcoring main steps [4.23]

Results from overcoring can provide a reasonable snapshot of the stress condition at the single point in time and should be interpreted based on an understanding of the temperature conditions and applied loading at the time of testing. To compare two overcoring tests performed in two nearby holes at different times, the tests should be done at about same month of the year to reduce the effect of seasonal temperature variation. Multiple overcoring tests are recommended within a single borehole to establish and decipher trends and account for potential variability in stress conditions that may be present at individual test sample location within the actual structure. Trends and stress conditions established from these multiple tests can then be compared with similarly performed test completed at a later time to evaluate the relative change in stress condition over that time period [4.24].

Evaluation of concrete stress by means of overcoring techniques requires the assumption of ideal behaviour: continuous, homogeneous, isotropic, and linear-elastic. Unfortunately, these conditions are seldom met completely in massive concrete causing errors (accuracy). Even when ideal conditions are met, some scattering of the results always occurs (precision). According to literature studies on rocks a variability of 10-20% is expected even in ideal conditions and for stress magnitudes an absolute imprecision of at least 1-2 MPa, regardless of stress component or measured value, is to be considered [4.23] [4.24]. These scattering is consistent with the results obtained in Chickamauga Dam when managing AAR concrete expansion. Fig. 4.32 shows a comparison of stress values measured through the overcoring techniques at different elevation in comparison to computed values, in the calibration process of the adopted finite element models [4.25]. However, even with this variability, the results from overcoring have been found to be useful in calibration analyses since in many cases the potential variability of computed stresses may be much greater that the overcoring variability.

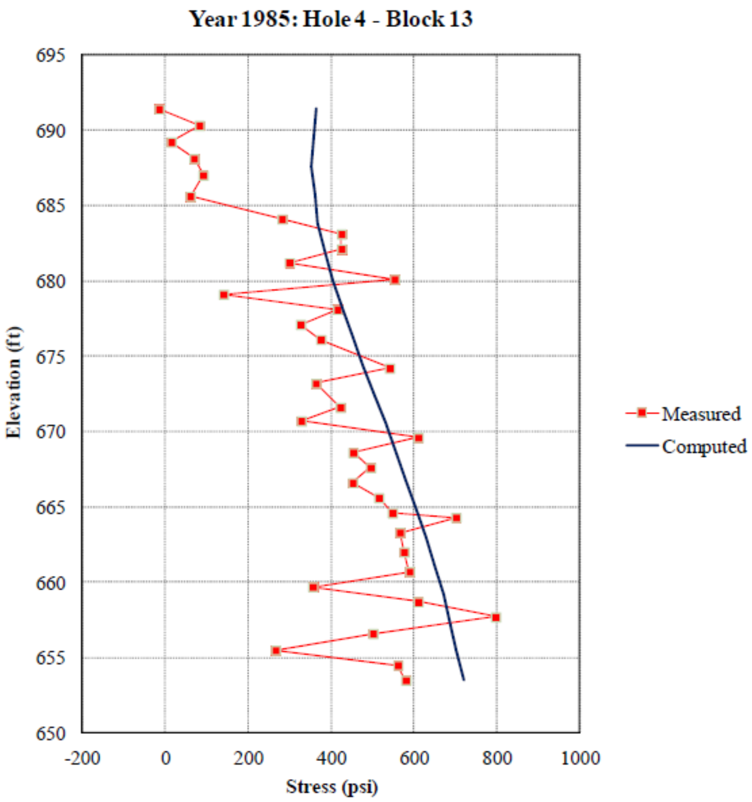


Fig. 4.32 – Scattering of overcoring measurements, compared to computed value at Chickamauga Dam [4.25]

One of the most widespread nondestructive technique to investigate the homogeneity of concrete and characterize its decay is the Ultrasound Pulse Velocity (UPV) technique. Its advantaged, compared to newer and more sophisticated non-linear elastic wave spectroscopy techniques, is that it can be implemented, in a relatively easy way, also in the field.

Seismic tomography is a geophysical method that can provide a high-resolution imaging of the concrete sections, in terms of seismic P-wave velocity (VP) and represents a useful tool for assessing concrete quality within large concrete dams, for detecting the profiles of internal cracks as well as for contributing to the safety control. In addition, the tomography results can be correlated with information from laboratory core analyses to produce cross sections of strength and modulus of elasticity values. This method may also be considered as a management tool to monitor the concrete dam behaviour along its working lifetime.

By sending sound waves through the dam body (between a pair of boreholes or between upstream and downstream faces) and recording them on the other side, at many locations and across many different angles, images can be produced that show, with high resolution details, the condition of the concrete within the structure.

Seismic tomography was performed at three cross sections of Seminoe dam (USA), to provide information on the spatial progression of concrete deterioration caused by the alkali aggregate reaction. The measured P-wave velocity plots (Fig. 4.33 downstream face. The concrete deterioration has progressed downward from the dam crest and inward from the downstream face above [4.26].

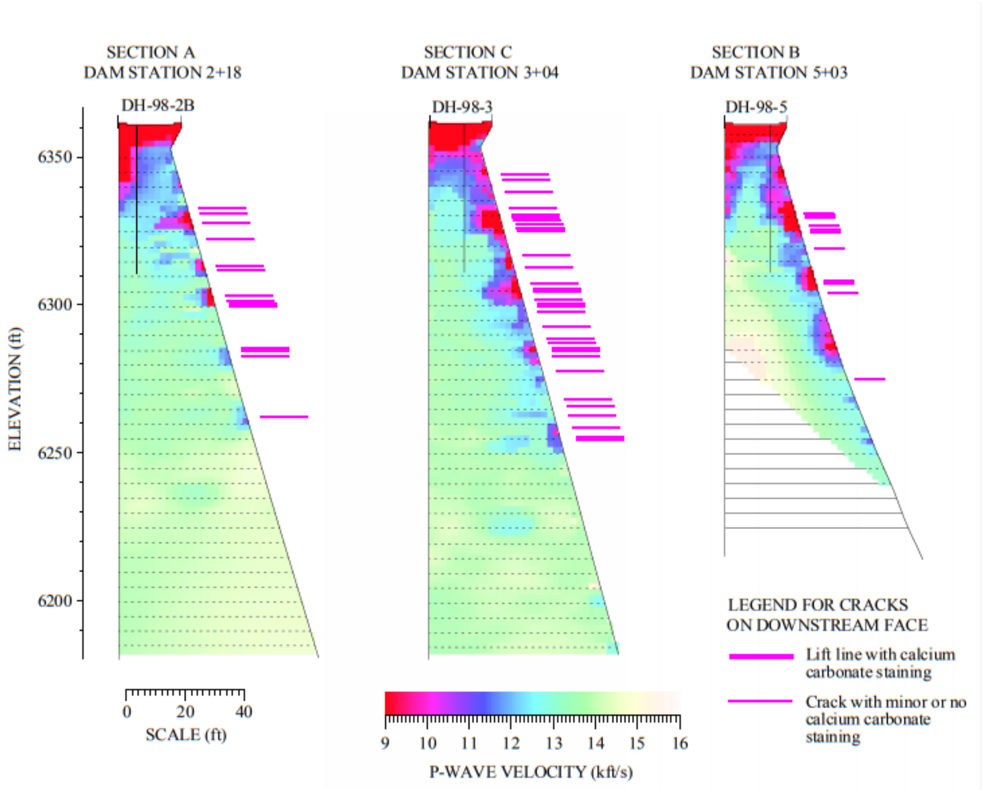


Fig. 4.33 - P-wave velocity tomograms for the three cross sections at Seminoe Dam [4.26].

4.6 Evaluation of residual expansion rate of concrete

In order to understand the future behaviour of concrete undergoing AAR expansion phenomena, it is important to estimate the total concrete expansion, i.e. the two components of expansion separately: first, the expansion that has occurred up to the time of investigation (current concrete expansion); and second, the potential for further expansion (residual concrete expansion). The aim is to reach a comprehensive explanation of the way in which the concrete dam behaves under the expansion phenomenon such as AAR, to evaluate its safety conditions, optimize the maintenance, to choose the most appropriate solution and to optimize the chosen intervention.

The knowledge of the current expansion is usually obtained through numerical analyses specifically carried out to find the best fit between the relevant quantities measured on the dam (stresses and displacements, for example vertical displacements measured by leveling and horizontal displacements measured by collimation) and the same quantities computed by means of the model (Chapter 6 - Structural analysis to assess behaviour).

However, the evaluation of the residual concrete expansion of AAR is a very difficult task and, in practice, this information may reliably be obtained only from field long time monitoring.

To compensate this limitation, accelerated expansion tests have been used in the last decades, on concrete cores suitably taken from the structure and subjected to long-term testing (at least one year) in controlled environment (saturated air conditions with R.H. > 95% and temperature of 38°C). This testing method is considered the most realistic as concrete is tested with its own alkali content at humidity and temperature conditions that accelerate the reaction but do not greatly differ from usual structure conditions.

Number and location of core samples should be carefully selected, considering the heterogeneity of dam concrete, and core sizes properly related to the maximum aggregate size. Special care should also be taken to prevent any concrete drying after coring, in order to avoid neglecting AAR expansions. However high humidity conditions in long-term testing may promote alkali leaching from concrete specimen, leading to an underestimation of the expansion, and this problem should be dealt with in the best possible way. Different approaches are proposed by existing guidelines, for example by the French testing method LPC n° 44 [4.27] and by the core expansion test procedures adopted in the UK (Appendix A of ISE Technical Guidance [4.10]) or in Canada (Appendix B of the “Guide to the evaluation and management of concrete structures affected by AAR” [4.12]). Special storage conditions of cores wrapped and then sealed in polyethylene bags, inside small containers with a measured amount of water at the bottom are sometimes suggested, in order to minimize both drying and leaching effects.

Core weight changes against core expansions must be continuously monitored in order to distinguish the different phases that usually occur in the test development (Fig. 4.34). An initial phase characterized by a rapid swelling due to water uptake by concrete cores and partially by new reaction (conditioning “phase a”) is followed by a slow rate and continuous expansion for most of the testing time (“phase b”). This last is to be considered as the residual expansion rate of concrete and not as the total residual expansion, intended as the remaining total expansion that the concrete can potentially develop.

This residual expansion rate may, however, appear to be too optimistic by neglecting the expansion occurring during the first 8 weeks of testing, as prescribed by LPC n° 44 [4.28]. If “phase a” were restricted to only few weeks, the “phase b” would be different and composed by a non-linear expansion at the beginning of the “phase b”, followed by a linear expansion, with a constant rate [4.9]. Indeed, the starting point for the residual expansion measurements represents the most difficult and complex element of this test.

A third phase can also be observed, if finally the cores are dried up to the initial weight (irreversible expansion). In contrast to the S-shaped expansion curve observed in structures, the concrete cores lack an initiation period as AAR is already progressing. It should be noted that laboratory expansion evaluation is generally carried out in terms of levels of severity of uniaxial free expansions. Examples of test methodology to measure the residual expansion rate under different stress conditions and the suppression pressure under uniaxial and multiaxial conditions are documented in the literature [4.29].

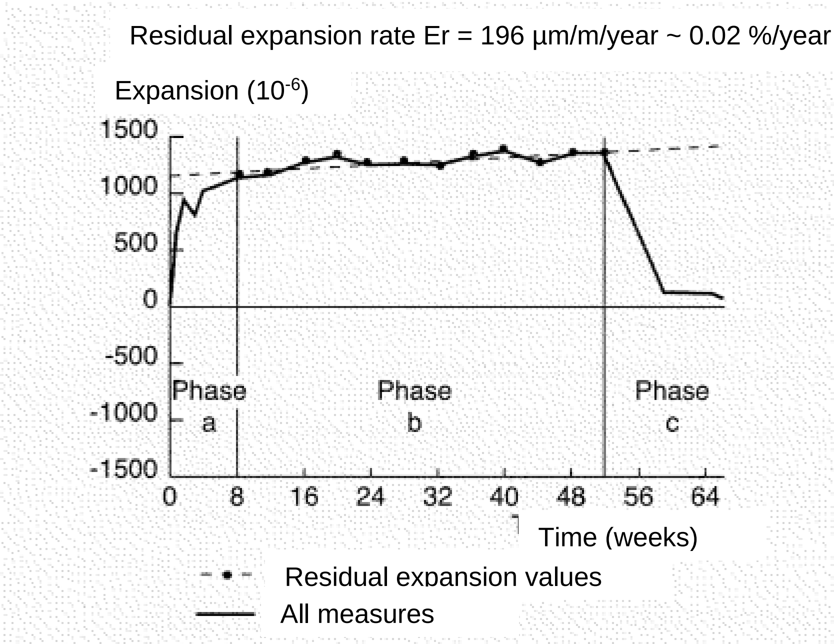


Fig. 4.34 - Different phases of expansion during core expansion measurements [4.27]

The more marked effect of alkali leaching in the laboratory test small cores, compared to the massive concrete in the field conditions, can lead to a flattening-off of the expansion curve and the consequent underestimate of the field residual expansion rate. The use of larger cores, e.g. 150 mm in diameter instead of 100 mm generally used, could contribute to reducing this effect [4.3].

Furthermore other uncertainties come from the stress state difference between laboratory and field, from the difference of ASR-gel nature between the long term reaction in the dam (calcium-silica gel) and the short term reaction in the accelerated tests (alkali-silica gel), from the difference in the temperature, which is normally lower in the field, from the limited duration of the test, from the relation between the core diameter and the maximum aggregate size, from the orientation of core in relation to the casting direction (anisotropy), etc. Besides, the presence of cracks in the cores can modify the value of the final measured expansion.

The influence of so many factors on the core expansion tests can explain why results obtained from this laboratory test should be used with caution and special care, even if literature cases are reported in which the measured expansion rates in the laboratory are consistent with those measured by an extensive instrumentation system in the field [4.29].

Expansion of concrete cores immerse in NaOH 1M solution at the temperature of 38°C is another testing method that can assist in the interpretation of the results of the main testing method (saturated air conditions at 38°C). However, due to the extremely highly basic solution used, it is only recommended for determining the “residual aggregate reactivity” in the concrete under examination, providing information on the expansion level that can be potentially reached in the presence of continuous alkali supply and whether the reaction and its effects are to be considered complete [4.30]. Furthermore, even in the presence of highly-reactive aggregates, the concrete might not expand in the existing dams without an high concentration of alkali hydroxides in the concrete pore solution.

Expansion guidelines have also been proposed in France for the evaluation of the residual expansion rate of concrete suffering Internal Sulphate Reaction (ISR) [4.31]. Concrete cores taken from structures are immerged in water at a temperature of 20°C and the free expansion is measured until stable volumetric conditions are reached. During the test, measures of weight, dynamic elastic modulus are suggested and, at the end of testing, a confirmation of the cause of expansion by means of Scanning Electron Microscope analysis is required, together with a compressive strength evaluation.

Expansion tests carried out on concrete of Fagilde dam (Portugal), affected by both ASR and ISR, showed that specimens exposed to water moisture (residual internal sulphate reactivity) experienced higher values than those stored in the alkali solution (residual alkali-silica reactivity). The specimens observed by SEM, after the completion of the tests, revealed the presence of ettringite in all specimens, occurring in the paste-aggregate interfaces, which was attributed to an internal sulphate reaction. These particular results have been explained by a depletion of the sources of reactive silica in concrete [4.32].

Several attempts have been made to correlate laboratory and field by special procedures to assess if laboratory tests on cores alone may lead to fairly reliable estimation of the residual concrete expansion rate in field conditions, in the absence of information coming from the in-situ monitoring of concrete deformation and movements. One of these is that proposed by Canadian researchers [4.3] [4.30] [4.33], based on the results of the two above mentioned laboratory expansion tests on cores (residual concrete expansivity rate in saturated conditions and residual aggregate reactivity in NaOH solution), on water-soluble alkalis content in the concrete obtained by hot water extraction (paragraph 4.3 – laboratory investigations) and on reasonable estimates of the humidity, temperature, and restraints (reinforcement, pre-stressing, post-tensioning, confinement, loading, etc.) in the field. Various coefficients are proposed to estimate the rate of ASR residual expansion in concrete structure in service (Paragraph 6.1.2).

A global methodology has also been proposed in order to evaluate the future progression of the concrete dam swelling due to AAR, by-passing the challenge of obtaining a realistic estimation of the residual expansion from concrete core expansion tests [4.21]. According to this proposal, the amplitude of future swelling is not based on results of laboratory expansion tests but assessed through an inverse analysis using a finite element numerical model whose reliability is verified by independent in-situ measurements, using the overcoring method. The variation of the vertical displacement measured on a point of the dam, combined with the laboratory determination of the chemical kinetic parameters representing the advancement of the reactive silica consumption for each aggregate size range in the concrete, allows, through the adopted inverse finite element method, other displacements of the dam and a realistic stresses field to be assessed. Based on this work, a prediction of the dam's displacements, damage and stress fields could be estimated for the coming years.

However, frequently the various laboratory-based estimates of future expansion progression remain subject to considerable uncertainty, and the only reliable information available for making decisions is the current rate of expansion if measured with accurate instrumentation. Consequently, in such cases no attempt is made to estimate the total residual expansion but it is assumed, conservatively, that the current rate will continue for the foreseeable future.

4.7 References

[4.1] RILEM State-of-the-art Report, "Guide to diagnosis and appraisal of AAR damage to concrete in structures – Part 1 Diagnosis (AAR 6.1), Springer, Godard, B., de Rooij, M., Wood, J.G.M. Eds., vol. 12, p. 87, 2013.

[4.2] BSA (Britisb Cement Association), "The diagnosis of Alkali-Silica Reaction, Report of a working party, 1992.

[4.3] Fournier,B., Sanchez, L., Beauchemin, S., "Outils d'investigation de la réactivité alcalis-granulats dans les infrastructures en béton", Final Report, Université Laval, Canada, August 2015.

[4.4] Amberg. F., Bremen, R., Brizzo N., "Rehabilitation of the Pian Telessio dam affected by AAR-reaction", Proc. 23th ICOLD Congress, Brasilia, May 2009, Q. 90 – R. 213.

[4.5] ITCOLD (Italian National Committee on Large Dams), “Il taglio con filo diamantato negli interventi di ripristino delle dighe in calcestruzzo soggette a rigonfiamento” ITCOLD Bulletin, 2008 (In Italian).

[4.6] Bastian O., “AAR and ISA at two dams in Switzerland”, Workshop on Chemical expansion of concrete in dams & hydro-electric projects”, ICOLD/SPANCOLD/International Journal of Hydropower & Dams Workshop, Granada, Spain, 18-19 October 2007.

[4.7] RILEM, “Recommended Specification: AAR-7.3 Preliminary international specification to minimize damage from Alkali-Aggregate Reactions in concrete – Part 3: Concrete dams and other hydro structures”, in RILEM Recommendations for the prevention of damage by Alkali-Aggregate Reactions in new concrete structures, RILEM State-of-the-art Report, Springer, Nixon, P.J. and Sims, I. Eds., vol. 17, pp. 155-168, 2016.

[4.8] LCPC (Laboratoire Central de Ponts et Chausséès), “Détermination de l’indice de fissuration d’un parament de béton”, Méthode d’essai LPC 47, 1997.

[4.9] Leeman, A. and Griffa M.,“Diagnosis of alkali-aggregate reaction in dams”, SFOE-Project SI/500863-01, EMPA Swiss Federal Laboratories for Material Science and Technology, 2013.

[4.10] ISE (Institution of Structural Engineers),“Structural effects of alkali-silica reaction. Technical guidance on the appraisal of existing structures”, 1992.

[4.11] USBR (U.S. Bureau of Reclamation), Report DSO-2014-03, “Seminoe dam – Assessment of concrete by quantitative methods – The petrographic Damage Rating Index”, 2014.

[4.12] CSA A864-00 (Canadian Standards Association), “Guide to the evaluation and management of concrete structures affected by Alkali-Aggregate Reaction”, 2000.

[4.13] Thompson, G.A., Steele, R.R., Coulson, D.M., “Management of concrete growth at the Mactaquac generating station”, Proc. 2nd International Conference on Alkali Aggregate Reactions in hydroelectric plants and dams, USCOLD, Chattanooga, Tennessee, October 1994.

[4.14] US Army Corps of Engineers – Nashville District, “Center Hill dam project”, ICOLD Workshop of chemical expansion on concrete in dams and hydro projects, Granada, 18-19 October 2007.

[4.15] Dodson, A., Curtis, D., “Fontana and Hiwassee Dams – Managing Alkali-Aggregate Reactions”, International. Workshop on Alkali Aggregate Reactions, 81th ICOLD Annual Meeting, 20 Aug 2013.

[4.16] Rivard, P. & Ballivy, G., “Assessment of the expansion related to alkali-silica reaction by the Damage Rating Index method”, Construction and Building Materials, Vol. 19, pp. 83-90. 2005.

[4.17] Smaoui, N., Bérubé, N.A., Fournier, B., Bissonnette, B., Durand, B., “Evaluation of the expansion attained to date by concrete affected by alkali-silica reaction. Part 1: Experimental study”, Canadian Journal of Civil Engineering, Vol. 31,2004, pp 826-845.

[4.18] Chrisp, T.M., Wood, J.G.M., Norris, P., “Towards quantification of microstructural damage in AAR deteriorated concrete.” Proc. International Conference on Recent developments on the fracture of concrete and rocks, Elsevier Applied Science, London, pp. 419-427, 1989.

[4.19] Smaoui, N., Bérubé, M.A., Fournier, B., Bissonnette, B., Durand, B., “Evaluation of the expansion attained to date by ASR-affected concrete – Part I: Experimental Study.” Canadian Journal of Civil Engineering, 31, 826-845, 2004.

[4.20] Reinicker, B.R., Abedzadeh, F., Charlwood, R.G., Cima, J.A.,”Evaluating in-situ stress behaviour in Alkali-Silica Reactive dams: Roanoke Rapids Dam”, Proc. 34th USSD Annual Meeting and Conference - Dams and extreme events — Reducing risk of aging infrastructure under extreme loading conditions, San Francisco, 7-8 April 2014.

[4.21] Garbowski, T., Maier, G., Novati, G., "Diagnosis of concrete dams by flat-jack tests and inverse analyses based on proper orthogonal decomposition", Journal of Mechanics of Materials and Structures Vol. 6, n. 1-4, January-June 2011.

[4.22] Nadim, C., Grimal, E., Lahaie, F., Bourdarot, E., Bigarré, P.," Full 3D stress field diagnosis at the heart of a large gravity dam", Proc. 6th International Conference on dam engineering, Lisbon, February 2011.

[4.23] Amadei, B., Stephansson, O., "Rock stress and its measurement", Chapman & Hall, London, 490 p., 1997.

[4.24] Reinecker, J., Stephansson, O., Zang, A., "Stress analysis from overcoring data - Guidelines: overcoring, World Stress Map Project, 2008 - http://dc-app3-14.gfz-potsdam.de/pub/guidelines/WSM_analysis_guideline_overcoring.pdf.

[4.25] Niznik, J., Curtis, D., "TVA's Chickamauga Lock – Managing Alkali-Aggregate Reactions", International. Workshop on Alkali Aggregate Reactions, 81th ICOLD Annual Meeting, 20 Aug 2013.

[4.26] USBR (U.S. Bureau of Reclamation), Safety Office Report DSO-02-03, "Seismic tomography of concrete structures Phase II: Hardware and software developments – Phase III: field tests at Seminoe dam", September 2002.

[4.27] LCPC (Laboratoire Central de Ponts et Chausséès), "Alcali-réaction du béton. Essai d'expansion résiduelle sur béton durci", Projet de méthode d'essai LPC 44, 1997.

[4.28] Multon, S., Barin, F. X., Godard, B., Toutlemonde, F.," Estimation of the residual expansion of concrete affected by Alkali Silica Reaction", Journal Of Materials In Civil Engineering, ASCE, January 2008.

[4.29] Lo, K.Y., Hafney, A. M.," Measurements of residual expansion rates resulting from Alkali-Aggregate Reaction in existing concrete dams", ACI Materials Journal, V. 96, n. 3, May-June 1999

[4.30] Fournier,B., Bérubé M.A., Folliard K., Thomas, M., "Report on the diagnosis, prognosis, and mitigation of Alkali-Silica Reaction (ASR) in transportation structures", Federal Highway Administration Report FHWA-HIF-09-004, January 2010.

[4.31] LCPC (Laboratoire Central de Ponts et Chausséès), "Réaction sulfatique interne au béton – Essais d'expansion résiduelle sur carottes de béton extraites de l'ouvrage ", Méthode d'essai LPC 67, 2009.

[4.32] Fernandes, I., Silva, S.A., Gomes J. P., De Castro, a. t., Noronha F., "Characterization of AAR in Fagilde dam", Proc. 13th International Concerence on AAR, Trondheim, Norway, 2008.

[4.33] Bérubé, M.A., Smaoui, N., Bissonnette, B., Fournier,B., "Outil d'évaluation et de gestion des ouvrages d'art affectés de réactions alcalis-silice (RAS)", Études et Recherches en Transport, Ministère des Transports du Québec, 2005.

5 PHYSICAL EFFECTS IN DAMS & HYDROELECTRIC PROJECTS

5.1 Expansive Reactions of Concern

By far the most frequent process leading to the expansion of concrete in dams is Alkali-Aggregate Reaction (AAR). Of the two main types of AAR, ASR (Alkali Silica Reaction) is involved in most of the cases of AAR, and ACR (Alkali-Carbonate Reaction) is reported in a few cases [5.1] [5.2] although this reaction classification may be in fact a form of ASR. (Chapter 2).

Several expansion cases have recently been attributed to aggregates containing iron sulphides as documented in detail in Chapter 3. Heat-induced Delayed Ettringite Formation should not occur if adequate precautions have already been taken to avoid thermal cracking. External Sulphate Attack will not give large scale expansions and is not a realistic possibility in dams other than perhaps foundations.

The magnitude and duration of the expansion can be a key factor in estimating the remaining service life of a dam and it is very difficult to estimate in the cases where expansion is continuing. The earlier notion that with AAR the alkali source is the cement is now realized often to be only part of the story; in many cases alkalis become available from certain aggregates with time. This will clearly affect the duration of the reactions and in many cases will cause the reaction effectively to continue indefinitely.

The vulnerabilities of dams to chemical expansion of concrete are clearly specific to the dam type. Details of most of the cases mentioned below may be found in proceedings of international conference as [5.3] [5.4] [5.5] [5.6] [5.7].

The observed patterns of behaviour are summarized again here to assist in identification and emphasize the importance of prevention of these adverse effects.

5.2 Commonly Observed Impacts

In this section only general comments are made regarding impacts on various dam types. However, when addressing project situations and the development of management plans, more specific assessments should be made including identification of potential failure modes (PFMs).

5.2.1 *Gravity Dams*

In gravity dams, the crest usually rises and moves upstream and the expansion effects have caused concern particularly at changes of geometry; for instance, at the change from a straight section to a smaller curved abutment section (e.g., Chambon, Fontana and Salanfe where cracks occurred), at junctions with intakes (e. g., Beauharnois, Rapides Farmers and La Tuque), or at spillway piers where large shear stresses and cracking may occur (e.g., Conniston, Hiwassee, Mactaquac, Sartigan and Temiscouata), or in some cases at abutments where unplanned loads develop (e.g., uplift at Chambon and San Esteban). Differential expansion due to varying reaction rates within the structure (e.g., American Falls, Center Hill, Chambon, Val de la Mare) may also cause internal cracking around the galleries and local loosening of horizontal joints with possible concerns for downstream sliding stability under either static or seismic loading. There are many cases of surface cracking, (eg., Illsee) some of which simply causes minor deterioration (e.g., Clanwilliam), but in others increases susceptibility to freeze-thaw damage (e.g., Dale, Paugan, Conniston).The horizontal expansions are clearly hindered by the structure and therefore an increase of compression stress is expected.

5.2.2 *Arch Dams*

In arch dams, there are several cases of substantial upstream and vertical deformations, some as much as 15 cm. These deformations and associated compressions may not be of direct concern in the main body of the dam, but can be quite relevant for the development of diagonal cracks on the downstream face parallel to dam/foundation contact (e.g. Sera, Seminoe). Such a structural crack may be of major concern for the shear strength and for the transmission of the trust forces to the abutments. Loss of joint shear strength can also be of concern in static and seismic conditions (e.g., Stewart Mountain, where anchors were installed, and Owyee), particularly at the shoulders of the arch dam. The buildup of compressive stress in the arch should not be so relevant thanks to the flexibility of the structure, but potentially could be of a certain concern in relation with a non-uniform distribution of the expansion over the dam thickness.

In some of these, the reaction is believed to have terminated (e.g., Stewart Mountain, Gene Wash, Copper Basin. In others, the expansion rates are slow and ongoing and concern focuses on possible long-term effects (e.g., Alto Ceira, Cahora Bassa, Kougha, Santa Luzia). Local problems may occur at the abutments or spillway openings if significant structural discontinuities exist (e.g., Kariba, Santeetlah), since these may cause structural cracking. Differential expansion and freeze-thaw damage may also be of some limited concern at a thin walled multiple arch dam (e.g., Stolsvatn, which is now being replaced).

5.2.3 *Buttress, Hollow Gravity and Ambursen Dams*

Expansion effects in more complex geometries may be more severe. Stress concentrations may occur in buttress, hollow gravity dams or Ambursen dams. In such cases, expansion induced differential deformations could possibly affect load carrying components or cause cracks and leakage, e. g., at the junction of the upstream slab and downstream walls at the crest in Ambursen dams (e.g., Pracana, Bloytjern and Tislei). In hollow gravity dams, like Poglia dam, due the particular geometry of the dam blocks, a very complex stress-strain may take place, inducing crack pattern in the internal wall (Poglia).

5.2.4 *Spillways*

Concrete movements have frequently caused problems with loss of spillway gate clearances with a resulting limitation of gate operation and impacts on flood handling capacity (e.g., Asejiri, Friant, Hirakud, Hunderfossen, Mactaquac). These deformations may be accompanied by inclined cracking in the piers supporting the gate guides which could affect their stability. Differential expansion of the embedded parts and host concrete may cause stressing of the gate guides and anchors which could cause separation and impact gate operations. Bridge decks frequently are affected by movements of the piers and abutments, the expansion joints close and spalling may occur affecting load bearing capacity.

5.2.5 *Intakes*

In intakes where the structure consists of large water passages with relatively thin piers between them and inside them (e.g., Mactaquac), there is concern about the effects of cross-flow deformations "racking" the piers, which if severe could affect stability. Differential movements in the upstream-downstream direction tends to cause systematic diagonal cracking in the piers (e.g., Mactaquac, Saunders) which has been found to be shallow in the known cases, but will eventually reduce their shear and vertical load carrying strength. Depending on the geometry of the intake and gate guides, concerns may arise with gate clearances. Concerns may also arise in gate guides with stressing of steel embedded parts and their anchors due to expansion of the attached concrete although these appear to have a great capacity to absorb such effects.

5.2.6 *Powerhouses*

The most frequent effect of chemical expansion in powerhouses is deformation or “ovalling” of the concrete surrounding the units. This affects turbine runner and generator air gap clearances and alignment (e.g., Beauharnois, Chickamauga, Kpong, Mactaquac, Moxoto, Saunders, Warsak), and distorts the head covers making removal and re-installation difficult (e.g., Mactaquac). In some cases, movements cause build-up of stresses in the penstocks, which may distort the stay rings (e.g., Mactaquac), leakages around the units or even cause rupture of the penstock (e.g., Warsak). Local stress buildup in embedded parts due to differential expansion can be of concern in discharge rings, draft tube liners, etc. Internal differential expansion in the substructure concrete may cause shear stresses in the draft tube piers (e.g., Mactaquac, Saunders) which can lead to diagonal cracking and concerns about reduction in load carrying capacity. Differential expansion movements frequently cause stressing of the superstructure frames and floor supports, changes to crane rail alignment and clearances.

5.3 Case Histories

5.3.1 *Objectives and Contents*

A set of summaries of a set of chemical expansion cases for various dam types and associated hydraulic structures from various geographic regions are presented to provide:

- Representative examples of a range of manifestations of expansive reactions in different structural configurations (dam types, geometries etc) that are significantly affecting safety and operations.
- Identification of issues and impacts resulting from the expansive reactions.
- Examples of the role of investigations, testing, monitoring, modelling.
- Methods of managing the effects, do nothing, monitor, interventions, etc.
- An assessment of the short and long term effectiveness of the management strategies adopted.

Information for the selected case histories are mainly from papers presented at the ICOLD AAR Workshops in 2007 [5.5] and in 2013 [5.6], with a range of issues and management. For these papers author’s name(s) and Workshop data are provided in the following format: [Author, ICOLD 2007 or ICOLD 1013]. For papers from other sources, author’s name(s) and data are similarly presented, together with a reference number for details (paragraph 5.4), in the following format: [Author, Date, [5.XX]].

Table 5. 1 presents a matrix of the selected case histories versus the structure type and main management approaches and interventions. This is intended to illustrate generally the most frequently used approaches.

Table 5. 1- Matrix of cases versus structure types and management technique

Case Number	Project Name	Reaction Type	Structure Type							Management/Intervention						
			Gravity	Arch Dam	Buttress	Spillway	Intake	Powerhous	Lock	Anchors	Grouting	Slot Cuts	Coating	Gate	Monitor/FE	Replace
1	Kariba	ASR		■		■								■	■	
2	Cahora Bassa	ASR		■		■								■	■	
3	Nalubaale	ASR		■		■		■		■	■				■	
4	Kleinplaas	ASR	■			■									■	
5	Kouga	ASR		■		■									■	
6	Matala	ASR		■		■				■	■			■		
7	Chambon	ASR	■			■				■	■	■	■	■	■	
8	Temple-sur-	ASR	■			■				■	■		■	■	■	
9	Bimont	DEF		■						■	■		■		■	
10	Poglia	ASR			■							■	■		■	
11	Piantelessio	ASR	■	■								■			■	
12	Pracana	ASR			■					■	■		■		■	
13	Alto Ciera	ASR		■												■
14	San Esteban	ISA	■	■		■					■	■	■		■	
15	Tavascan/T/G*	ISA	■			■					■		■			
16	Isola§	ASR	■	■											■	
17	Illsee§	ASR	■	■						■	■	■	■		■	
18	Salanfe	ASR	■							■	■	■			■	
19	Stolsvatn	ASR	■	■						■	■		■			■
20	Mactaquac	ASR	■			■	■	■		■	■	■	■	■	■	
21	Beauharnois	ASR	■				■	■		■	■	■			■	
22	Saunders	ASR	■			■	■					■			■	
23	Otto Holden	ASR	■			■		■							■	
24	Fontana	ASR	■			■				■	■	■			■	
25	Hiwassee	ASR	■			■						■		■	■	
26	Chickamauga+	ACR	■			■		■	■	■	■	■		■	■	
27	Terry	ASR				■			■	■	■				■	
28	Stewart Mtn	ASR		■						■	■				■	
29	Seminoe	ASR		■							■		■		■	
30	Friant	ASR	■			■								■	■	
31	Center Hill+	ACR	■			■						■			■	
32	Roanoke	ASR	■			■				■	■				■	
33	Santeetlah	ASR		■		■				■	■	■		■	■	
34	Moxoto	ASR	■			■		■		■	■	■		■	■	
35	Pedra	ASR	■			■						■		■	■	
36	Billings-Pedras	ASR	■			■		■				■		■	■	
37	Furnas	ASR				■								■	■	
	Tot. n. of cases		23	15	2	25	3	7	2	18	21	17	10	14	31	2

*Tavascan, Toran and Graus dams § ASR and ISA +ACR and ASR

5.3.2 *Noteworthy Features of Selected Cases*

The following cases have been selected in view of their potential value in guiding management decisions on other projects. Key features are highlighted in the short summaries in this section.

Noteworthy features include the particular nature of the expansive reaction (type, expansion rate, duration), local or general extent, the epidemiology and diagnosis of the reaction, significant effects on structures or equipment, innovative instrumentation, modelling or interventions, and lessons learned regarding the effectiveness of the management strategy. They are grouped by geographic region.

Africa.

1. **ASR Case: Kariba, (Arch Dam & Spillway) - Zimbabwe/Zambia** [Goguel & Gurukumba, ICOLD 2007; Gurukumba, ICOLD 2013; Noret, Clave, Gurukumba & Chibvura, 2017 [5.8]

Kariba dam is a 128 m high double curvature arch dam built on the Zambezi river in the late 1950s and provides 1,830MW power generation capacity. The dam body has been known to be subject to ASR since the late 1960s. The concrete swelling is affecting the spillway sluices geometry and has damaged the upstream stoplog roller path upper sections. Monitoring has shown a sustained moderate vertical concrete expansion rate varying between 23 to 47 µε/year but new data suggests the rate is slowing. The concrete swelling phenomenon at Kariba has been and is still being managed through continued inspection of the structure and its appurtenances, an extensive monitoring system and safety assessment studies, numerical modelling studies and remedial measures to mitigate the swelling effects. The remedial measures under consideration include repairs to the stoplog roller paths and provision of new upstream set of stoplogs together with an emergency gate.

2. **ASR Case: Cahora Bassa, (Arch Dam & Spillway) - HCB, Mozambique** [Tembe, Carvalho, Hattingh, Oosthuizen, 2014 [5.9]

The Cahora Bassa Dam is a 170m high double curvature concrete arch dam located on the Zambezi river in Mozambique and was completed in 1975. The installed capacity of the powerhouse is 2075 MW. Concrete swelling due to ASR has been observed since 1977 at a rate of approximately 25 to 40 µε/year depending on the instrument type. The swelling process of the concrete is still under way and no significant decrease in swelling is evident. There is an extensive instrumentation and monitoring system with proper redundancy in place which has been used not only to support detailed finite element modelling but also to facilitate proper behaviour analysis. The development of diagonal cracks on the upper third of the downstream face parallel to dam/foundation contact is evident. The swelling has also caused small deformations of the spillway gate supports which have affected gate clearances. A spillway radial gate rehabilitation project has been completed to address the clearances. The evolution of the process is being managed by updating the monitoring system including the geodetic survey system to facilitate monitoring of absolute displacements, the in-situ measurement of stresses and strains through overcoring and finite element models to follow this process.

3. **ASR Case: Nalubaale (Arch Gravity Dam & PH) - UEGCL, Uganda** [Brueckner, Ndugga, Meri, Mahsen, 2017 [5.10]

The Nalubaale Hydroelectric Power Station (formerly Owen Falls Dam) is located on the Nile river in Uganda. The project consists of a powerhouse with an installed capacity of 150MW and gravity intake and an arch gravity dam and was constructed in the early 1950's. Deterioration of the powerhouse structure was first noticed in 1964 in the form of hairline cracking in concrete elements. More pronounced cracking was observed in 1977. Stabilization of the spiral casings using post-tensioned anchors was implemented between 1989 and 1991 before ASR was established as the cause of the expansion and cracking. Extensive cracking has occurred in the powerhouse generator surround concrete and floor beams and affects the crane beam alignments. No significant impacts have been identified in the intake or main dam. The expansion strain rate has been estimated to be in the range 30 to 45 µε/year. A life extension program for the powerhouse is under development. The following are under considerations: structural modifications to control ASR expansion such as slot-cutting to accommodate expansion; installation of post-tensioned anchors to restrain the expansion; partial or full replacement of affected mass concrete; and, structural modifications to address the integrity of the structural frame such as installation of replacement downstream columns and strengthening of the downstream column in the region above the crane rail.

4. **ASR Case: Kleinplaas Dam (Gravity Dam) - DWA, South Africa** [Hattingh, ICOLD 2013]

Kleinplaas Dam is located on the Jonkershoek river in South Africa and consists of a 25.5 m high uncontrolled ogee concrete gravity spillway section in the river section and flanked by a rockfill embankment with a clay core on the left flank. The dam functions primarily as a balancing dam for a scheme supplying water for domestic use to the Cape Town metropolitan area. The dam was completed in 1982. Subsequently evidence of swelling was observed including pronounced cracking and opening of horizontal construction joints of the concrete spillway section. Monitoring of the swelling has been done since 1996 with 3D-crack gauges and since 2000 with a geodetic survey system on the crest. Since 2000 vertical swelling of between 20 and 42 με/year is evident. Total vertical strain of approximately 850 με was estimated in 2014. Some decrease in vertical swelling is evident since 2009. Continued monitoring of the swelling behaviour is currently taking place. A detailed structural analysis is planned and any required rehabilitation could follow.

5. **ASR Case: Kouga Dam (Arch Dam) - DWA, South Africa** [Hattingh, Oosthuizen, 2012 [5.11]

Kouga Dam is a 69-m double curvature concrete arch dam located on the Kougha river in South Africa and was completed in 1969. The dam has been extensively monitored during and after its completion in 1969. Initially the dam behaved as expected but since 1972, it became clear that some form of concrete swelling was occurring. In addition to this, some inelastic movement of the right flank followed a few years later that complicated the behaviour model of the structure even further. The static monitoring system of Kouga Dam showed continuing expansion since the early signs of swelling became evident. Initially only clinometers, pendulum clinometers and a geodetic network of targets on the downstream face of the dam wall were used. The static monitoring system was extended to include 3D-crack gauges, sliding micrometers and Trivecs. Subsequently real-time 3D crack gauges, a GPS system and a permanent ambient vibration system were added. Vertical swelling of around 25 με/year was initially observed but has reduced to less than 10 με/year since 2000. Horizontal swelling has however continued unabated. Sufficient redundancy in the monitoring system has added confidence to the interpretation of unusual behaviour of some of the blocks during low water levels (3D-crack gauges and ambient vibration results). Rehabilitation of Kouga Dam is currently under consideration – possibly building a replacement structure on the downstream side. Foundation investigations have been done and detailed numerical behavioural model have been compiled using the monitoring results.

6. **ASR Case: Matala Dam (Spillway) – PRODEL, Angola** [Casagran, Pradolin, Victor, 2016 [5.12]; Bouayad, 2017 [5.13]

The original concrete structures of the Matala Hydroelectric Development, built in the 1950's and located in Angola, are affected by AAR. The presence of this phenomenon was, and still is, manifested in deformations of spillway piers, original pedestals for flap gates and other structures. These deformations affected the operation of the flap gates (reducing spill capacity) and resulted in important/critical bridge roller bearing rotations. The concrete structures also exhibited severe cracking. Rehabilitation works were carried out to maintain the spillway discharge capacity and to restore the integrity of the structures. Works included construction of a new gated spillway (using portions of the existing spillway), pier pinning, bridge roller support rehab, concrete repairs. Feedstock from the original quarry was used and fly ash was added to the concrete mix.

Europe.

7. **ASR Case: Chambon (Arch-Gravity Dam & Spillway) - EDF, France** [Bourdarot, ICOLD 2007; Chulliat, Grimal, Bourdarot, Boutet, Taquet, 2012 [5.14]; Grimal, ICOLD 2013]

Chambon Dam is a large curved gravity dam completed in 1934 and affected by concrete swelling due to ASR. Key issues are related to (1) the high compressive stresses developing inside the dam and the abutments and the associated potential risk of thrust towards upstream in the curved zone and shear along concrete-rock interface at the spillway located in the left part, and, (2) displacements towards downstream in the central part and right bank and potential risk of tensions in the upstream face and shear at the foundation interface.

A first remedial works campaign was performed in the 1990-1997 period (new spillway construction, decommissioning of the old one, cracks grouting, PVC geomembrane installation on the upper 60 m of the upstream face to control uplift pressures, and 8 slot cuts). All these works proved their effectiveness by the recovery of a part of the irreversible displacements of the curved part and reducing compressive stresses in the upper dam part.

In 2007, the closure of the slots and a restart of the upstream movement of the curved part justified a reassessment of the dam mechanical behaviour and complementary investigations. Particular attention was paid to the vertical cracks located in the upper part which may create, under seismic events, potential unstable blocks. Taking into account the results from the last available numerical modelling, a new works campaign was carried out in 2013 and 2014 which included: seven slots-recutting with deepening of two (down to 40 m); installation of upstream to downstream post-tensioned anchors supplemented by a composite carbon-fiber net in order to reinforce the confinement of the upper part; and, replacement of the PVC membrane.

8. **ASR Case: Temple-sur-Lot (Gravity Spillway) - EDF, France** [Bourdarot, Sellier, Multon, Grimal, 2010 [5.15]]

The Temple-sur-Lot dam was built between 1948 and 1951 and has been in operation since 1951. It includes a gate-structure dam equipped with four double-leaf vertical lift gates (20 m wide, 10 m high).

In 1960, an inspection revealed the existence of cracks on the upstream part of spillway piers. In the following years, difficulties in the operation of the bulkhead gates led to several interventions on the mechanical parts embedded in the concrete structure. Reinforcement of the monitoring system provided a better description of the deformation of the piers and laboratory investigations revealed the existence of swelling phases inside the concrete.

During the period 1983-1988, extensive works were carried out on the piers, such as anchoring, and epoxy and polyurethane grouting. Recently (2002- 2003), the guidance system of the gates was modified in order to accommodate the concrete deformations. The effects of the swelling process on the structure include general rising of the piers (1mm / year) and tilting of the lateral piers towards the gates (0.9 mm /year). Analyses, including reinforcement of the monitoring, laboratory investigations and FE modelling, were carried out in order to explain these particularities. The lateral movements of the piers towards the gates can be mainly explained by the humidity gradients between the faces.

9. **DEF Case: Bimont (Arch Dam) - SCPARP, France** [Noret, Laliche, 2017 [5.16]]

Bimont Dam is located near Aix-en-Provence and was first brought into service in 1952, and became a part of the Société du Canal de Provence concession in 1963. The dam is a concrete, double-curved, arch-type structure measuring 86 m in height and 180 m in length along the crest. It consists of 15 cantilevers between two abutments, and its thickness ranges from 4 m at its crest to 13 m at its foot.

The dam developed a network of cracks in some cantilevers soon after construction. Their existence was initially put down to geology, but subsequent investigations showed that specific areas of concrete were affected by Delayed Ettringite Formation (DEF). This phenomenon brought about changes in the dam's equilibrium, resulting in the formation of superficial and internal cracks. A numerical model of the dam with elasto-plastic features ran in parallel with two special investigation campaigns. These measures allowed for a more in-depth understanding of how the network of cracks was formed, its spatial extent, and its probable future evolution, information which proved invaluable for the design of the dam renovation programme. The planned rehabilitation project will involve the treatment by cement grouting of the cracks and joints, waterproofing of the dam upstream face and the provision of vertical anchors to improve stability of the right abutment.

10. **ASR Case: Poglia (Hollow Gravity Dam + Wing Wall) - EDISON, Italy**, [Mazzà, Donghi, Marcello, A., Marcello C., 2008 [5.17], Donghi, Marcello C., Sainati, 2013 [5.18]

Poglia dam, located in the Lombardia Region (North of Italy) is a hollow gravity buttress structure (Marcello type) owned by Edison. Its aim is hydroelectric power generation. Since the 1970s the monitoring system started to show a slow drift in elevation of the different dam blocks and, less evident, in the upstream-downstream direction. For that reason the owner carried out investigations to reach a clear explanation of the causes which gave rise to the above said drift. The full reliability of measurements and the geotechnical survey excluded the presence of problems related to the stability of foundation and abutments. Hence, the presence of possible expansive phenomena in the concrete has been explored. To this aim an on-site and laboratory campaign was carried out and the presence of Alkali-Aggregate Reaction in the concrete has been ascertained. The phenomena are moderate but, due to the non-rectilinear longitudinal dam axis and to the particular geometry of the dam blocks, a relatively severe stress-strain state, additional to the one due to operational loads, has taken place in the dam body. An advanced three-dimensional, non-linear finite element model was applied to simulate the expansion AAR phenomena, and calibrated on the basis of the measurements recorded on the dam to analyse possible future scenarios, to investigate different hypothetical structural interventions and to give confidence during the work-in-progress phases. The final decision undertaken by the designer was to cut at the contraction joints in order to reduce the compression stresses in the blocks and in the right gravity shoulder. The works have been carried out during the spring 2005 in a very short stretch of time, in order to minimize the out of service time of the plant. The dam monitoring system, suitably improved during the works, has allowed to reach a comprehensive knowledge of the actual dam safety conditions.

The joints were cut by means of a "diamond wire", and carried out on all the contraction joints with the following aims:

- the recovery of the displacements of the right wing gravity shoulder;
- re-establishment of the behaviour of each block according to the original design scheme, reducing the high compressive stresses due to AAR.

The attainment of these aims has been estimated by means of a detailed finite element model and will be confirmed with the monitoring system installed on the dam, suitably integrated during the rehabilitation works.

A safety assessment of the main block against sliding has been carried out using a limit equilibrium finite element analysis. By making the extreme assumption of the dam sub-divided in several blocks by horizontal and vertical cracks, it has been found a safety coefficient greater than 2 times the maximum hydrostatic load.

11. **ASR Case: Piantelessio (Arch-Gravity Dam) - Iren S.p.A., Italy** [Amberg, Stucchi, Brizzo, 2013 [5.19]]

The Pian Telessio arch gravity dam is located in the Orco Valley (Piedmont, northern Italy) impounding a reservoir with a capacity of 24 million m^3 for a normal operating level at 1,917 m a.s.l. The dam is 80 m high, with a crest length of 515 m. The crest thickness is 5.7 m, while it increases towards the base where it reaches a maximum of 35 m. The dam is equipped with a peripheral joint which separates the dam body from the foundation slab (Pulvino).

The dam began operation in 1955 approximately after 5 years of construction. In a first period of roughly 20 years, the dam presented a regular and fully reversible behaviour, while since the second half of the 70s the dam is showing an upstream drift in a radial direction. The permanent displacement at crest level reached in 2008 almost 60 mm at the central pendulum. In addition to permanent displacement, horizontal cracks appeared in the upper inspection gallery, which are neither visible at upstream nor downstream faces. After excluding other causes for this observed behaviour, such as for example movements of valley flanks, it was assessed that the permanent dam deformations are caused by an ongoing alkali-aggregate reaction (AAR).

In order to avoid conditions with high compressive stress at the dam heel, a limitation in the minimum water level was adopted as a temporary measure in 2003. The operational limitation in the long term was not acceptable. Rehabilitation works consisting in the execution of 16 vertical slots by means of diamond wire were proposed and finally executed in 2008. The height of the main slots is 39 m in the central part, while it is limited to 31 m and 21 m towards both flanks. Between the main slots, secondary slots of 21 m height were realized.

Once the swelling stresses had been released, the slots were grouted in order to recover the arch effect required to support the pressure at full water reservoir. The rehabilitation works were carried out satisfactorily and since 2009 the dam is again under normal operation conditions.

12. **ASR Case: Pracana (Buttress Dam) - presently EDP, Portugal** [Camelo, ICOLD 2007; Batista, ICOLD 2013]

Pracana dam is a buttress concrete gravity dam with 12 buttresses, 3 massive blocks in each bank, a height of 60 m and crest length of 245 m. The project was constructed in the period 1948 to 1951 for a previous owner. Since 1952, several anomalies in the dam were detected and which continuously increased. In 1971 restrictions were applied to the reservoir level. In 1972 and 1973 various unsuccessful repair works were attempted.

In 1977 the ownership of the scheme was transferred to EDP. In 1978 the reservoir was emptied due to insufficient spillway capacity; progressive deterioration of the dam; and low safety factors. The dam exhibited: intensive cracking on the upstream face; important cracks on downstream face in the transition between the web and the head of the buttresses; significant vertical cracks in the webs near the foundation; and, cracks along horizontal construction joints. This was accompanied by seepage through horizontal cracks (concrete lift joints) with excessive carbonation. Large displacements were measured by geodetic methods and analysis showed non-reversible displacements.

Mineralogical and petrographic analysis of aggregates and cement paste identified gel formation and ASR. The concrete, also suffered from insufficient fines and high w/c ratio, which led to high capillarity porosity and open "channels" along the interface with the aggregates.

Sliding along horizontal cracks was identified as a critical safety scenario.

In 1985, evaluations concluded that the expansion phenomenon in the concrete was considered the main cause of the dam deterioration. It was also suggested that this expansion only developed in the presence of infiltrated reservoir water and stability conditions should be acceptable if uplift effects into concrete cracks could be avoided. The integrity of the dam's concrete could be improved by crack treatment and grouting. A global foundation treatment should be undertaken and a careful dam monitoring program should be set up.

The dam rehabilitation program was executed in the period 1988 to 1992 and included foundation struts between buttress webs, an upstream foundation plinth, foundation treatment, concrete regeneration including cement and epoxy grouting, installation of an upstream watertight system and improvement of the monitoring system.

The upstream watertight system consisted of a PVC membrane with a HDPE geogrid for drainage. The main aims of this system were to prevent contact between the concrete and the reservoir water to limit the future potential ASR, and to limit uplift in the cracks and thereby improve the stability.

The reservoir was refilled in 1992. Reported results of observation to 2007 showed evidence that the expansion phenomenon in the concrete was still present, but significantly attenuated. The upstream membrane was suggested to be effective both in terms of limiting water access to the concrete and thereby limiting restarting of ASR expansion effects and also improving stability by limiting uplift pressures in cracks. Continuous monitoring of the expansion phenomenon evolution was recommended.

13. **ASR Case: Alto Ciera (Arch Dam-replaced) - EDP, Portugal** [Camelo, ICOLD 2007, Batista, ICOLD 2013]

Alto Ciera was a 36 m high, 85 m crest length concrete arch dam built in 1949 with ASR and a fairly high expansion rate in the range of 120 με/year with no sign of slowing. It has been subject to progressive displacements, radial upstream and vertical upwards and cracking of dam's body.

The dam was subjected to comprehensive investigations which showed great heterogeneity of the swelling process with estimated potential expansions up to 650 με, intensive cracking special concentrated in the shoulders and on the crest; relatively great depth of the cracks in a thin arch structure; and relative intensive leakage through the body dam. It was not possible to estimate the stresses in the structure as well as to predict the future behaviour of the dam. The rehabilitation of the dam was considered to be very difficult and expensive. In 2013 a new dam was been completed 200 m downstream and the old dam is under demolition.

14. **ISA Case: San Esteban (Gravity-Arch Dam) - Endesa, Spain** [Gil, ICOLD 2007]

San Estaban dam is a 115 m high gravity-arch dam with crest length of 295 m with adjacent 4,500 m^3/s spillway completed in 1955 on the river Sil in Spain.

The following structural anomalies were identified: wet lift joints in the high zone of the dam; misalignments of blocks at the crest; irreversible movements of joints in upper zones; wet cracks and lift joints in the upper gallery; cracking of the faces of two blocks; and progressive elevation and upstream movement of the crest.

An investigation program with drilling and permeability tests showed honeycombed concrete, deficient lift joints, connections between drill holes and shallow cracks and obtaining in-situ stresses by the overcoring method. Materials testing showed granite, diabase, gneiss and shale (presence of pyrite) aggregates with fractured structures, altered crystals and the presence of reactive quartz and ettringite and expansive gel products. The concrete had high porosity, normal mechanical characteristics, cement with a high content in CaO. A finite-element model was used for diagnosing expansion and confirmation of the phenomenon.

A remedial program of structural rehabilitation and waterproofing was carried out that included injection of epoxy resin in defective lift joints and facing of the upstream face with 12,000 m^2 of dual-reinforced resin laminate installed in 9 layers with prior cleaning (5 mm thick) and able to adjust to concrete deformations with a modulus and tensile strength capable of supporting concrete cracks of 3 mm and resistant to internal pressures due to drawing down reservoir.

The results of the actions included: filled lift joints and a rehabilitated structure, elimination of seepage but continuation of the expansion phenomenon.

Future actions include: visual inspection, numerical cartography, monitoring including in-situ stresses, additional rehabilitation based on the use of mathematical models.

15. **ISA Cases: Tavascan (Gravity+Spillway w radial gates), Toran (Gravity+Spillway) & Graus (Gravity+Spillway) – Endesa, Spain** [Río, Espinós, ICOLD 2007]

Tavascan dam is located on the river Noguera de Cardós in the Pyrenees, Spain. It is a gravity dam with radial gated spillway with maximum height: of 31 m commissioned in 1965. The aggregate is a metamorphic slate. Anomalous behaviours, including cracking in the spillway piers, were identified in 1981.

Toran Pont De Rei dam is located on the river Toran, in the Pyrenees, Spain. It is a gravity dam with a fixed lip spillway with maximum height: of 36 m. It is constructed with a metamorphic slate aggregate and was commissioned in 1960. Anomalous behaviours included cracking in the inspection gallery.

Graus Dam is located on the river Tavascan in the Pyrenees, Spain. It is a gravity dam with a fixed lip spillway and with a maximum height of 29 m. It was constructed with a metamorphic slate aggregate and entered service in 1971. Anomalous behaviours, including cracking in the downstream face, were detected in 1986.

Reports for all three dams indicate that movement began about 10 years after construction and some form of waterproofing was carried out in year 33, 1998. The rate of vertical movement increased slowly over at least 40 years but no changes in the rate have been observed since waterproofing was carried out. However, the rate of upstream movement stabilized in 12 to 18 years and reduced in 19 to 32 years and after waterproofing (year 33), has stabilized up to now. It is not clear if this reduction is because of waterproofing or would it have decrease naturally anyway.

16. **ASR/ISA Case: Isola (Arch-Gravity Dam) - AXPO, Switzerland** [Otto, ICOLD 2007]

Isola dam is a concrete arch-gravity with a maximum height of 45 m, with a representative height of 30 m, and crest length of 290 m and located 1,600 m above mean sea level. This is a wide dam with crest length/height ratio of 9.7. It was completed in 1960. The mean concrete temperature is reported to be 7 °C.

Pendulum readings indicate an upstream crest displacement starting 20 years after construction and reaching 35 mm in 2005 after 45 years. Vertical rise of the crest reached a maximum of nearly 5 mm in the same period which corresponds to an accumulated vertical strain of approximately 200 micro-strain in the upper part and 50 micro-strain in the lower part. Cracking was detected in 1978 in the central downstream face and near the abutments in 1986. A crack was detected in the upstream face in 1989. In addition, cracks are apparent in the upper gallery. A horizontal compressive stress of 6 MPa has been measured.

It appears that the rate of development of expansion and associated effects, such as cracking, is slow at this dam, possibly due to the low mean annual temperature. It is suggested that most of the expansion occurs in the summer months. Laboratory tests are reported to suggest that the accumulated expansion strain could increase substantially above the present values. To our knowledge, so far, no interventions have been planned.

17. **ASR/ISA Case: Illsee (Gravity & Arch Dam) - AXPO, Switzerland** [Otto ICOLD 2007; Leroy, Amberg, ICOLD 2013]

Illsee dam is a concrete arch-gravity with a maximum height of 25 m, crest length of 270 m and located 2,360 m above mean sea level. It was built in 1926 and heightened in 1943. The horizontal layout includes two straight sections and a curved arch dam at one end.

The arch crest has been recorded to displace upstream by 45 mm in the sixty year period 1946 to 2005. The straight section near the junction with the arch section has been recorded to displace downstream by 30 mm in the same sixty year period. Cracking is visible on the downstream face at the junction between the center straight section and the arch section. The accumulated expansion strain has been calculated to be in the range 400 to 500 με in the 60 year period. Cracking is reported to have evolved in the arch section initially as horizontal cracks on the downstream face, and then horizontal cracking on the upstream face followed by opening of joints. The mean concrete temperature is reported to be 5 °C.

An impervious membrane was added on the upstream dam face in 1997 in a first phase of interventions, and slot cutting and reinforcement with anchorages were added in 2012-13.

The owner concluded in 2007 that the rate of expansion of 10 to 30 με /year is increasing with time after decades with swelling observed mainly in summer. Remedial measures (membranes on the upstream face) have shown no effect on the rate of swelling after 10 years. It is noted that AAR and sulphate reactions happen at low temperatures (5° to 7° C). This dam is still in the initial phase of swelling even after 40 to 60 years of operation and the swelling potential has been suggested to be much larger than the state to date.

18. **ASR Case: Salanfe (Gravity Dam) - ALPIQ, Switzerland** [Leroy, Amberg, ICOLD 2013]

Salanfe Dam is a gravity dam built in 1953 and now owned by ALPIQ. The dam is 616 m long, 52 m high with a crest width of 5 m. It has 42 blocks which are placed in 4 straight sections with one sharp change in alignment (concave downstream) in plan.

Vertical expansion has been measured for the last 15 years at a strain rate of 45 με/year. The geodetic network results show a significant upstream movement of the dam with a differential movement occurring at the change in alignment. Extensive surface cracking has occurred on the downstream face in the region of the corner and in the access gallery.

A program of slot cutting to reduce the stress concentrations was planned and 3 slot cuttings performed during the autumn 2012. The campaign was aimed at verifying the results of the numerical model and assessing the methodology and the performance of the work to be performed. It was decided to perform one of the cuts along a vertical joint and two others in the middle of a block. The slot cutting in the vertical joint caused slivers of concrete to fall into the slot preventing closure. The numerical model indicated an almost full closure of the 15 mm wide slot. Measurements showed a closure about halfway. Therefore, it was decided to increase the number of slot cuts to 22 and cut 20 m deep and to reduce the width of them to 11 mm. All remaining slot cuts were to be executed in the middle of blocks.

In 2013 is was too early to draw definitive conclusions regarding the global behaviour of the dam but some preliminary results indicated that although most of the slot cuts closed approximately by 8 mm, some of them completely closed after a few days. This was interpreted to be a demonstration again the high heterogeneity of the ASR affected zones within the concrete mass.

It was concluded that: slot cutting seems to be an efficient mean to reduce the compressive stresses due to concrete swelling induced by ASR; the numerical modelling of the behaviour of the dam affected by ASR before and after rehabilitation offers the opportunity to adjust the procedure and the strategy of rehabilitation; it is also essential to increase the monitoring efforts of the dam before performing slot cuttings: adapted and improved equipment are necessary in order to follow precisely the behaviour of the structure during and after relaxation of the stresses; and simple procedures and installations are possible for performing slot cuttings. Slot cutting can then be executed efficiently.

19. **ASR Case: Stolsvatn (Multiple Arch-replaced) - E-CO Energi, Norway** [Gunleiksrud, 2008 [5.20]

Stolsvatn dam was a multiple cylindrical arch concrete dam with vertical upstream faces and pairs of concrete buttresses built in 1948 on the Drammen river in Norway. The dam consisted of 13 arches in reinforced concrete, was 18 m at its highest, with total crown length of 520 m. The buttresses suffered from extensive surface cracking with concern about this allowing corrosion of the reinforcement. Epoxy coatings were applied to the external surfaces of the buttresses and additional anchors installed. It was found that AAR was present causing continual expansion of the concrete with surface cracking that facilitated freeze-thaw damage.

For a long time local residents expressed concerns regarding the safety of the dam despite numerous repairs. The dam was decommissioned in 2009 and replaced by a new embankment dam downstream.

North America / Canada & USA.

20. **ASR Case: Mactaquac, (Gravity Intake, Spillway & PH) - NB Power, Canada** [Curtis, ICOLD 2007; Fletcher, Curtis, ICOLD 2013]

Mactaquac GS is a 670 MW hydroelectric project consisting of a zoned embankment dam and a powerhouse with a concrete gravity intake and spillways completed in 1968. In this case ASR has developed and is continuing at a rapid rate in all concrete structures. Diamond wire saw cut slots were first developed at this project in 1988 and have been used to: control the deformations and cracking of the intake and other gravity sections; maintain clearances of spillway gates; maintain turbine runner and generator air gap clearances; reduce vertical misalignment of turbine shafts and stress build up in wicket gates. Extensive post-tensioned anchors were installed in the intake to secure stability during and after initial slot cutting. A comprehensive high precision instrumentation system was installed. A special purpose finite element concrete growth modelling program was developed to use the instrumentation data to assess existing conditions and plan and monitor effectiveness of interventions and estimate future service life. In December 2016, NB Power recommended maintaining the station to its intended lifespan and is proceeding with remediation plans on that basis.

21. **ASR Case: Beauharnois (PH and Gravity Abutment) - Hydro Quebec, Canada** [Gocevski, ICOLD 2013]

The plant has 38 generating units in three connected powerhouses with a total installed capacity of 1,903 MW. Construction of the facility began in 1930 and was completed in 1961. The effects of concrete expansion were first attributed to ASR at Beauharnois in the late 1960s. The major rehabilitation actions to date include:

- 1970 – 1971: Stability improvement – Strengthening (pinning to the foundation) of the right gravity dam by Post tensioning cables plus grouting of cracks;
- 1972 – 1975: Slot-Cutting by overlapping drill holes between the gravity dam and the water intake structure; and between the gravity dam and the administration building;
- 1980 – 1981: Slot-Cutting by overlapping drill holes between the units 35 and 36;
- 1995 – 1996: Controlled Separation: Powerhouse1 - water intake structure (slabs only);
- 2003: Controlled Separation: Powerhouse3 - water intake structure (slabs only).
- Observations regarding the major interventions:
- In 1970-71 strengthening of the Right Gravity Dam by post-tensioning cables and grouting of cracks was performed. HQ previously reported that longitudinal expansion increased after the interventions at the Right Gravity Dam causing more cracking of the walls of units A, B and 1. The expansion at the middle of the dam increased.
- It was also reported previously that the slot cut that was made by overlapping 4-inch diameter drill holes between the approximately 115 m-long concrete Right Gravity Dam and the SW corner of the powerhouse in 1972 to 1975 to relieve the expansive thrust on the powerhouse structure slot closed suddenly and shook the powerhouse and that stay vanes in Units A+B were damaged and the gates experienced operational difficulties.
- It was also reported that when a 4-inch overlapping drill hole slot cut was made in 1981 between units 35 and 36 that this affected the alignment of the adjacent units and caused interruption of production of unit 36 and the gate for unit 37 jammed.

The plant continues to operate effectively with ongoing monitoring and maintenance.

22. **ASR Case: R.H. Saunders (Gravity Dam, PH) - Ontario Power Generation, Canada** [Ho, Eastman, Adeghi, 1995 [5.21]; Eastman, Ho, Adeghe, 1998 [5.22]

The R.H. Saunders Generating Station is a 16 unit, 912 MW hydroelectric project spanning the St. Lawrence river and was constructed in the period 1954-58.

Equipment operating problems were first detected in 1972 when one of the generating units tripped due to distorted generator rotor-stator air gaps. Other issues include: deformation of throat ring clearances and turbine runners scrubbing the throat rings; misalignment of generator-turbine components, cracking of support beams; opening of construction joints in the water passages and generator deck; and cracking and dislocation of governor supporting walls.

A program of slot cutting was implemented starting in 1993 to relieve stresses and stabilize the oval shaped deformations of the throat ring and air gaps. The slots were designed to also provide accommodation for future slot cuts.

OPG reported in 1995 that three 15 mm slots were successfully established in 1993 and 1994 and the equipment and structural responses were favourable. Concrete slot cutting was viewed as a better alternative to the conventional grinding of throat rings, the modification of equipment or eventual throat replacement. Consequently, slots were cut between all other units in the period 1995 and 1996. A total of 16 slots were cut in the powerhouse concrete. The slots provided stress reduction in the concrete around the throat rings, reduced and stabilized the oval-shaped throat ring and rotating equipment openings. Slot cuts between units improved runner and generator air gap clearances and provided allowance for future concrete growth.

From 2012 to 2014, a detailed condition assessment and evaluation of the generating units was carried out. Clearance reduction, discharge ring deformation (ovalling) and further structural damage due to continued concrete growth have been reported, especially in the past couple of years. Slot monitoring instruments indicate 9 out of the 16 slots have closed completely and others are about 90% closed. Longitudinal compressive stresses have been re-established, and discharge ring have deformed.

Based on the condition assessment, the OPG plans to recut the slots, starting 2017, as part of the ASR maintenance program to ensure reliable operation for the future.

23. **ASR Case Otto Holden (Powerhouse and Spillway) – Canada** [Khoral, Hafez, Zhao, Hong, 2016 [5.23]]

Otto Holden Generating Station is an 8-unit 243 MW Power plant located on the Ottawa river in Ontario, Canada and completed in 1952. The east and west gravity dams, powerhouse, sluice gates and log sluices have been affected by ASR. Fly ash was used as partial replacement for Portland cement in the mass concrete that was used in the gravity sections. The structural instrumentation measures concrete growth rates in the range of 15 to 50 $\mu\varepsilon$/year.

Since the early 1970s, the station has experienced a number of structural and operational problems due to mass concrete deformation. Some of the major problems included: reduction of runner clearances to the extent that several runners were scrubbing the head cover and bottom ring seals; misalignment of the turbine/generator assembly; seizing of the wicket gates in their bushings; misalignment of the crane rails; severe cracking of carious structural components in the powerhouse; and high tensile stresses in stay vanes.

Unlike the sister plant, R.H. Saunders GS, where slot cutting was needed as one of the measures to manage AAR effects on the rotating equipment, slots were not needed at Otto Holden because of the absence of significant compressive stresses in the concrete structure. There is also little evidence of significant ovaling taking place around the rotating equipment. As a result of AAR-induced concrete movements and accompanied unit misalignment, a program of overhauls and realignment was performed from 2000 to 2008. During the overhauls, the units were modified to reduce the effects of AAR and to facilitate future realignments. A key component of the program is the modification of pivot ring so that it can be adjusted to accommodate induced movements.

In 2014, a detailed condition assessment was carried out to evaluate the current condition of the units. As a result of the condition assessment, a decision was made to overhaul the units to mitigate the unit alignment problems and extend the operating life of the generating station.

The planned overhauls will include disconnection of the top speed ring from the concrete embedded anchor bars and the pit liner so that the offset between the top and bottom speed rings can be reduced. This will improve unit alignments and reduce the frequency of unit overhauls.

24. **ASR Case: Fontana (Gravity Dam & Spillway) - TVA, USA** [Dodson & Curtis, ICOLD 2013]

Fontana dam was completed in the early 1940s and is subject to ongoing ASR. By 1970 a significant crack had developed in the curved section of the dam. TVA drilled overlapping boreholes to develop a wide slot at that location in the dam to de-stress the cracked area and limit further development of the cracks. This slot closed fairly soon afterwards. In the 1990s TVA installed vertical post-tensioned anchors to improve lift joint shear strength for stability, and installed a new wide slot using the diamond wire saw cutting technique. This slot cut is being used to assist managing the ASR induced safety impacts on the dam. The build-up of loads in the anchors is monitored using load cells. The anchors are de-stressed periodically as necessary although TVA reported in 2013 that one of the anchors had failed due to overstress. Monitoring has subsequently been increased. The dam continues to perform safely.

25. **ASR Case: Hiwassee (Gravity Dam & Spillway) - TVA, USA** [Dodson & Curtis, ICOLD 2013]

The dam consists of a 307-foot high gravity type concrete structure, a central spillway with seven radial gated bays, four sluices, and a 2-unit hydroelectric plant completed in 1940.

Evidence of growth discovered as early as 1939, strain meters in test cylinders indicated growth shortly after the concrete hardened. Spillway gate binding in the 1960s from mass concrete pushing into spillway. Significant cracking was found in the late 1970s in the curved sections near both abutments. Initial investigations into alkali-silica reactivity were made in 1978.

Greater than 35 mm permanent vertical expansion and more than 6 mm upstream movement since 1940. High longitudinal stresses and significant structural cracking near the abutments. Spillway gate binding and gate anchorage problems. Vertical growth has caused unit misalignment problems in the powerhouse.

Slot cuts were installed at each end of the gravity section at the start of the curved sections and at each end of the spillway in 1993 and 1994. In 2004 anchors were installed to improve the stability of gate piers. In 2006 it was confirmed that there was an adequate amount of potentially reactive material to sustain future expansion. The recalibration of the finite element model in 2007 allowed a revision in the slot cutting plan with a reduce frequency of re-cuts. The finite element model is currently being used to assess the timing of future slot cutting at the dam (narrow slots at the spillway). The finite element model is also being used to assess the dam stability for updated PMF and seismic loading.

26. **ACR Case: Chickamauga, (Lock and PH) - TVA/USACE, USA** [Niznik, Curtis, ICOLD 2013]

Chickamauga Lock and Dam was commissioned in 1940. The lock is owned by TVA and operated by USACE. The lock chamber size is 18 m x 110 m with normal lift of 15 m. The evolution of the expansion issues, which have been referred to as ACR, included surface cracking developed in 1943 in the lock junction portion of the lock. Extensive pattern cracking was observed in 1955. Concrete growth was identified at the powerhouse and significant cracks were found in the upper river approach wall and supporting piers in 1964/5.

Interventions from 1965 to 2012 included: installation of vertical multi-strand post tension anchors in all the chamber blocks, the upper sill, and upper and lower approach blocks along with horizontal post tensioned bars at various locations; slot cutting of upper river approach walls, post tensioning with a combination of multi-strand post tension anchors and bars; replace lower miter gate anchorages; shear pins to reinforce discharge ports in lower river approach wall, cut slot and reinforce discharge ports; chemical and cement grout used to reduce leakage; structural modifications to lower gates/anchorage connections, replace quoin blocks and gate seals; and rework of floating mooring bit tracks.

Future activities include: lock dewatering and inspections every 3 years; annual diving inspections and alignment surveys; continuous monitoring of instrumentation; lock operations personnel training to identify any changes and instructed to immediately report it to the lockmaster; update of finite-element analyses every 3 years; and pursue funding to complete construction of new lock and decommission existing lock.

27. **ASR Case: Terry Lock and Dam, (Lock & Spillway) - USACE-LRD, USA** [Charlwood, Rutherford, Curtis, Winterr F., Winters L., Terry, 2005 [5.24]

The David D. Terry Project in Little Rock, Arkansas, is part of the McClellan-Kerr Arkansas River Navigation System and is owned and operated by the US Army Corps of Engineers. The project was completed in 1969 and consists of a 33 m wide by 183 m long navigation lock and gated spillway with eighteen 18 m wide by 8 m high radial gates. The concrete piers at the spillway are subject to Alkali-Silica Reactions (ASR) and related expansion and some are subject to severe cracking which has affected their stability.

Testing and analysis has confirmed the hypothesis that the cracking is caused by differential expansion between pier concrete lifts constructed with 35 MPa and 20 MPa concretes. Rates of expansion have been found to vary between piers.

Repairs were completed in 2004 and 2005 for the six most seriously affected piers. These repairs included grouting and installation of inclined anchors and borehole extensometers to monitor future expansion. The condition of the concrete appears acceptable, with inferred compressive strengths above the specification values. The additional stress in the post-tensioned trunnion anchors appeared to be small at the time of the repairs. The extent of expansion of the 35 MPa concrete and the associated strain build-up in the anchors is to be re-assessed periodically.

28. **ASR Case: Stewart Mountain (Arch Dam) - USBR, USA** [Fiedler, ICOLD 2013]

Stewart Mountain Dam is a concrete thin arch dam located in Arizona. The dam is 380 m long, 63 m high, and was built between 1928-30. The dam includes a 13,000 kilowatt (kW) hydroelectric generating unit.

Evidence of concrete deterioration was first noted in 1935 as cracks, expansions and deflections; ASR was confirmed in 1943. Major dam safety modifications were initiated in 1980s mostly driven by new loads, AAR was not specifically addressed in these modifications.

Even with evidence of AAR reaction in most core specimens, overall the compressive strength (average of 36 MPa), modulus of elasticity and Poisson's ratio have not been reduced substantially since 1946. Movement of the dam since 1968 has been less than potential indicated by lab measurements of expansions due to AAR. Potential for continued AAR still exists but is of low magnitude; 3 of 7 cores from 1977 exhibited expansions in 100 percent relative humidity.

Despite potentially reactive aggregate and relative humidity favourable to expansion; movements of the dam have essentially stabilized. It is concluded that production of reaction product has essentially stopped. Recent analyses found no significant issues identified as a result of ASR, the concrete in the interior of the dam is very strong (37 MPa).

ASR at Stewart Mountain Dam resulted in significant expansions of the concrete arch but strength was not significantly affected and the reaction has stopped. Modifications to dam were required for other structural issues not related directly to ASR (unbonded lift lines were largely attributed to construction practice). Movement between dam and power plant resulted in replacement of steel penstock and addition of expansion coupling.

29. **ASR Case: Seminoe (Arch Dam) - USBR, USA** [Fiedler, ICOLD 2013]

Seminoe Dam is a concrete arch dam located on the North Platte river in Wyoming. The dam was completed in 1939 with a structural height of 90 m and a crest length of 162 m. The dam is experiencing concrete expansion, cracking and deterioration due to Alkali-Silica Reaction (ASR) and freeze-thaw damage. The mass concrete did not include entrained air and total air content is about 1 percent by volume.

Seminoe Dam did not exhibit the deleterious effects of ASR until after 1950. Deterioration at the top of the dam was initially attributed to freeze-thaw damage. It is likely that a slow growth from ASR was causing surficial cracking of the concrete, allowing freeze thaw damage to mask the apparent cause from internal expansion. ASR attributed to alkali-reactive quartzite was not identified until the 1970s. Compressive strength of the upper concrete has deteriorated significantly.

Instrumentation measurements indicate the crest of the dam has deflected horizontally upstream about 18 cm and vertically upward about 5 cm since 1979. Deformation causes an upstream flexure of the cantilevers of the dam and results in horizontal and diagonal cracks on the downstream face of the dam.

Repairs to reduce further damage from freeze-thaw included: resurfacing of the dam roadway; epoxy sealing of the curb/gutters/sidewalk and the top of the parapet walls; enlarging selected expansion joints through the downstream parapet wall and replacement of the concrete in the top of the outlet and penstock intake structures.

Seismic Tomography tests were performed in 2000 at three cross-sections of the dam and measured P-wave velocities in the concrete. Plots show lowest values near the crest, on the upstream face above the reservoir level and along the downstream face.

A 2004 finite element model study showed reasonable comparison to stresses and deformations measured in upper portion of dam in 2003. ASR expansions in upper portion of arch results in significant increase of load into abutments which could result in block movement along joints.

The concrete damage in the upper portion of the dam is caused by a combination of ASR, freeze-thaw damage and a lack of confinement. The concrete is continuing to deteriorate and there are no indications that the situation will stabilize. Remedial action will likely be required in the near future. Additional concrete testing is being performed and will be the basis for revisiting risks and a dam safety decision.

30. **ASR Case: Friant (Gravity Dam & Spillway) - USBR, USA** [Fiedler, ICOLD 2013]

Friant Dam is a concrete gravity dam on the San Joaquin river in central California. It was built between 1937 and 1942. Friant dam was Reclamation’s first extensive use of pozzolan (pumicite) to reduce cement content and heat generation within concrete; mass concrete contained 20 percent pumicite by weight of cement. Within several years after construction, concrete in portions of the dam developed abnormal cracking. Examination of cores indicated that alkali-silica reactivity involving andesite particles had developed. A separate structural mix was used for the spillway concrete that didn’t include pozzolan and experienced significant ASR reaction. Worst impacts of ASR were at the dam crest parapets and curbs; blocks 367 and 42 adjacent to the spillway; the spillway piers, walls and gallery; and the river outlet works valve deck at the control house.

ASR appears to be most severe in concrete which contains high-alkali cement, does not contain pozzolan, and there is less confinement or restraint. These conditions are met for the concrete above elevation 170 m across the dam and structural concrete associated with the spillway. However, no stability issues were identified from 2004 analyses.

The spillway was originally regulated by three drum gates, which were separated laterally by concrete piers. As a result of expansion in the dam concrete, primarily at the outside ends of the two outside gates, spillway gates were replaced in 1997-98. The crest gates were designed to accommodate up to 15 cm of inward movement of the adjacent blocks and adjustable end seals were provided.

ASR damage and gate biding at Friant Dam is a function of concrete mix (whether or not pozzolan was used with high and low alkali cements) and lack of confinement at gate openings. Crest gates with adjustable seals have bought more time to deal with future expansions.

31. **ACR/ASR Case: Center Hill (Gravity Dam & Spillway) - USACE, USA** [Hull, ICOLD 2007; Henderson, ICOLD 2013]

Center Hill Dam was constructed by the U.S. Army Corps of Engineers in 1948. The lake has a dual purpose: electricity production and flood control. The structure is 79 m high, and composed of concrete and earth structures, with 8 spillway gates that are 15 m wide each. Center Hill Lake is one of four major flood control reservoirs for the Cumberland river.

From 1951 to the early 70's no problems were noted. In 1970 survey monuments were installed. In 1974 bridge expansion joints began closing, bridge support rockers began tilting and surface cracking was noted. In the 1980s, bridge joints at end spans were completely closed, more cracks appeared, and surveys show dam is increasing in elevation. In 1983 gate seals on end Tainter gates bind during operation. Multiple spillway openings were smaller than design dimension by over 2.5 cm. Petrographic examinations in 1983 indicated the potential for Alkali Carbonate Reaction (ACR) due to coarse aggregates. Overcoring stress analysis showed stresses in the longitudinal direction ranged from 0.7 MPa to 5 MPa. Compressive strength was 25 MPa and tensile strength was 2.75 MPa. Petrographic Evaluation in 2001 could not confirm ACR but found ASR gel in most of the cores.

15 mm slots were cut in the non-overflow gravity section at each end of the spillway in 2006 and 2007. Finite element stress analysis showed a 40% reduction in the stress concentrations in the end spillway openings as a result of slot cutting. The slots closed in 2007, approximately 1 year after original cut and were recut in 2009. In 2013 the slots remained open.

Future plans include: continue monitoring; maintain open monolith slot cuts; and update finite element analysis and contributing data.

32. **ASR Case: Roanoke Rapids (Gravity Dam & Spillway) - Dominion Energy, USA** [Reinicker, Cima, ICOLD 2013]

Roanoke Rapids Dam is a 930 m long, 22 m high concrete structure completed in 1955. It includes four distinct sections: a 177 m long curved south non-overflow section (SNOS); a powerhouse with four 26 MW generating units; a 323 m long gated section with 24 radial gates; and a 228 m long fixed crest overflow section.

In 1995-1996 gate binding occurred at the gates at the ends of the spillway. 6mm shaved at each gate to maintain clearances. In 1996-1999 powerhouse units were re-aligned and lift joint cracking was observed in the SNOS and the emergency spillway. In 2006-2008 studies confirmed ASR. In 2003 leakage was observed on the downstream face of the SNOS at the gallery entrance. An unusual feature of this case is that the SNOS is a curved gravity section with the upstream face being concave, that is, the opposite of an arch dam. The longitudinal ASR thrusts in the SNOS therefore caused a "bowing" action downstream which resulted in an underwater crack opening of 25 mm on the upstream face.

In 2010 the crack openings were grouted and mild steel dowels and re-stressable post-tensioned anchors were installed in the SNOS to maintain stability. Continued monitoring of tendon stresses and deformations, coupled with finite element modelling is being used to confirm stability as the ASR continues. The most recent periodic upstream crack underwater inspections confirmed no change in the grouted cracks. New overcoring stress data is being used to update the FEM stress calibration and re-evaluate stability. The possibility of slot cuts is being considered as a possible option but is not planned at this stage.

33. **ASR Case: Santeetlah (Arch Dam) - Brookfield Renewable Energy, USA** [Mochrie, ICOLD 2013]

Santeetlah is a concrete gravity and arch dam located on the Cheoah river in North Carolina and constructed in 1927 and originally owned by ALCOA. The structure is 320 m long; and 61 m high with a remote powerhouse with 40.4 MW licensed capacity. The dam consists of a right non-overflow gravity section, an integral intake, a right thrust block spillway, an arch section, a left thrust block spillway and a left non-overflow gravity section.

The structure has long been recognized to have used high alkali cement and reactive aggregates which led to AAR. Cracking of surface concrete has been observed throughout its history with upstream displacement of arch and thrust blocks.

Interventions included concrete added in 1928 to address stability and seepage. Flood modifications in 1938 included the lowering the crest of arch and wing wall sections raised and concrete added to thrust blocks. In 1947-1950 expansion slots were cut. In 1968 post-tensioned anchors were added. In early 1990's the slots started to close with deformations at the left side contact. In 1999-2003: the right and left side slots were re-cut and the right-side slot was re-cut in 2012. Throughout this period the dam has been regularly inspected and judged to be safe to operate. In 2013 there was no sign of a reduction in the vertical expansion rate. Presently the AAR is not causing instability or operating problems.

South America/Brazil.

34. **ASR Case: Moxoto, (PH) - CHESF, Brazil** [Kuperman, ICOLD 2013]

The Apolônio Sales (Moxotó) Hydro-power plant generates 440MW, was built between 1972 and 1977 and is part of Paulo Afonso Hydroelectric Complex. Since the early stage of the commercial exploitation, the units presented an abnormal performance with progressive shifting and tilting of the turbine shaft. The first visual evidences of Alkali-Aggregate Reaction were several cracks noticed on walls and slabs in 1979. The cutting of three expansion slots between the concrete blocks, performed in 1988-1992 period, improved the performance of the generating units for some time. Meanwhile, the concrete expansion cumulated stresses and strains in the turbine parts fixed in the concrete, such as the stay vanes and the discharge rings. To counteract these effects, a rehabilitation process was implemented. The concrete swelling phenomenon at Moxoto has been and is still being managed through continued inspection of the structure, an extensive monitoring system and safety assessment studies, numerical modelling studies and remedial measures to mitigate the swelling effects. It is reported that there are signs that the swelling effect is decreasing.

35. **ASR Case: Pedra, (Gravity Dam & Spillway) - ANEEL, Brazil.** [Kuperman, ICOLD 2013]

The Pedra Hydropower plant, is a 60m high gravity dam located in Contas river, Bahia state, generates 48MW and was built between 1964 and 1968. In 1991 the presence of ASR was detected in the concrete structures. Besides presenting several places with map cracking, the concrete expansion has caused a lateral thrust on the end pillars of the spillway, transmitted by the adjacent blocks, resulting in operating problems with the left end gate. In 2009, to mitigate the effects of the expansion, four slots were cut with diamond coated cables. A monitoring system composed of multiples rods extensometers, direct pendulums, tri-orthogonal joint meters and bench marks was installed to assess the safety of the structure and to inform whether other rehabilitation procedures will be necessary in the future.

36. **ASR Case: Billings-Pedras, (Gravity Dam & Spillway) - ELETROPAULO, Brazil.** [Kuperman, ICOLD 2013]

Billings-Pedras dam dates from 1936. The concrete structure has a maximum height of 31 m and is 145 m long, comprising three water intakes for a powerhouse that was not built, a gravity dam 35 m long and a space left in the concrete for a lock, that was also not built. AAR was identified in 1992. Instrumentation was installed consisting in tri-orthogonal joint meters, convergence meters, rod extensometers, surface marks and three-dimensional finite-element analysis was performed. In 1996, one of the floodgates began to present operational difficulties, attributed to the deformation of the guides by the expansive reaction. This problem was corrected in 2004/2005 by replacing the guides in the 3 gates. In 2002 a part of the concrete structure was coated with mortar added with micro-silica, for testing. After 4 years, the mortar fell off at many points and the cracks in the original concrete tended to surface again. In 2006 rehabilitation of the structure was carried out by installing drainage at the foundation, adding weight to improve stability conditions, cracks were sealed with injection of polyurethane, and shotcrete over steel mesh was also used in parts of the structure. The rate of expansion due to AAR is variable from 8 to 35 με/year, depending on the part of the structure that is considered.

37. **ASR Case: Furnas (PH & Spillway) - FURNAS, Brazil** [Kuperman, ICOLD 2013]

The Furnas Hydropower plant was built in the early 1960s and has an installed capacity of 1,216 MW. The first noted signs of Alkali-Aggregate Reaction were detected in the mid 70s, when the following events were detected: unleveled central wall blocks and adjacent blocks; cracking on powerhouse structures; cracking on top of spillway pillars. The first measurable displacements occurred in 1976 when an average annual rate of expansion of 22 με/year, was detected.

In 1980, tri-orthogonal meters have been installed on crest joints and on the dam drainage gallery. In 1992/1993, a dragging process was identified on shoe fixed parts of spillway sluice-gates. In 1995 a significant reduction was detected in the annual expansion rate, down to 8 με/year,. During tests of turbine # 6, various equally spaced diameters along the circumference of the bottom stationary wearing ring were found to be out of round. Also, the underlying bottom ring onto which the wearing ring was bolted was also found to be out of round. Further investigation revealed that all 20 wicket-gate stems were found to be out of the vertical plumb line by various amounts. A series of interventions were required in order to restore the design geometrical and dimensional conditions. A monitoring system was installed and a 3D finite-element thermo-chemo-mechanical expansion model was applied in order to analyse how the ASR is affecting the structure.

5.4 References

[5.1], “Alkali-Carbonate reaction in Concrete from Chickamauga dam powerhouse”, Miscellaneous paper n. 6-659; Vicksburg, Mississippi (USA), June 1964.

[5.2] Wong., G., S., “Alkali-Carbonate rock Reaction at Center Hill dam”, U. S. Army Engineer Waterways Experiment Station, Corps of engineers, Miscellaneous paper SL B7 5; Vicksburg, Mississippi (USA), July 1987.

[5.3] CEA (Canadian Electrical Association), CANCOLD (Canadian National Committee on Large Dams), Proc. International Conference on “Concrete Alkali-Aggregate Reactions in Hydroelectric Plants and Dams”, Fredericton, New Brunswick, Canada, October 1992.

[5.4] USCOLD (United States Committee on Large Dams), Proc. 2nd International Conference on “Alkali-Aggregate Reactions in hydro-electrical plants and dams”, Chattanooga, Tennessee, USA, 22-27 October 1995.

[5.5] ICOLD, SPANCOLD (Spanish Committee on Large Dams), International Journal on Hydropower & Dams, Proc. Workshop on “Chemical expansion on concrete in dams and hydroprojects”, R.G. Charlwood and J.M. Buil Sanz Eds., Granada, 18-19 October 2007.

[5.6] ICOLD, USSD (United States Committee on Large Dams), Proc. International. Workshop on Alkali Aggregate Reactions, 81th ICOLD Annual Meeting, Fontana, Tennessee, USA, 20 Aug 2013.

[5.7] French Committee on Large Dams and Reservoirs, Proc Symposium on the Dam Swelling Concrete, DSC 2017, Chambéry, France, Sellier, A., Grimal, É., Multon, S., Bourdarot, E., Eds.; Wiley, London, 13-15 June 2017; 2017.

[5.8] Noret, C., Clave, O., Gurukumba, K., Chibvura, C. "Kariba Dam, more than 50 years of operation and surveillance", Water Storage and Hydro Development for Africa, International Journal for Hydropower and Dams, Marrakech, Morocco, March 2017.

[5.9] Tembe, I., Carvalho, E., Hattingh, L., Oosthuizen, C., "The importance of redundancy in the surveillance of aging dams – the Cahora Bassa experience", International Symposium on Dams in global environmental challenges, 82nd ICOLD Annual Meeting, Bali, Indonesia, June 2014.

[5.10] Brueckner, R., Ndugga, N., Meri, T.C., Mahsen, M., "Alkali-aggregate Reaction at Nalubaale Hydropower Station, Uganda", Water Storage and Hydro Development for Africa, International Journal for Hydropower and Dams, Marrakech, Morocco, March 2017.

[5.11] Hattingh, L.C., Oosthuizen, C., "Unusual Behaviour of a Large Arch Dam in South Africa as a result of Swelling due to Chemical Reaction", Q95-R33, International Symposium on Dams for a changing world, 80th ICOLD Annual Meeting, Kyoto, Japan, May 2012.

[5.12] Casagran, G., Pradolin L., Victor, J., "Dam Rehabilitation in the presence of AAR – The Matala Hep", International Symposium, 84th ICOLD Annual Meeting, Johannesburg, South Africa, May 2016.

[5.13] Bouayad, A., "Reconstruction de l'évacuateur de crues de l'aménagement hydroélectrique de Matala, Aspects de la sécurité des barrages". Water Storage and Hydro Development for Africa, International Journal for Hydropower and Dams, Marrakech, Morocco, March 2017.

[5.14] Chulliat, O., Grimal, E., Bourdarot, E., Boutet, J.M., Taquet, B., "Le gonflement des barrages en beton. apports des recherches scientifiques : application au barrage du Chambon et a son confortement". Q95-R158, International Symposium on Dams for a changing world, 80th ICOLD Annual Meeting, Kyoto, Japan, May 2012.

[5.15] Bourdarot, E., Sellier, A., Multon, S., Grimal, E.,"A review of continuum damage modelling for dam analysis", European Journal of Environmental and Civil Engineering, Published Online: 05 Oct 2011.

[5.16] Noret, C., Laliche, K. "Bimont Dam Case: Studies and investigations inside the dam body, Proc. International Workshop on Dam Swelling Concrete DSW 2017, Chambéry, Savoie, France, June 2017.

[5.17] Mazzà, G., Donghi, G., Marcello, A., Marcello C., "The rehabilitation of Poglia dam: the support of experimental investigations and numerical modelling in the design and during the work-in-progress stages". International Symposium on Operation, Rehabilitation and Upgrading of Dams, 76th ICOLD Annual Meeting, Sofia (Bulgaria), June 2008.

[5.18] Donghi, G., Marcello, C., Sainati F., "Behaviour of the Poglia dam after 7 years from the completion of remedial works, carried out in order to reduce the effects of AAR process". Proc. 9th ICOLD European Club Symposium, Venice, Italy, April 2013.

[5.19] Amberg, F., Stucchi, R, Brizzo, N., "The effect of temperature on the development of the Alkali Aggregate Reaction at the Pian Telessio dam", Proc. 9th ICOLD European Club Symposium, Venice, Italy, April 2013.

[5.20] Amberg Gunleiksrud, O., E-CO Vannkraft, Personal Communication, 2008

[5.21] Ho, M. S., Eastman, K., Adeghe, L, "Recent Development in R. H. Saunders GS Concrete Growth Mitigation Project, Proc. 2nd International Conference on "Alkali-Aggregate Reactions in hydro-electrical plants and dams", Chattanooga, Tennessee, USA, 22-27 October 1995.

[5.22] Eastman K.T., Ho M.S., Adeghe L.N., "Innovative techniques to mitigate concrete expansion – R.H. Saunders GS", HydroVision Conference, Reno, Nevada, 1998.

[5.23] Khoral, P., Hafez, A., Zhao, D. Hong, E., “Operating Hydro Turbines with AAR Induced Growth in Concrete Dams – Otto Holden Generating Station”, HydroVision International Conference, July 2016.

[5.24] Charlwood, R.G., Rutherford, J. H., Curtis, D., Winters, F., Winters, L, Terry D., D., “Lock and dam ASR Repairs”, 25th USSD Annual Meeting and Conference on Technologies to Enhance Dam Safety and the Environment, Salt Lake City, Utah, USA, 6-10 June, 2005.

6 STRUCTURAL ANALYSIS TO ASSESS DAM BEHAVIOUR

6.1 Frameworks for Analysis

In order to assess safety conditions of dams and hydro-electric structures affected by AAR, forecast future behaviour, maintenance planning, and identification and design of any required countermeasures, structural analyses are of primary importance. This chapter presents a framework to assist in the selection of computational models for structural analyses for these purposes.

The establishment of a complete expansion model would need to include both chemical and physical kinetics. The various chemical reactions which cause expansion of concrete are complex and have yet not been fully understood so they are not addressed in fully this Bulletin. However, the main physical behaviour which affect the magnitude and the spatial distribution of AAR expansion can be identified to a sufficient extent to be useful in engineering practice.

In such investigations, the analyses should account for the following aspects:

- geometry of the dam;
- concrete properties;
- external loading conditions including temperature and moisture effects;
- expansion mechanisms within the concrete.

The analysis of concrete expansion caused by AAR might address the influences of:

- variability in concrete composition and heterogeneity (aggregate reactivity and alkali content);
- moisture content and its influence on the concrete expansion;
- effects of temperature on expansion rates; and
- stress state and its influence on the anisotropic expansion rate and creep.

In addition, the presence of AAR in concrete affects its mechanical properties, including stiffness, strength, creep and relaxation properties.

Various computational models have been developed starting in the late 1980's with simple formulations that evolved into more sophisticated mechanical models which have been implemented into the numerical solutions and used successfully in practical applications.

Current modelling approaches may be characterized according to the treatment of time, related processes and material properties and the appropriate method should be chosen to suit the particular project requirements, available analysis tools modelling capability and input data and the level of the engineering investigations.

The calibration of the model to the real structure is an essential and complex part of any analysis. Laboratory data can help estimate the input parameters but verification with field measurements and tests are crucial. Model calibration to deformations alone can be misleading in terms of associated stress estimates and the use of in-situ stress measurements will assist in improving this when stress related issues are important.

Most computation models used on large concrete dams have been developed for unreinforced mass concrete which is appropriate for the main body of large dams. However, in other appurtenant structures, such as powerhouses, spillway piers and bridge decks, reinforcement has also been incorporated. Reinforcing steel would typically reduce rates of concrete expansion even though the reinforcement ratios are relatively low. The slight restraint offered by the reinforcing steel may provide a small but beneficial compressive stress in concrete.

In preliminary investigations, a simple analysis approach may be implemented to model the state of the structure at a given point in time and not to simulate the entire load path of the AAR evolution and deformation development through time. The structural model is calibrated at a certain instant in time for known structure deformations and any significant evidences (cracks, stresses), using effective mechanical properties adjusted to approximate the path dependent AAR concrete expansion behaviour and moisture and temperature conditions. Normally, it is necessary to use a three-dimensional model although in some cases a two-dimensional model may be sufficient to provide some limited understandings. In such model, the accumulated expansion and mechanical properties can be either estimated a priori or established through an iterative process to correlate the model behaviour with deformation and stress data and other known conditions. With this approach, only particular states are reproduced, for example the current state or a future one, without following the expansion process through time. However, the path dependent aspects of the behaviour are frequently very important, and the results from such a simplification can be misleading. In particular, the use of simple models, such as equivalent temperature change models, which can appear to be well adjusted to match some deformations, may in fact be seriously misleading in terms of local deformation and particularly stresses and generally should not be used except for preliminary estimates in some simple cases.

A second approach is to model the progression of the AAR expansion through time, usually in a time step analysis. The load path dependency and resulting expansion heterogeneity and anisotropy within the structure is accounted for at each time step using relationships describing the important factors influencing the expansion rate (usually stress state, temperature, moisture and creep). The expansion phases, as discussed in section 6.1.1 can be modelled as required and the expansion rate input initially as an isotropic “free strain rate”, that is the expansion rate without the modifying effects of stress. According to the model used, said “unrestrained expansion rate” (sometimes referred to as the “free expansion rate”) can depend from the temperature, the concrete composition and the moisture. In practice moisture does not seems to be relevant in dams, but it plays certainly a role in other structures (e.g. powerhouse). The effects of stress on the effective expansion rate is very important and should be considered using a suitable expansion rate versus stress relationship that can also consider anisotropy as discussed below in section 6.1.5. The input “unrestrained expansion rate” then becomes a stress dependent “effective expansion strain rate” which is updated by iterating at each time step to address the coupling between the three-dimensional stress state and the expansion strain rate which may become significantly anisotropic. In addition, the effects of creep relaxation on the stress state should also be considered as this usually becomes significant due to the long-time frames of expansion in dams. However, in practice it might be quite difficult to distinguish the effect of creep from the similar effect of compressive stress on the actual expansion rate. The expansion strain is usually treated as an initial load vector for the time incremental stress computations. The effects of temperature and moisture can be included either based on steady state distributions or as changing throughout the year based on transient analyses as discussed in sections 6.1.3 and 6.1.4. Usually the moisture and temperature behaviour are considered to be independent from the expansion rate and deformation response of the structure, that is, coupled in one direction only.

In this second approach the expansion process can be better followed than with the first approach, since the expansion usually has a strong influence on the stress distribution that, for its part, has an influence on the future expansion. With this approach, the propagation phases can be modelled with a simple linear relation without the stabilization phase of the reaction, since in practice this phase is often not reached within the project management time-frame. However, several authors proposed a complete S-shaped function for the expansion-time relation based largely on laboratory testing, in order to consider a case where the expansion development starts slowly then increases and eventually stabilizes at some time in the future. Since there are numerous cases observed in the field where the expansion rate in the propagation phase appears essentially constant and does not appear to be stabilizing, the premature reliance on this long-term condition should only be considered if there is clear evidence to support it.

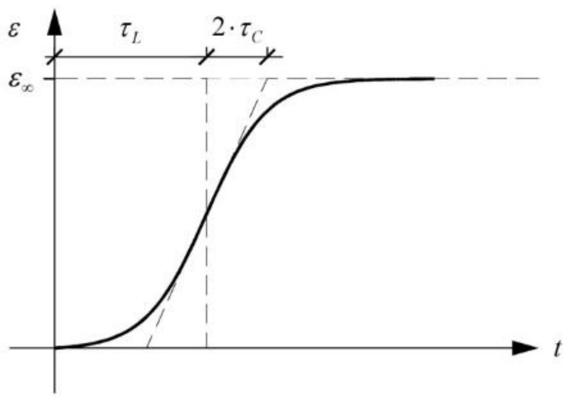
This approach has been found to be manageable and effective in the engineering practice. Some examples of applications of this approach can be found in the papers of Charlwood et al [6.1], Curtis et al. [6.2] [6.3], Reinicker et al. [6.4], Giuseppetti et al. [6.5] Amberg [6.6] [6.7] , Batista et al. [6.8], Capra and Bournazel [6.9], Grimal et al. [6.10], Leger et al. [6.11], Saouma et al. [6.12] [6.13] and others. In some of these approaches the expansion parameters for the structure are estimated based on laboratory tests.

In the following sections, the most significant and used formulations for simulating the expansion evolution and accounting for the factors affecting it are discussed. The use of laboratory test data has been used to help define key parameters and relationships, but their use needs to be made consistent with field measurements and behaviour by thorough calibration of the model as described in section 6.2.

6.1.1 *Evolution of Expansion with Time*

In this section various relationships that have been proposed for characterizing the unrestrained expansion of concrete affected by AAR in time are presented. These relationships have been developed generally based on laboratory experiments. The expansion relations discussed in this section are for “unrestrained expansion”, sometimes referred to as “free expansion” but can be adapted to take into account of the effects of actual temperature, moisture or stress. The effects of phenomena that influence this “unrestrained expansion” are discussed in subsequent sections.

In 1998 Larive [6.14] proposed the following equation for the variation of “unrestrained” expansion *ε(t)* in time based on laboratory expansion tests and is illustrated in Fig. 6.1.

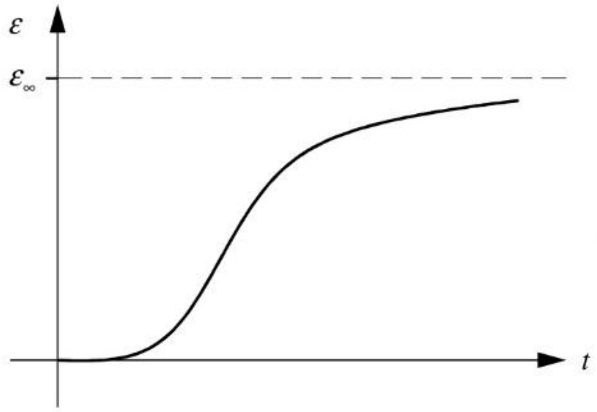


$$\varepsilon(t) = \varepsilon_{\infty} \cdot \frac{1 - \exp(-t/\tau_C)}{1 + \exp(-t/\tau_C + \tau_L/\tau_C)}$$

Fig. 6.1 - Different phases of expansion during core expansion measurements (Larive [6.14])

The function in Fig. 6.1 is characterized by ε_{∞}, the ultimate expansion strain, and two parameters τ_L and τ_C, which are termed the “latency time” and the “characteristic time” of the expansion, respectively. It has been proposed that by varying these two parameters, the shape of the curve and the duration of the “initiation” and “propagation” phases can be adapted to match measured expansion data. However, as discussed in section 6.2, frequently the field measured expansion curves show a much more marked linear phase that is suggested by such “S” curves and a modification of the laboratory based curve is required.

Brunetaud [6.15] proposes a slight modification to the Larive function, by adding a time dependent coefficient *β(t)*, the parameters of which can be selected to better reproduce the long term expansion of the reaction as shown in Fig. 6.2.

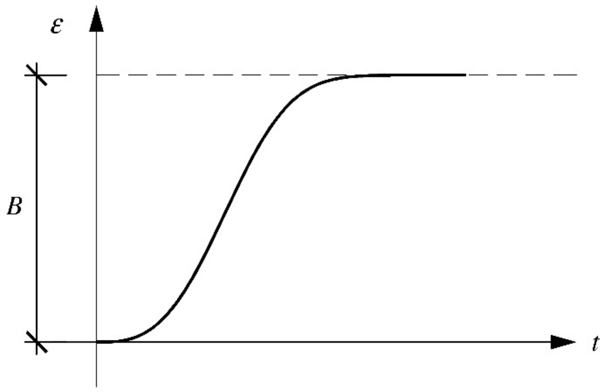


$$\varepsilon(t) = \varepsilon_\infty \cdot \frac{1 - \exp(-t/\tau_C)}{1 + \exp(-t/\tau_C + \tau_L/\tau_C)} \cdot \beta(t)$$

$$\beta(t) = 1 - \frac{\varphi}{\delta + t}$$

Fig. 6.2 - Expansion development (Brunetaud [6.15])

Another example of S-shaped function describing the unrestrained expansion in time is the one proposed by Aguado et al.[6.16] for a sulphate reaction as shown in Fig. 6.3:



$$\varepsilon(t) = B \cdot \left\{1 - \exp\left[-(t/C)^p\right]\right\}$$

Fig. 6.3 - Expansion development (Aguado et al. [6.16]): parameters B defines the rate, C can increase or reduce the initiation phase, while parameter p influences the expansion rate in the propagation phase

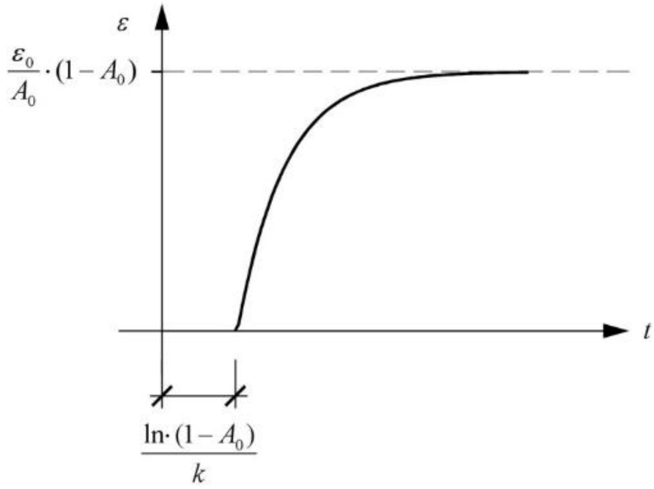
The above presented S-shaped functions are defined for constant conditions in terms of temperature or moisture. Therefore, accounting for variable conditions, such as those occur in a real structure, is not trivial. Different ways are possible. An example can be found in Amberg et al. [6.7], where the Larive-Brunetaud formulation for simulating the behaviour of an arch-gravity dam affected by ASR was used, considering the effect of the variable temperature condition in the dam.

A clear approach in case of variable temperatures, consists into reproducing the kinetic of the chemical reaction with following equation proposed by Capra and Bournazel [6.9]. Here the alkali consumption is simulated in a first step and the expansion in a second. The reaction rate depends linearly on the reactant concentration:

$$\frac{dA}{dt} = k \cdot (1 - A)$$

where A is the ratio of alkalis reacted (variable between 0 and 1) and k is the reaction rate.

The expansion is then assumed to be linearly proportional to the ratio of alkali reacted A. Additionally, to account for an initial time with no expansion, a threshold A_0 is introduced, below which the expansion remains zero. The resulting expansion development with time is presented and illustrated in Fig. 6.4:



$$\varepsilon(t)=\begin{cases}0 & \text{for } A<A_0\\ \dfrac{\varepsilon_0}{A_0}\cdot\left(A-A_0\right) & \text{for } A\geq A_0\end{cases}$$

Fig. 6.4 - Expansion development (Capra and Bournazel [6.9])

6.1.2 *Linking Laboratory and Field Behaviour*

The best reliable way to evaluate the potential for further expansion (residual concrete expansion) in existing structures is to use their in-situ monitoring but, in practice, at least over five years are usually required to obtain reliable information, as the AAR contribution needs to be isolated from seasonal variations in temperature. This is why accelerated laboratory tests at high temperature (38°C) and humidity conditions (sometimes also in 1 N NaOH solutions) are commonly carried out on cores taken from the dams, leading to estimates within shorter periods.

However, the expansion of concrete obtained in laboratory conditions may not be directly used to assess the potential for further AAR expansion rate in structural analysis of actual structures. As already pointed out in the section 4.6, the influence of so many factors affecting these core expansion tests is such as to challenge the reliability of the obtained results, that need to be verified ex-post with field long time monitoring and calibrated with the state and the behaviour of the actual structure. In particular, the outcome from accelerated laboratory tests is usually quite different from the behaviour of concrete observed in the field due to the high alkali solutions and high temperatures used to conduct an accelerated test and may perhaps lead to significantly inaccurate results.

Unfortunately, information relating expansion results obtained in the laboratory with those observed in the field is currently very limited, although research in this area is continuing. For example, the RILEM Technical Committee TC 258 AAA (Avoiding Alkali Aggregate Reactions in Concrete - Performance Based Concept) is preparing an overview of "lab-field correlation" from exposure sites and real concrete structures in services, with a view to possible future recommendations for the use of laboratory developed results.

In the present state of knowledge, a useful reference to deal with this challenge is the “Procedure for laboratory assessment of the current rate of ASR Expansion in field concrete” suggested by Bérubé et al. ([6.17] and Appendix I of [6.18]). This methodology had been applied by the authors to a number of existing structures in Québec (Canada) with some applications to dams. Bérubé et al. proposed the use of the results of core testing in air at relative humidity above 95% and temperature of 38°C to estimate the expansion in the structure. The experimental expansion value

(%/year) of this test after 1 year testing (Fig. 4.34 in Section 4.6) is used to determine the Coefficient RCE or Residual Concrete Expansivity, which varies from 0 to 16 (Table 6.1([6.18]).

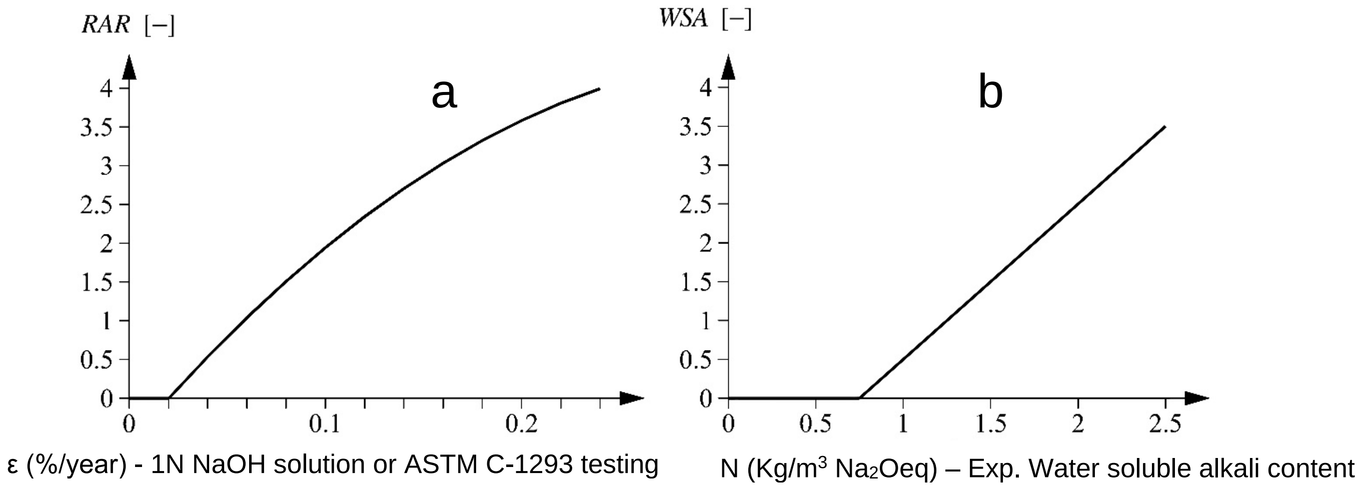
Table 6.1 – Correlation between 1 year expansion on concrete cores in laboratory (RH>95% and 38°C) and the RCE Coefficient [6.18]

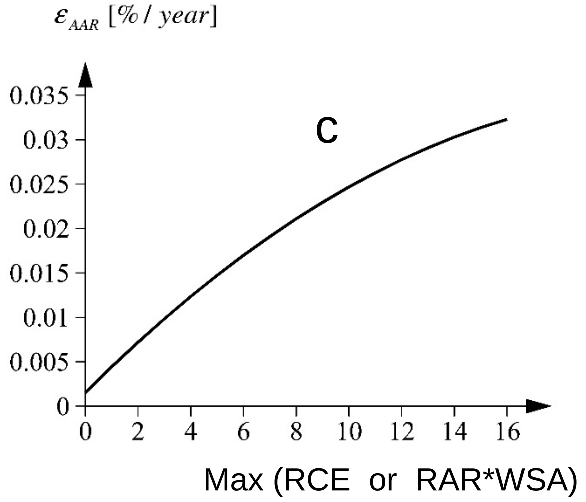
expansion rate (%/year)	RCE Coefficient	expansion rate (%/year)	RCE Coefficient
< 0.003	0	0.015 to 0.02	6
0.003 to 0.005	1	0.02 to 0.025	9
0.005 to 0.01	2	0.025 to 0.03	12
0.01 to 0.015	4	> 0.03	16

Alternatively, the results of core testing in a 1N NaOH solution at 38°C (Section 4.6) or the results of the Concrete Prism Test ASTM C-1293 [6.19] (high alkali content concrete in high humidity conditions at 38°C) on aggregates extracted from cores can be considered, to determine the Coefficient RAR or Residual Aggregate Reactivity, which varies from 0 to 4 (Fig. 6.5 a). But this last Coefficient have to be corrected accounting for another Coefficient, WSA (Water-Soluble Alkali), which also varies from 0 to 4, determined by means of the experimental water soluble alkali content of the concrete cores (hot-water extraction method in Appendix H of [6.18]) as shown in Fig. 6.5 b.

The maximum values between the Coefficient RCE or the product of RAR*WSA (both from 0 to 16) is taken to estimate the current rate of AAR expansion (ε_{AAR} %/year) that is expected in the coming years in concrete structures in service, according to the formulae presented in Fig. 6.5 (c). ε_{AAR} is the equivalent expansion rate to be applied to the model for unrestrained conditions, high humidity and 38°C. Corrections for different conditions have to be taken into account according to following sections.

Modelling results derived from this procedure should be validated for each application against in-situ measured expansion rates in the subject structure accounting for the restraint and other site-specific conditions.





$$RAR = -44.153 \cdot \varepsilon^2 + 29.660 \cdot \varepsilon - 0.582 \geq 0$$
$$WSA = 2.000 \cdot N - 1.500 \geq 0$$
$$\varepsilon_{AAR} = 0.001 \cdot \left(-0.065 \cdot C_3^{\,2} + 2.966 \cdot C_3 + 1.531\right)$$

Fig. 6.5 – (a) Relationship to correlate the expansion rate of 1N NaOH or ASTM C 1293 testing to Coefficient RAR – (b) Relationship to correlate the experimental water soluble alkali content of the concrete cores – N - to Coefficient WSA – (c) Relationship to correlate the expansion rate observed in laboratory (Max between RCE or RAR*WSA) with the expansion rate in the field ε_{AAR} (%/year) to be used in the model

6.1.3 *Moisture*

Several authors (Bérubé et al. [6.17], Capra and Bournazel [6.9], Leger et al. [6.11], account for moisture with a coefficient ($\varepsilon_{RH} / \varepsilon_{\infty}$) that reduces the total unrestrained concrete expansion (Fig. 6.6). Capra and Bournazel [6.9] propose the following analytical function for such a coefficient:

$$\frac{\varepsilon_{RH}}{\varepsilon_{\infty}} = \left(\frac{RH}{100}\right)^{8}$$

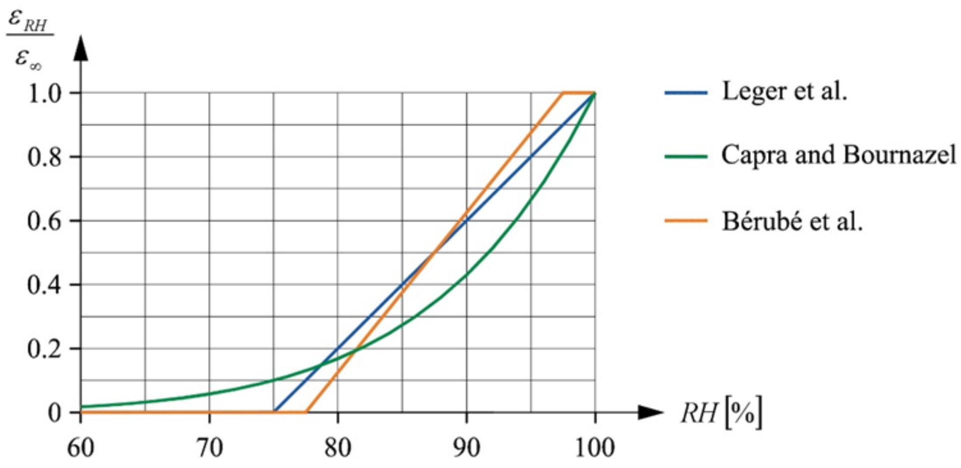


Fig. 6.6 - Effect of moisture (RH: relative humidity) on expansion (Bérubé et al. [6.17] Capra and Bournazel [6.9] , Leger et al. [6.11]

Poyet et al. [6.20] considers a coupling between the saturation ratio *Sr* and the reaction kinetics proposed by Capra and Bournazel:

$$\frac{dA}{dt} = k \cdot \alpha \cdot (\beta - A)$$

where:

α = parameter, dependent on the saturation ratio, which modify the kinetics of the reaction;

β = parameter, dependent on the saturation ratio, which modify the maximal reaction advancement;

Poyet et al. [6.20], basing on the results of some laboratory tests, conclude that the relationship between the two parameters and the saturation ratio is linear with:

$$\alpha = Sr$$
$$\beta = Sr$$

This approach has been applied in other expansion models, such as the one of Grimal et al. [6.21], with good agreements with the actual behaviour of the considered structures.

6.1.4 *Temperature*

In cases where the temperature varies significantly, either spatially or temporally, the relationship between the reaction rate and temperature can be included into the model according to the Arrhenius equation:

$$k = k_0 \cdot \exp\left(-\frac{E_a}{R \cdot \theta}\right)$$

where k_0 is a pre-exponential factor, E_a is the activation energy (40-50 kJ/mol, typical value for mass concrete) which is considered to correspond to the rupture of silanol bonds (Si—OH), R is the universal gas constant (8.314 J/mol/K) and θ is the absolute temperature (in Kelvin). The Arrhenius equation is linear in a semi logarithmic space, if plotted versus the inverse of the temperature.

The relationship is agreed to hold for temperatures up to around 38°C-50°C and not above. As a very rough approximation it may be assumed that the reaction rate doubles for every 10°C increase in temperature. For a given concrete, the activation energy can be determined by running expansion tests at least 3 temperatures in the range in which the relation is considered valid.

The first order kinetic law presented by Capra and Bournazel [6.9] is thus modified as following:

$$\frac{dA}{dt} = k_0 \cdot \exp\left(-\frac{E_a}{R \cdot \theta}\right) \cdot (1 - A)$$

Larive [6.14] proposed the following laws for modifying the characteristic time and the latency time in function of the temperature:

$$\tau_C(\theta) = \tau_C(\theta_0) \cdot \exp\left[U_C \cdot \left(\frac{1}{\theta} - \frac{1}{\theta_0}\right)\right] \qquad U_C = 5'400 \pm 500\,K$$

$$\tau_L(\theta) = \tau_L(\theta_0) \cdot \exp\left[U_L \cdot \left(\frac{1}{\theta} - \frac{1}{\theta_0}\right)\right] \qquad U_L = 9'400 \pm 500\,K$$

where $\tau_C(\theta_0)$ and $\tau_L(\theta_0)$ are the time constants at the reference temperature θ_0, U_C and U_L are the activation energy constants. Also, Larive's laws are linear in a semi logarithmic space, if plotted versus the inverse of the temperature. Hence, the laws can be determined with a linear fitting of the results of three tests at different temperatures.

The parameters defining the time dependent coefficient *β(t)* proposed by Brunetaud [6.15] can also be defined as a function of the temperature:

$$\delta(\theta) = \delta(\theta_0) \cdot \exp\left[U_\delta \cdot \left(\frac{1}{\theta} - \frac{1}{\theta_0}\right)\right]$$

$$\varphi(\theta) = \varphi(\theta_0) \cdot \exp\left[U_\varphi \cdot \left(\frac{1}{\theta} - \frac{1}{\theta_0}\right)\right]$$

The following Fig. 6.7 shows the change of the reaction kinetic with the temperature according to Larive and Capra and Bournazel: the higher is the temperature, the more rapid is the reaction development and short the initiation time.

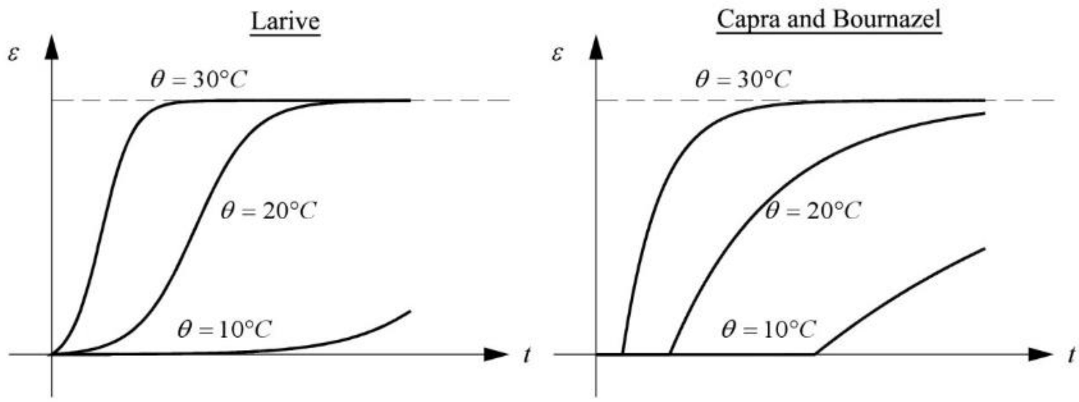


Fig. 6.7 - Effect of temperature on the reaction development accordin to Larive [6.14] and Capra and Bournazel [6.9]

The relationship between temperature and reaction rate would be non-linear, but in the typical range of temperature recorded in dams the non-linearity is practically negligible [6.7].

The effect of the temperature on the reaction development can also be accounted for with the maturity approach [6.22]. The maturity approach can be used with any type of function simulating the development of the expansion with time. Based on the Arrhenius equation, an equivalent time is defined as follow:

$$t_{eq} = \sum_{0}^{t} \exp\left[-\frac{E_a}{R}\cdot\left(\frac{1}{\theta}-\frac{1}{\theta_0}\right)\right]\cdot \Delta t$$

where:

t_{eq} = equivalent time at the reference temperature;

θ_0 = absolute reference temperature [K].

Using the above equation, the actual time is converted to an equivalent time, in terms of reaction advance, at the reference temperature. As an example, 1000 days at 20°C is equivalent to 484 days at 10°C, assuming an activation energy of 50 kJ/mol. The following Fig. 6.8 shows the effect of the application of the maturity approach on the development of expansion recorded at two different temperatures.

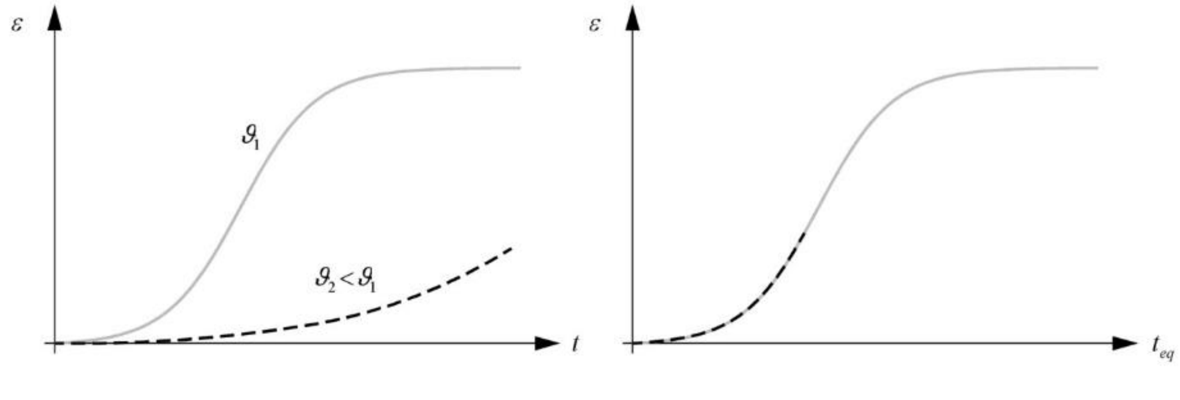


Fig. 6.8 - Effect of the maturity approach

In most cases the use of spatially varying mean annual temperature profiles from steady state analyses may be used to represent the effect of temperature on expansion rates. In cases where it is necessary to simulate the variation of expansion rate throughout the annual cycles, it has been found that transient analyses can be adequately performed using 12 to 24 increments per year to capture the effects of cyclic temperature variation. [6.4] [6.12].

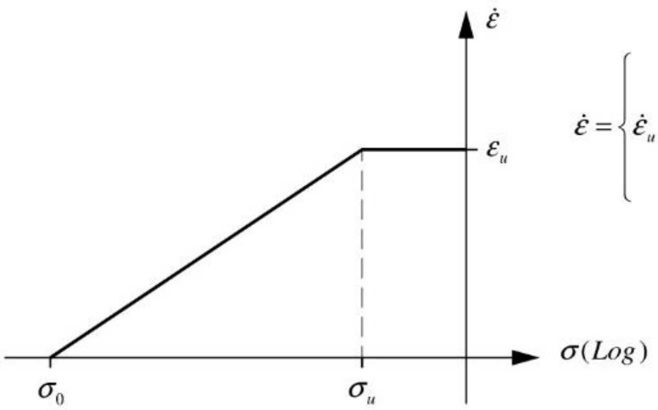
6.1.5 *Stress state*

In some early investigations, modelling the AAR expansion of concrete was treated as an equivalent to a linear thermal isotropic expansion, uncoupled from the mechanical process. Although these early models could match displacements, frequently they predicted much higher stresses than were actually present in the real structures. Subsequently, an improved agreement between both measured displacements and stresses was obtained using a concrete growth model which assumed the effect of stress dependency on concrete expansion in the three principal directions was uncoupled i.e., the concrete growth rates in each principal stress direction were controlled by the principal stress in that direction.[6.1] [6.2] [6.23].

The anisotropic behaviour was clearly identified at the Mactaquac Intake structure, where the vertical rise of the Intake was monitored before and after slots were cut between each Intake unit. Prior to slot cutting the longitudinal compressive stress in the upper portion of the Intake was found to be about 2.8 MPa and after slot cutting this stress was relieved. However, the measured rate of vertical rise did not change as a result of slot cutting and the rate of concrete longitudinal expansion increased and both responses confirmed an uncoupled response to the imposed stress changes i.e., the mean stress changed, but the vertical stress and corresponding expansion did not change.

The stress state has therefore a strong influence on the concrete expansion rate caused by AAR and in order to reasonably match the deformation and stress state in a structure it is normally necessary to account for this influence in the analytical model.

The stress-dependent anisotropic expansion rate function that was developed to capture these properties for modelling Mactaquac Dam and other projects [6.23] has the expansion rate varying with the logarithm of concrete principal stresses as shown in Fig. 6.9:



$$\dot{\varepsilon} = \begin{cases} \dot{\varepsilon}_u & \text{if } \sigma_i \geq \sigma_u \\ \dot{\varepsilon}_u - \dot{\varepsilon}_u \cdot \dfrac{Log\,\sigma_i/\sigma_u}{Log\,\sigma_0/\sigma_u} & \text{if } \sigma_0 \leq \sigma_i < \sigma_u \\ 0 & \text{if } \sigma_i < \sigma_0 \end{cases}$$

Fig. 6.9 - Effect of compressive stress on concrete expansion rate (Curtis [6.2])

where:

ε_u = unrestrained expansion rate;

σ_i = principal compressive stresses (i = 1,3);

σ_0 = compressive stress above which the expansion rate is negligible (in the range 5 - 10 MPa);

σ_u = compressive stress below which the expansion rate is equal to the unrestrained rate (approximately 0.3 MPa).

The formula in Fig. 6.9 was successfully implemented in the finite element (FE) used in the remedial measures design process at Mactaquac Dam [6.1]. With the use of this relationship, the results of the FE analysis conducted for Mactaquac Dam matched well to field measured deformations and stresses by the overcoring method. In addition, the early use of the FE model for the intake and diversion sluiceway showed that it made very good predictions of slot cut response. In this case the critical compressive stresses were estimated from back calculation to be in the range 5.5 to 8.3 MPa and the anisotropic effects of stress on expansion rates were clearly exhibited. The anisotropic response of expansion rates to stress has also been observed at various other dams [6.2] [6.6].

Subsequently, Curtis et. al. [6.3] has extended the relationship to include stress dependence in the tensile zone as illustrated in Fig. 6.10.

The stress dependent growth law characterizing AAR-induced growth of concrete is given as follows:

if $\bar{\sigma}_i \leq -\sigma_{lim}$,

$$\dot{\varepsilon}_{gi}(t) = \dot{\varepsilon}_{g0}(t)(m+1)$$

if $-\sigma_{lim} < \bar{\sigma}_i < 0$

$$\dot{\varepsilon}_{gi}(t) = \dot{\varepsilon}_{g0}(t)\left[1 + m(\frac{\bar{\sigma}_i}{-\sigma_{lim}})\right]$$

if $\bar{\sigma}_i < 0$ and $\dot{\varepsilon}_{gi}(t) > \dot{\varepsilon}_{max}$

$$\dot{\varepsilon}_{gi}(t) = \dot{\varepsilon}_{max}$$

if $0 \leq \bar{\sigma}_i \leq \sigma_0$

$$\dot{\varepsilon}_{gi}(t) = \dot{\varepsilon}_{g0}(t)$$

if $\sigma_0 < \bar{\sigma}_i < \sigma_{lim}$

$$\dot{\varepsilon}_{gi}(t) = \dot{\varepsilon}_{g0}(t) - K\left[log(\frac{\bar{\sigma}_i}{\sigma_0})\right]$$

if $\bar{\sigma}_i \geq \sigma_{lim}$

$$\dot{\varepsilon}_{gi}(t) = 0$$

Where

$\dot{\varepsilon}_{gi}(t)$ = AAR-induced strain in principal directions at a time (t).

$\dot{\varepsilon}_{g0}(t)$ = unrestrained concrete growth rate at low stress at time (t). This varies depending on the location in the structure i.e., a calibration input variable.

$\dot{\varepsilon}_{max}$ = maximum unrestrained concrete growth rate (250 micro-strain/year was used).

K = slope of the line defining the concrete growth rate versus the log of stress.

$\bar{\sigma}_i$ = stress in any of the three principal strain direction (*i* = 1 to 3).

σ_0 = low compressive stress cut-off, such that the values of AAR-induced strain due to compressive stresses between and equal to 0 and σ_0 are equal to the unrestrained growth rate (a value of 40 psi or 0.3 MPa was used).

σ_{lim} = cut-off stress level beyond which no growth is assumed to occur if the concrete is in compression (a value of 800 psi or 5.5 MPa was used).

m = slope of the line defining the concrete growth rate in tension. A value from 0 to 3 may be taken (a value of 2 was used).

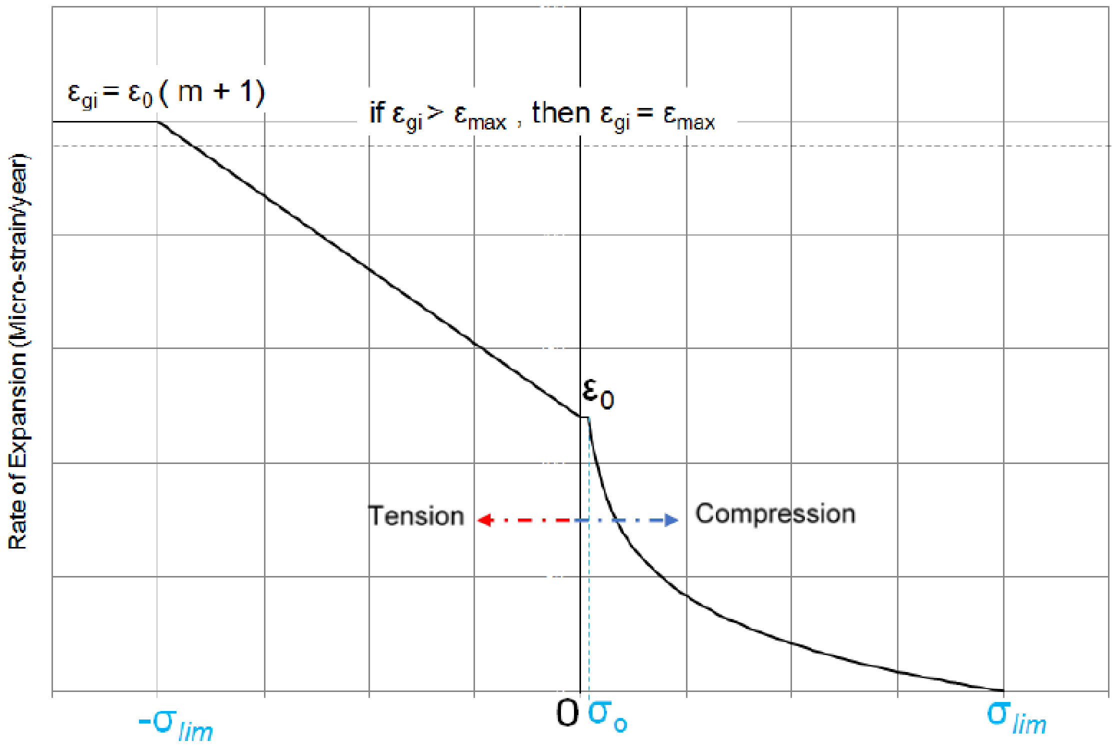


Fig. 6.10 - Updated Concrete Growth Law (Curtis et al [6.3)

Multon [6.24] suggested a different function (Fig. 6.11) to account for the effect of compressive stress on the concrete expansion. The following expression gives the factor that is applied to the unrestrained expansion:

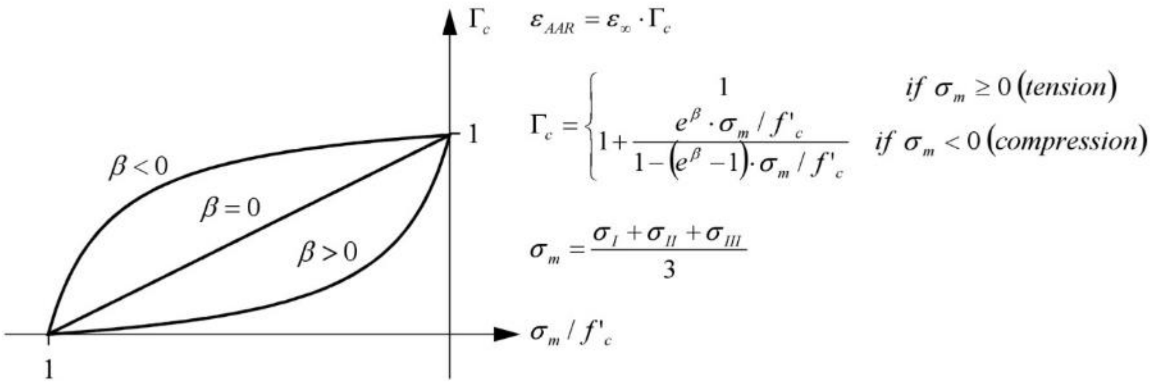


Fig. 6.11 - Effect of compressive stress on expansion (Multon [6.24])

where:

σ_m = average principal stress;

f'_c = compressive strength.

In this approach the expansion rate is a function of the average principal stress and therefore anisotropy effects are not taken into account. Saouma and Perotti [6.12] have accounted for this effect (Fig. 6.12), by assigning weights for each of the three principal directions, such that the sum of the weights is equal to 1.

Saouma and Perotti [6.12] also propose to consider a reduction of the expansion in case of tensile cracking. The following factor is applied to the unrestrained total expansion:

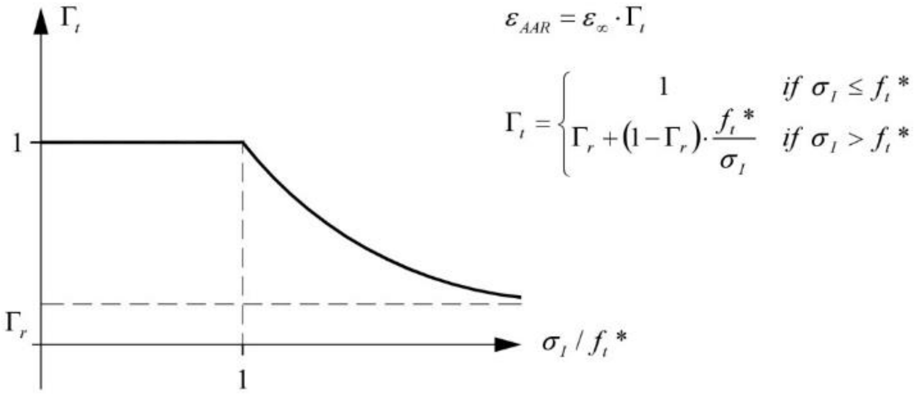


Fig. 6.12 - Effect of tensile stress on expansion (Saouma and Perotti [6.12])

where:

Γ_r = residual expansion under tensile stresses;

f_t * = fraction of the tensile strength beyond which the gel is absorbed by the cracks;

σ_I = maximum principal stress.

Saouma and Perotti [6.12] suggest also a relationship between the latency time of the Larive S-shaped function and the compressive stress for accounting the retardation effect of the compressive stress (Fig. 6.13).

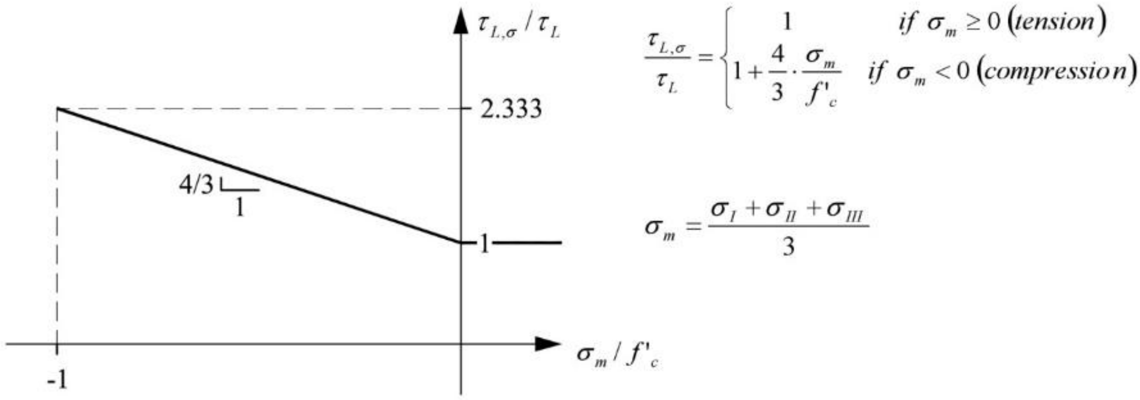


Fig. 6.13 – Effect of stress on latency time (Saouma and Perotti [6.12])

Saouma [6.25] used the results of laboratory tests to develop weighting factors to be applied to a volumetric AAR expansion and thereby proportion concrete expansion in each direction.

6.1.6 *Creep*

A simplified numerical model to include the creep effect is based on the following relation (Curtis [6.2])

$$\varepsilon_{creep}(t) = \frac{\sigma_0}{E_0} + \frac{\sigma_0}{E_1} \cdot \left(1 - e^{-\frac{t}{t_1}}\right)$$

where:

E_1 = parameter to account for creep;

t_1 = parameter to account for creep;

ε_{creep} = creep strain;

σ_0 = initial stress;

E_0 = elastic modulus;

t = time.

The parameters E_1 and t_1 can be obtained from experimental tests on loaded standard specimens.

Fig. 6.14 – Creep model: a) standard solid model; b) creep experimental data [6.3]

Fig. 6.14 shows that a good correlation was obtained between measurements and analysis [6.3].

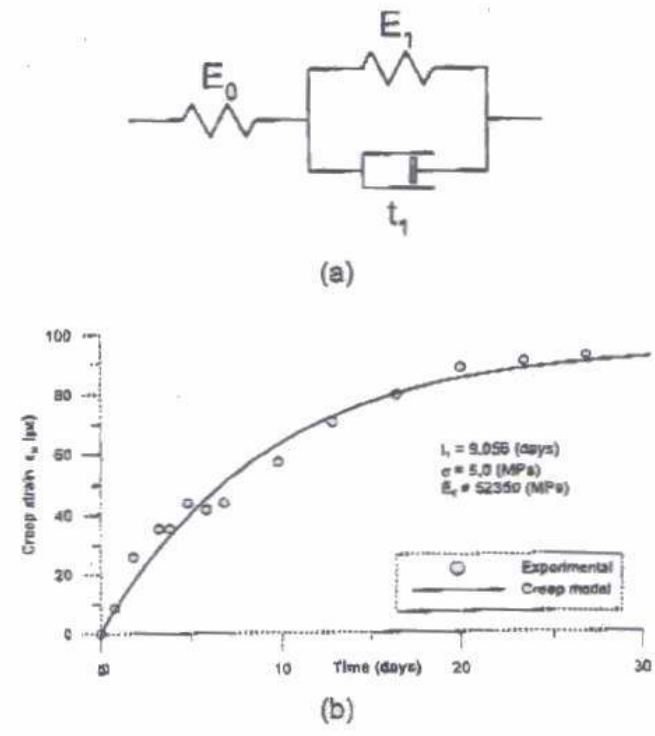


Fig. 6.14 – Creep model: a) standard solid model; b) creep experimental data [6.3]

Other simplified methods used to account for the creep effects are based on a reduction of the modulus of elasticity in order to obtain an “effective elastic modulus”:

$$E_{eff} = \frac{E_{ci}}{1+\phi}$$

where:

E_{eff} = effective concrete elastic modulus to be used in the numerical model;

E_{ci} = instantaneous elastic modulus.

The parameter ϕ can range between 0.2 for short term loads and 2.5 for long term loads, e.g. slot cutting, according to Charlwood [6.1]. Leger et al. [6.11] suggest for ϕ a value of 0.5 for variable loads (seasonal variation of temperature and water level) and 1.5 for AAR induced stress, assuming E_{ci} equals to 22.9 GPa.

6.1.7 *Effects on Mechanical Properties*

Various relations have been suggested to account for the material properties degradation with the development of the AAR.

Leger et al. [6.11] propose a reduction of the tensile strength (f_t) and of the elastic modulus (E_c) with time:

$$f_t = f_{t0} \cdot (1 - k_2 \cdot t)$$
$$E_c = E_{c0} \cdot (1 - k_3 \cdot t)$$

where:

f_{t0} = initial value of the tensile strength;

E_{c0} = initial value of the elastic modulus;

k_2 = parameter that define the tensile strength reduction (estimated from literature or laboratory tests);

k_3 = parameter that define the elastic modulus reduction (estimated from literature or laboratory tests);

Saouma and Perotti [6.12] relate the parameters reduction directly to the occurred expansion:

$$f_t = f_{t0} \cdot \left[1 - (1 - \beta_f) \cdot \frac{\varepsilon}{\varepsilon_\infty}\right]$$

$$E_c = E_{c0} \cdot \left[1 - (1 - \beta_E) \cdot \frac{\varepsilon}{\varepsilon_\infty}\right]$$

where:

β_f = parameter that define the tensile strength reduction;

β_E = parameter that define the elastic modulus reduction;

$\frac{\varepsilon}{\varepsilon_\infty}$ = ratio of the current expansion to the final one;

6.2 Calibration of Analytical Models to Field Behavior

The expansion model for concrete affected by AAR to be used in structural evaluations will represent the combination of the factors presented in Section 6.1. Some of these factors may have a negligible effect on the analysis results, others a relevant one. For example, hydraulic structures are usually highly saturated and variations in the moisture levels have limited effects on concrete expansion. In such case the saturation ratio can be assumed for simplicity equal to 1 for the entire dam with satisfactory accuracy in the analysis [6.6] [6.12]. Also, Amberg et al. [6.6] obtained very satisfactory results in the final assessment of the concrete expansion without considering the effect of stress and moisture for the specific case of Pian Telessio arch-gravity dam. On the other hand, the model established for the design of the rehabilitation works, considered an anisotropic expansion following the compressive stresses, but resulted finally to not be sufficiently reliable, since it was not able to reproduce the behaviour of the structure during and after slot cutting. In general, it is suggested to start the analysis using a simple model that can be expanded based on analysis results obtained from the initial models.

The input parameters of an expansion model can in some cases be estimated initially from laboratory test results and consider evolution curves such as those presented in section 6.1.1 but will need to be confirmed using field measurements. The expansion obtained with laboratory tests may vary significantly from the actual behaviour in the field, even after taking into account the appropriate corrections for the different moisture, temperature and stress conditions. This difference is illustrated by the difference in shapes of the “evolution” curves shown in section 6.1.1 with measured field curves as shown in section 3.1. In many cases, after the “initiation phase”, the “propagation phase” exhibits a linear path for many years with no sign of the “stabilization phase”. The reasons for these important differences are twofold. Firstly, the differences between laboratory test conditions and concrete mix compositions, including size of aggregates, and alkali availability and field conditions and the actual structure concrete. Secondly, in large structures with ASR, in the long term, there may be supplementary supplies of alkalis from the aggregates and recycling of the reactions which can cause the reaction and expansion to continue for long periods of time.

Models should therefore be calibrated against the data recorded from the monitoring system (displacements and stresses) and from field investigations and inspections (stresses, cracks). Data should be recorded for a long period of time to develop a clear trend in the dam performance. Levelling measurements are useful to highlight an expansion process. In addition, by comparing the measurements along the height of the dam over an extended period of time allows a good estimation of the total expansion occurred in the structure.

Concrete expansion as a result of AAR and other reactions within structures is not uniform and is related to the as built variations of mix designs, cements, aggregates and concrete construction and to the actual influences of moisture, temperature and stress distribution in the structure. This clearly complicates the assessment of expansive chemical reactions in dams.

Basically, it should be recognized that the effects of an internal expansion are different from an external load. While a structure stretched by external loads exhibits tensile stresses, a structure affected by internal expansion, which in an unrestrained case would stretch the structure even more, exhibits compressive stresses in cases where the expansion is restrained by the static system.

Another characteristic of non-uniform internal expansion is that stresses and observed displacement are not directly correlated. This can be explained by considering the simplified model presented in Fig. 6.15 [6.6]. For the same model, formed by 3 strain-elements, and for 3 different load cases, the same displacement of the common node towards the left hand side is obtained.

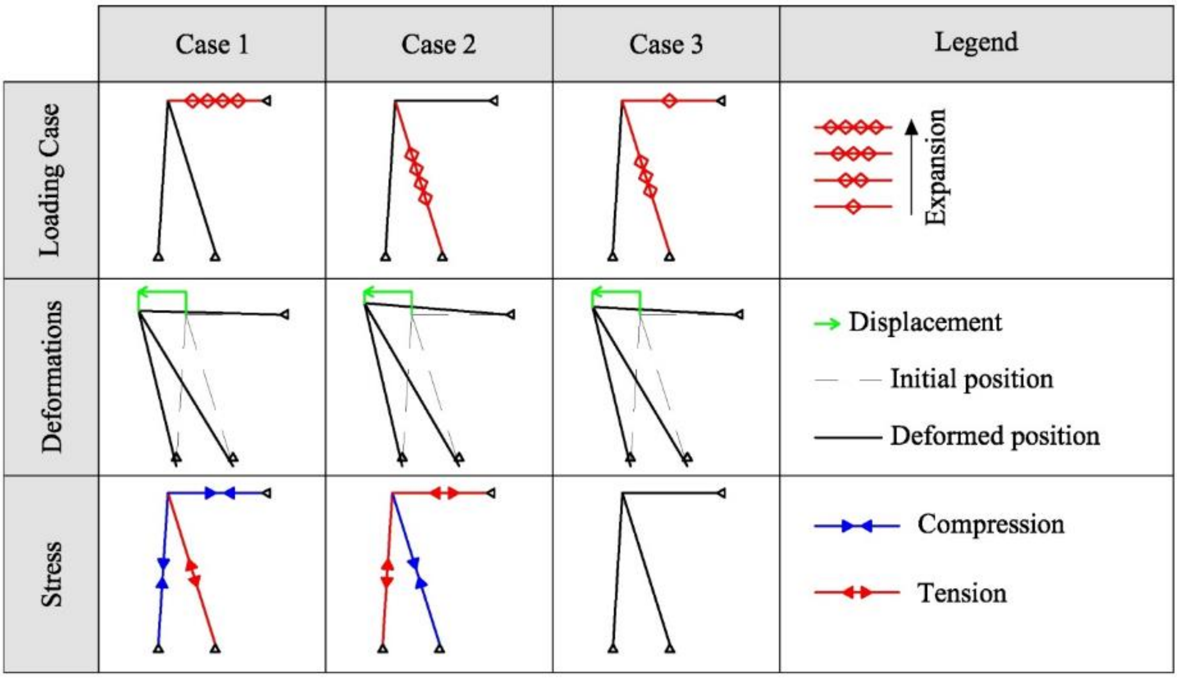


Fig. 6.15 – Variation of static equilibrium due to internal expansion [6.6].

For Case 1 an expansion only in the horizontal element is considered. Since its free expansion is partially hindered by the stiffness of both the sub-vertical elements, a compressive stress might result in the horizontal one. Since the two other elements are pushed towards the left-hand side, a tensile stress is obtained in the right element and a compressive in the left one.

In Case 2 a different loading case is considered: an expansion occurs only in the right sub-vertical element. The elongation of said element is somehow hindered by the left sub-vertical element, and due to their interaction, a rotation and deformation towards the left-hand side occurs. The horizontal element responds to this deformation. Finally, both the horizontal and the left sub-vertical elements are in traction, while the right sub-vertical element is in compression.

Case 1 and 2 might show the same macroscopic deformations, but the internal stresses are completely different. One might also imagine a combination of these two loading cases, where finally no stresses are induced within the model. This situation is represented by Case 3.

This example is simplified but allows to highlight the difficulties encountered by assessing the actual internal equilibrium of a static undetermined structure, such as an arch dam. In an arch dam or curved gravity dam, the arch effect is represented by the horizontal element, and the cantilever effects by both the sub-vertical elements. In these structures the stress state cannot be assessed by means of the observed macroscopic deformations because the two properties are not directly related. In other words, the precision of a calibration in terms of displacements does not ensure the correctness of the predicted stress state.

In order to assess with more accuracy in the stress state of a dam, the calibration should be done for an isostatic situation, such as after slot cutting when the dam cantilevers are independent from each other. However, this situation is often not available for a dam before the remedial works. Some indications can be obtained from stress measurements, but these are seldom available and often not reliable.

In absence of indications that suggest other stress state, the most critical one between those giving the same permanent displacement should be assumed for assessing the structural safety of the dam.

A reliable and correct comprehensive assessment of a massive structure with internal concrete expansion requires engineering judgement, while numerical analyses can only be a support of this judgement.

6.3 References

[6.1] Charlwood, R.G., “A review of alkali-aggregate reactions in hydroelectric plants and dams” Hydropower and Dams, Vol. 1, n.3, pp. 73-80, 1994.

[6.2] Curtis, D.D., “Modelling of AAR Affected Structures Using the GROW3D FEA Program”, Proc. Second International Conference on Alkali-Aggregate Reaction in Hydroelectric Plants and Dams, USCOLD, Chattanooga, Tennessee, pp. 457 – 478, 1995.

[6.3] Curtis, D.D., Feng, L., Sooch., G., Zheng, J., Fletcher, J. “Practical Analysis and Assessment of AAR Affected Dams and Hydroelectric Plants”, USSD Annual Conference, Denver, 2016.

[6.4] Reinicker, B., Abedzadeh, F., Charlwood, R. Cima, “An advanced model to simulate ASR behaviour at Roanoke rapids dam”, USSD Annual Meeting and Conference, Phoenix, Arizona, 2013.

[6.5] Giuseppetti, G., Donghi, G, Marcello, A., “Experimental investigations and numerical modelling for the analysis of AAR process related to Poglia dam: evolutive scenarios and design solutions”, SEPREM International Congress on Conservation and Rehabilitation of Dams, Madrid 11-13 November 2002.

[6.6] Amberg, F., “A review of expanding concrete cases and consequences on dam performance” Hydro 2012 Conference, Bilbao, 2012.

[6.7] Amberg, F., Stucchi, R., Brizzo, N., “The effect of temperature on the development of Alkali Aggregate Reaction at the Pian Telessio dam”, 9th ICOLD European Club Symposium, 2013.

[6.8] Batista, A., Dias, I., Gomes, J.P., Carvalho, E., Tembe, I. “Updating of the behaviour analysis and interpretation of Cahora Bassa Dam (Mozambique) considering the mid-bottom spillway structures”. Proc. Third International Dam World Conference, Brazil, 2018.

[6.9] Capra, B., Bournazel, J., “Modelling of Induced Mechanical Effects of Alkali Aggregate Reactions,” Cement and Concrete Research, Vol. 28, n.. 2, pp. 251-260, 1998.

[6.10] Grimal, E., Sellier, A., LePape, Y., Bourdarot, E. “Creep, shrinkage, and anisotropic damage in alkali-aggregate reaction swelling Mechanism-Part I: A constitutive model” ACI Materials Journal, Vol. 105, n. 3, pp.:227-235, 2008.

[6.11] Léger, P., Côte, P., and Tinawi, R., “Finite Element Analysis of Concrete Swelling due to Alkali-Aggregate Reactions in Dams,” Computers & Structures, Vol. 60, n.. 4, pp. 601-611, 1996.

[6.12] Saouma, V, Perotti, L., “Constitutive model for alkali-aggregate reaction”, ACI Material Journal, Vol. 103, n. 3, pp. 194–202, 2006.

[6.13] Saouma, V., Perotti, L., Shimpo, T., “Stress Analysis of concrete structures subjected to Alkali-Aggregate Reactions”. ACI Structural Journal, pp.532-541, Sept-October 2007.

[6.14] Larive, C., “Apports Combin'es de l’Experimentation et de la Modélisation à la Comprehension del’Alcali-Réaction et de ses Effets Mécaniques,” PhD thesis, Thése de Doctorat, Laboratoire Central des Ponts et Chaussées, Paris, 1998.

[6.15] Brunetaud, X., "Étude de l'influence de différents paramètres et de leurs interactions sur la cinétique et l'amplitude de la RSI au béton", PhD thesis, Ecole centrale de Paris, 2005.

[6.16] Aguado, A., Rodríguez-Ferrán, A., Casanova, I., "Funciones de ajuste del comportamiento de expansiones de hormigón de presas", Proc. Int. Congress on Conservation and Rehabilitation of Dams, Madrid, Spain, pp. 137-143, 2002.

[6.17] Bérubé, M.-A., Frenette, J., Pedneault, A., Rivest, M., "Laboratory assessment of the potential rate of ASR expansion of field concrete" Cement, Concrete and Aggregates, vol. 24, n. 1, pp. 13-19, 2002.

[6.18] Fournier, B., Bérubé, M.A., Folliard, M., Thomas, M., "Report on the Diagnosis, Prognosis, and Mitigation of Alkali-Silica Reaction (ASR) in Transportation Structures", Report FHWA-HIF-09-004, January 2010.

[6.19] American Society for testing and Materials, ASTM C1293-18a "Standard Test Method, for Determination of Length Change of Concrete Due to Alkali-Silica Reaction", Philadelphia, 2018.

[6.20] Poyet, S., Sellier, A., Capra, B., Foray, G., Torrenti, J.M., Cognon, H., Bourdarot, E. "Influence of water on alkali-silica reaction: Experimental study and numerical simulations" Journal of Materials in Civil Engineering, Vol. 18, pp. 588-596, 2006.

[6.21] Grimal, E., Sellier, A., LePape, Y., Bourdarot, E., "Creep, shrinkage, and anisotropic damage in alkali-aggregate reaction swelling Mechanism-Part I: A constitutive model" ACI Materials Journal Vol. 105, n. 3, pp.227-235, 2008.

[6.22] ASTM Standard C 1074, "Standard Practice for Estimating Concrete Strength by the Maturity Method", ASTM International, West Conshohocken, PA.

[6.23] Thompson, G. A., Charlwood, R. G., Steele, R. R. Curtis, D. D., "Mactaquac generating station intake and spillway remedial measures". Proc. 18th International Congress on Large Dams, Durban, South Africa, Vol. 1, Q-68, R. 24, pp. 347 – 368, 1994.

[6.24] Multon, S., "Evaluation Expérimentale et Théorique des Effets Mécaniques de l'Alcali Réaction sur des Structures Modéles," PhD thesis, Université de Marne la Vallée, France, 2003.

[6.25] Saouma, V., "Numerical Modelling of AAR", CRC Press, 2014.

7 MANAGEMENT OPTIONS FOR CHEMICAL EXPANSION IN EXISTING DAMS AND HYDROELECTIC PROJECTS

7.1 Overall Strategies for expansive phenomena

No effective intervention methodologies are today available to stop the reactions and therefore to permanently repair the structures affected by internal expansion phenomena. These processes will continue until the reaction system reaches a balance point, for example, in the case of AAR, when the reactive aggregate is run out or the alkali hydroxides in the concrete are consumed and fallen below the threshold level of the reactive aggregate. Two only treatments could be envisaged, able to directly act at the level of ingredients that produce the expansive reaction, i.e. the concrete drying (eliminating water) or concrete carbonation (reducing pore solution alkalinity). But unfortunately, both are generally impracticable and particularly for dams. Furthermore, the injection of lithium compound into the concrete, in order to change the nature and behaviour of the gel from expansive to essentially non-expansive, and then to suppress the expansion, have been shown to be effective only in laboratory-based research, and perhaps in some application at concrete pavements but not at massive concrete in the field [7.1] [7.2].

This is the reason why the only practically feasible way to deal with existing reactions in concrete is to extend the dam service life, according to the two following options:

1) Options to slow down the deterioration process, reducing or mitigating the expansion rate, for example:
   * installing waterproof membranes on the upstream face in order to limit the provision of water for the reaction;
   * injecting superficial cracks on the upstream face with cementitious or polymer types of grout in order to limit water ingress;
   * making the drainage system more efficient;
   * cooling of concrete by reducing the temperature, for example through watering, even if in this case additional water is provided;
   * increasing the compressive stress by means of anchors or additional masses.
   The possibilities to mitigate the reaction in an existing dam are rather limited and their effectiveness quite uncertain.

2) Options to partially accommodate the effects of concrete expansions, treating the consequences of the expansion that affect the structural safety as well as the consequences of the expansion on dam operation.
   Possible measures to accommodate structural safety are:
   * grouting of the structural cracks with cementitious grout or epoxy resins to enhance the structural continuity, reduce leakage or increase sliding safety on the lift joints;
   * improving the drainage and/or the grouting curtains to reduce uplift pressure and seepage
   * sealing the upstream face with a membrane to avoid possible water inflow through cracks, causing flooding in the galleries in the galleries or increase the uplift pressure;
   * reinforcing using anchors to increase the sliding stability of the damaged dam
   * slot cutting to relief the compressive stresses in particular in the cases where the expansion may have concentrated impacts along structural discontinuities, such as transition between a gravity and a curved dam or near spillway openings .
   These measures, and in particular the use of waterproof membrane and slot cutting are also beneficial for managing the expansion consequences on dam operation (water tightness, gate jamming etc.).

Finally, a possible remedial work, which is rather to be considered for the long term, is the replacement of a dam by a new structure or the partial change of the structure (e.g. moving of a spillway).

The management of concrete dams affected by expansive phenomena involves an overall interpretation of the results of both laboratory/field investigations and structural assessment by mathematical modelling, as discussed in the previous chapters. Based on the conclusions of this overall interpretation, all available options and measures should be considered before deciding upon and implementing the suitable level of intervention, among those indicate below:

- No further action necessary;
- Continue regular monitoring and inspections (in-situ and periodical laboratory investigations);
- Perform minor or periodic small scale or maintenance repairs to minimize deterioration;
- Perform significant large scale restoration, including stress relief, strengthening, post-tensioning interventions to reduce or restrain the expansion process;
- Replace the damaged structure.

Additional monitoring specifically dedicated to capture the behaviour of the dam during and after repair interventions, whether they are minor or at a significant large scale, such as slot cutting, must be also considered. Guidance lines are provided in the following Chapter 8.

The evaluation of strategies here reported, mainly focused on the AAR expansion case, may be probably extrapolated to the other massive expansion phenomena considered in this Bulletin, such as ISA and DEF, even if the available experiences acquired at this time are still far too limited.

7.2 An Assessment of the Effectiveness of Management Strategies and Interventions

The management strategies and interventions considered included: anchors, grouting, slot cuts, coatings including membranes, gate and equipment modifications, monitoring and finite element modelling, and decommissioning or replacement. Examples of cases where each of the following strategies and interventions have been used are identified in Table 5. 1, "Matrix of cases versus structure types and management technique”.

The assessment of the effectiveness of the different options here reported is mainly focused on the AAR expansion cases. However it might be probably extrapolated to the other massive expansion phenomena considered in this Bulletin, such as ISA and DEF, even if the available experiences acquired at this time are still far too limited.

7.2.1 *Anchors*

Both post-tensioned high tensile and unstressed mild steel anchors have been added in many cases to: improve the integrity and stability of internally cracked concrete gravity, arch and buttress dam sections, tighten weakened horizontal lift joints and foundation contacts to improve stability, and repair damaged spillway piers.

Anchors have been used in powerhouses to repair damaged floor beams, generator enclosure walls, tighten second stage concrete around scroll cases and seek to retain downstream sections.

In all the cases presented, anchors appear to have been effective in terms of improving integrity and stability. In cases where the anchors are in expanding concrete, there has been a related stress build up in the anchors. In some cases this could significantly overstress the anchors and needs to be monitored, particularly in post-tensioned anchors. Unstressed mild steel anchors or dowels may be able to handle the additional strain plastically but this needs to be examined on a case by case basis. There is one known case in a gravity dam where a vertical post-tensioned tendon failed. Load cells and destressing options have been provided in several cases. The concern regarding stress build up also applies to spillway radial gate trunnion anchors. Attempts to estimate these stresses have been made in some cases.

Although in theory, the compressive stress applied by anchors will reduce the rate of expansion parallel to the anchors, aside from one quoted case in a power house which warrants further investigation, we have not seen any reports which show that the reduction in expansion rate by the use of anchors provides a significant long term expansion reduction benefit.

7.2.2 *Grouting*

Cementitious and epoxy grouting has been employed in many cases to help manage the effects of expansion by improving the structural integrity of cracked concrete and filling opened lift joints. It has been recognized that, in cases where the expansion is continuing, periodic regrouting may be necessary.

However, in case of cementitious injections, due attention should be paid to a possible increase of the concrete alkali content caused by the alkali provided by the injected cement, potentially able to act as a new source of alkali, with the risk of re-starting the expansion process.

In addition, at some high-altitude projects, grouting of surface cracking has been effective in limiting the progression of freeze-thaw damage.

7.2.3 *Slot Cuts*

The creation of expansion joints by cutting slots has been utilized to accommodate expansion in dams, spillways and powerhouses for many years. In dams, slot cuts are usually used to help reduce stresses at structural discontinuities, changes in alignment or interfaces between arch and gravity section. The first case we are aware of was in 1947 when slots were cut in a gravity-arch dam using overlapping drill holes and which effectively reduced the stress concentration at structure interfaces.

Overlapping drill hole slot cuts were used in a few dams and powerhouses in the 1960s and 1970s. However, these slots were wide and the opening was created rapidly and, in some cases, the associated displacements have been reported to have had negative impacts on nearby concrete and equipment. The use of twisted steel wires with silicon-carbide grit was tried in one project in the late 1980s with limited success. In order to develop a more controlled approach, the use of thin (10 mm diameter) diamond wire saw cuts was introduced in 1988 in a large gravity intake. The first slot was cut slowly to allow continuous monitoring and closed after the initial cut due to the pre-existing longitudinal compressive stresses. The slot was recut to yield an open expansion joint which was the objective at that time. Subsequently, thicker (13 and 15 mm diameter) diamond wire saws were used to cut successfully at other locations. This technique has been applied subsequently at several other dams, spillways and powerhouses in the USA and Europe, including an arch dam. Recutting the slots has been found to be necessary in cases where the expansion is continuing. The technique has evolved since 1988. In particular, it has been recognized that destressing the concrete invites an increase in the local expansion rate and a possible increase in the rate of deterioration of the concrete by allowing more micro-cracking. Recent projects have limited the recutting so that the slot remains closed to maintain some reduced compression in the expanding concrete and thereby maintain some restraint on subsequent expansion and reduce the frequency of recuts.

In some dam cases, several slots have been cut along the length of the dam to distribute the stress relief and minimize stress concentration buildup at the ends. In others, cuts have been localized to the area of stress concentration. Both approaches appear to be working. The need for distributed slots may increase with higher accumulated expansion strains.

Gate binding in some spillways has been addressed by cutting slots immediately adjacent to the openings. Again, the need for multiple slots to distribute the relief will depend on the magnitude of the accumulated expansion strain.

Expansion in powerhouses in the secondary concrete around the units has been shown to have caused “ovalling” of the air gaps and throat rings. The initial treatment in several cases has been to adjust stator supports and grind the throat ring steel. In cases where this option has been exhausted, thin slot cuts have been made between the units and these have partially restored clearances. This relaxation may have partially opened concrete joints with some leakage, but this appeared preferable to excavating and rebuilding the entire unit.

Wide slots have been cut in some powerhouses to separate diverging floor slabs and beams and have allowed alternative supports to be inserted.

However, the possible downside of the reduction of compressive stress accumulated in the concrete is that, after the slot cut, the expansion rate, is no longer restrained by the compressive stress state, may lead to locally increased expansion rates and associated structural movements.

7.2.4 *Coatings and Membranes*

Epoxy coatings to attempt to prevent additional water penetration has been attempted in a few cases. This has not been successful as the epoxy cracks with ongoing expansion. It may have some short-term benefit in limiting freeze-thaw penetration.

PVC membranes, and one with a resin laminate, with drainage curtains have been used on the upstream face of several large dams. These have been successful in limiting water pressure in cracks and lift joints, thereby improving stability. The membranes also limit leakage and associated leaching on joints. Of the cases considered herein, only one appears to show any success in controlling the reaction. That case is Pracana (Portugal) which is an extraordinary case for many reasons where the reservoir was dewatered for 12 years while extensive rehabilitations took place and it appears that the expansive reaction has been significantly attenuated as a result. It also appears that since refilling the reservoir, the expansive reactions have not so far restarted significantly in the concrete structures, quite probably due to the presence of the upstream membrane.

7.2.5 *Gate and Equipment Adjustments*

Spillway gate binding has occurred at a number of cases due to pier deformations into the spillway openings. The initial approach in most cases has been to shave the gates to increase clearances. When that has been exhausted, the seals and/or roller bearings have been modified. In most cases, there have been designed to accommodate forecasts of future expansion. In one case, a new rubber gate was installed with the capacity to absorb future expansion movements.

Powerhouse superstructures and crane rails have been found to become deformed in several cases. These appear to have been effectively handled by structural modifications.

Modifications to generator equipment have also been made and are usually complex and site specific. These include turbine shaft re-alignments, modifications to stator supports, wicket gate adjustments.

7.2.6 *Monitoring and Finite Element Modelling*

Precise monitoring is playing a key role in most projects, firstly to provide reliable data to confirm the existing behaviour, and when interventions are made to monitor the response. In addition, the data are used to calibrate finite element models to help understand the existing behaviour and condition, and then to forecast future behaviour and, when appropriate, to simulate possible remedial strategies and interventions.

In a number of cases this has been sufficient so far, and no structural or equipment modifications have been implemented.

In most cases where significant interventions have been made, elaborate monitoring and modelling have been an integral part of the assessment and design process. Various special purpose finite element models adapted to model the expansive non-linear time and stress material behaviour have been used.

7.2.7 *Decommissioning or Replacement*

Decommissioning and replacement has occurred in a small number of cases. This includes two of the cases listed. To the authors' knowledge there are perhaps four or five others. In most cases these decisions have been precipitated by seriously declining material properties. In one it was based on economic analysis considering the cost of ongoing maintenance and repairs, as well as the indirect costs of maintaining public confidence.

In several projects, estimates of remaining economic service life are being made. These are very dependent of forecasts of future behaviour including estimates of the future expansion and on damage accumulation.

7.3 References

[7.1] Fournier, B., Bérubé, M.A., Folliard, M., Thomas, M., “Report on the Diagnosis, Prognosis, and Mitigation of Alkali-Silica Reaction (ASR) in Transportation Structures”, Report FHWA-HIF-09-004, January 2010.

[7.2] Stokes D. B., “The role of lithium to mitigate ASR in existing concrete”. Presentation in the Workshop for Use of Lithium to Mitigate Alkali-Silica Reactivity, Transportation Research Board, 81st Annual Meeting, Washington, D.C., USA,13 January 2002.

8 MONITORING

8.1 Introduction

An approach to the management of concrete dams suffering from expansive chemical phenomena should include a detailed investigation program, with both laboratory and in-situ testing (Chapter 4), a suitable structural analysis, performed through a properly calibrated computational model (Chapter 6), and appropriate management actions to be taken (chapter 7). However, whichever solution is adopted, it is generally recognized that most of the remedial measures are temporary solutions that may help to extend the service life of the concrete dam, ensuring compliance with pertinent safety and serviceability requirements, until the deleterious expansive process has possibly reached its own stabilized condition.

This is the reason why a systematic long term assessment is always required, based on a specifically designed monitoring plan in combination with increasingly up-to-date structural models, in order to follow the real development of the phenomenon, to ascertain the effectiveness of the repairing strategies adopted and to produce more and more realistic predictions of the dam behaviour, so keeping the situation under strict control.

Relevant and effective instruments and techniques must be selected to suitably assess future extent, distribution and development rate of the damage associated to the expansive reactions underway and affecting both structure safety and equipment serviceability. The priority zone of the dam, of the repaired works and of the critical equipment must also be targeted, together with the relative timing over the years. Monitoring investigations are expected to be carried out periodically, at suitable time intervals, depending on the specific dam condition. However, since AAR and ISA/DEF are relative slow reaction, too shorter periods between investigations would not bring any additional significant information on the reaction development.

8.2 Monitoring plan

Large concrete dams are usually instrumented from the time of construction to monitor any relative movements that may affect their safe and serviceability limit states. Such large-scale monitoring systems should be maintained and eventually updated and suitably tailored to follow at best the specific dam behaviour, the effectiveness of the repair interventions implemented and, in case it should become necessary, their adjustments or corrections. Mapping of concrete dam sections through suitably techniques such as tomography, to investigate the distribution of concrete quality (mechanical properties, cracks and large defects or voids) could be part of a large-scale monitoring system.

The possibility to include into the monitoring plan also small-scale measurements should be taken into account, for specific purposes as, for example to monitor the stress state in some particular critical areas, to control the development of crack mapping in special zones, to check the width of particular cracks or the opening of new cracks, to verify the equipment clearance, to evaluate the seepage flow condition through the dam or at some critical parts, to measure temperature and humidity, to estimate the concrete mechanical properties or to verify the changes of alkali content of concrete and the progression of the reactions in specific regions (on drilled cores).

Data generated from large-scale deformations and displacements measured on the dams, as well as from small-scale monitoring and testing, have to be properly analyzed to determine the actual expansion rate and progression of the deterioration process, together with, the potential for future concrete expansion and ageing. These data must also be used to better calibrate the mathematical model for the assessment of the dam structural behaviour, suitably modified to include the adopted repair interventions. Unexplained rates of the dam movements or unusual results may be indices of structural degradation and should indicate a need for improvement surveillance, inspection, testing and implementation of extensive monitoring programs, to enhance the technical bases for assuring continued safe operation of the dam.

A suitable monitoring program, specifically focused on the expansion phenomena, should include, in addition to the usual routine measurements, some of the following requirements:

- eventual extension of the existing global or local monitoring, to include new critical areas or elements
- Tomography or other non destructive techniques
- Stress measurements
- Crack mapping on dam faces and inside the structure
- Evaluation of the Damage Rating Index on cores or other methods of damage assessment
- Evaluation of concrete mechanical properties and possibly the creation of a material properties Database such as, for example, the Ageing Concrete Information System" (ACIS) adopted by USBR (Chapter 3)
- Measurements of seepage flow and temperature
- Measures to verify the normal function of main equipment, in relation to the expansive process underway (gates binding, misalignment of gate embedded parts, generator and turbine runner alignment and clearances)
- A proper timing for each measure of verification
- Eventual alert thresholds for particular measures, the exceeding of which may require the implementation of intervention strategies, or structural analyses
- Dam and powerhouse personnel training to identify any changes and instructed to immediately report it to the manager.

8.3 Performance Parameters for Monitoring

Historical experience indicates that the following are the main general performance parameters of developing expansion issues in concrete dams and hydro-projects which can be considered when planning surveillance and monitoring programs in the management strategy.

- Ongoing upstream/downstream and vertical deformations
- Development of structural cracks
- Leakage at horizontal lift joints
- Development of map cracking at the concrete surface, in particular on secondary structural elements
- Spalling of crest, parapet walls and bridge decks at vertical joints
- Separation of embedded parts, gate guides, etc.
- Changes in leakage at abutments

These general parameters need to be tailored to the needs different dam types. The following key structure specific indicators may be considered:

Arch Dams

- Upstream movements at crest
- Diagonal cracks on downstream face parallel to dam/abutment contact
- Structural cracks in galleries

- Signs of separation or slip on horizontal lift joints
- Excessive leakage affecting uplift pressures and leaching of joints

Gravity Dams

- Separation or slip on horizontal lift joints
- Cracks at structural discontinuities, changes in geometry
- Excessive leakage affecting uplift pressures and leaching of joints

Buttress, hollow gravity and Ambursen Dams

- Cracking at structural connections of slabs affecting water tightness
- Cracking and deterioration of buttresses
- Cracking and deterioration of internal wall of hollow gravity dams

Intakes

- Gate clearances and distortion of hoists
- Diagonal cracking in water passage piers
- Racking deformations and cracking of piers

Powerhouses

- Signs of 'ovalling' of units affecting runner and generator air gap clearances
- Loss of shaft alignment, head cover bolts misalignment
- Stress buildup in penstocks due to compression or shear effects
- Distortion or cracking of stay ring, wicket gates, liners, etc
- Diagonal cracking of draft tube piers
- Interference with draft tube gate clearances
- Distortion, separation or overstressing of embedded parts
- Leakage
- Misalignment of crane rails
- Distortion of superstructure frames

9 CONCLUSIONS & RECOMMENDATIONS

9.1 Conclusions

There are fundamentally two possible approaches to managing existing reactions in dams and hydro projects:

- Attempt to control and moderate the rate and duration of the expansive reactions, and
- Options to live with the expansion and take steps to treat the consequences of the expansion to extend the useful service life for as long as possible.

Both approaches typically involve ongoing monitoring and management.

9.1.1 *Controlling the Reaction*

There are a number of instances where membranes or coatings have been added to the upstream face, with good benefits regarding leakage and uplift control in cracked dams, but no cases are documented where this has measurably controlled the reaction and associated expansion rates, except for the Pracana case which is a special situation which is unlikely to be considered as a management option at other dams. Due to the massive sizes of the structures and the difficulty and slowness of removing water from the concrete, if it can ever be achieved the time required to dry the interior concrete to a degree that would affect the reaction, probably is in many hundreds of years, rendering this option currently impractical. This is the case even without water ingress, the residual water in the interior concrete after hydration will usually be sufficient in most cases to drive the reaction for many years. However, there are cases which show that such membranes can provide a water barrier effectively for long periods and can be effectively replaced if and when necessary.

Similarly, post-tensioned anchors have been installed in several cases, improving stability and structural integrity of the structure, but no cases where they have reduced the expansion rates are documented. Numerous laboratory and modelling efforts have shown that the expansion rate is reduced by compressive stress. However, for this to significantly reduce the rate of expansion, compressive stresses in excess of about 5 MPa are required to be present in the bulk of the structure. Except in smaller components, e.g. bridge beams or thin buttresses perhaps, the amount of post-tensioning steel required is impractical.

Although some limited success in concrete pavements have been reported, no successes in dams using chemical means, such as lithium salts (the effectiveness of which is dependent on reaction type) or carbon dioxide are known, due to difficulties in achieving extensive uniform penetration in massive structures.

In summary, despite the problems, the one approach to control AAR that may warrant further research may be to seek ways to accelerate the drying process by comprehensive sealing and long term treatment.

9.1.2 *Living with the Expansion*

The objective in most cases will be to maximize the remaining economic service of the project. The rate, magnitude and duration of the residual expansion will have been a key factor in managing the remaining service life of a dam or hydro project, even if it is very difficult to estimate in the cases where expansion is continuing. The earlier notion that with AAR the alkali source is the cement is now realized to often be only part of the long-term story; in many cases additional alkalis become available from certain aggregates or SCMs, or from a recycling process in the alkali-silica gel with time. This will clearly affect the duration of the reactions and in many cases, could cause the reaction to continue indefinitely.

A number of effective strategies to extend the life of projects with existing expansive reactions have been discussed. Only in cases where the reaction is ceasing or has ceased, can these be considered final solutions. In most of the cases, the reaction is continuing, and the strategy has to be one of ongoing management. In some cases, no immediate interventions are planned, the strategy is to monitor the evolution of the phenomena and the condition and safety of the project. In other cases, significant interventions have been made and in many of these, repeated interventions will be required.

It is usually appropriate to maintain an AAR focussed diagnosis program including in-situ and laboratory testing and a surveillance and monitoring program for several years to provide a firm basis to develop and confirm the effectiveness of the management plan. Such a plan will usually require structural modelling to understand the behaviour, identify key issues and investigate the effectiveness of possible management strategies.

Structural modelling of expansive behaviour requires specialized expertise and software to be effective. The chemo-mechanical processes involved in expansion of concrete are complex and current models attempt to include the key factors such as moisture, temperature, stress restraint and creep to varying extents as necessary to suit the application. Thorough calibration of the models needs to be thorough and can use laboratory test results as a starting point but usually will have to rely on matching deformations and in-situ stresses before being used for forecasting and remedial measures studies. Typical “Potential Failure Modes” need to be identified to facilitate their consideration in the dam safety assessments and monitoring.

The use of anchors and grouting to help maintain the integrity and stability of the concrete structures has been found to be effective option to help maintain safety and integrity in many cases.

Slot cutting has been effectively used in many cases in dams and powerhouses to control stresses, concrete cracking and equipment misalignments and clearance interferences. Current saw cutting techniques, combined with comprehensive monitoring and modelling, allow this to be done in a controlled way to minimize undesirable side effects. Multiple slot recutting has been demonstrated to be feasible and effective although there are no doubt limits to the number of times this can be done before concrete quality deteriorations or associated deformations in adjacent concrete and equipment become unacceptable. Slot cutting intervention is often coupled with the use of waterproofing membrane system to assure protection from water intrusion after slots have been accomplished, control uplift pressure in cracked dams and minimize deterioration. There are cases which show that such membranes can operate effectively for long periods and can be effectively replaced if and when necessary~~.~~

A wide range of spillway gate and powerhouse equipment adjustments and modifications have been developed and successfully implemented. Slot cutting has also helped maintain gate clearances in a number of cases.

A number of projects are in a monitoring stage, where they can be safety operated without major interventions at this time. These also rely on comprehensive monitoring and modelling to assist in understanding the current status and forecast future requirements. It is very important that the instrumentation and monitoring program is robust and will be sufficiently accurate and reliable for long time periods.

In the case of sensitive structures where the expansive behaviour will continue indefinitely, possibly due to supplementary alkali supplies from aggregates or recycling, there may be limits to sustainable life extension. However, to date, only a very few structures have had to be abandoned or replaced.

9.2 Recommendations

There is a clear need for ongoing research and development: 1) to better understand the expansion phenomena, especially their long term kinetics, 2) to optimize the laboratory testing, particularly in respect of the future potential expansion, in presence of processes such as alkali release from aggregates and alkali recycling; 3) to identify reliable procedures to correlate laboratory expansion test results to field conditions for concrete dams in services; 4) to model the structures recognizing the possibility of reactions continuing indefinitely and not slowing down according to an “S” shaped curve of expansion with time and 5) to seek more definitive management strategies.