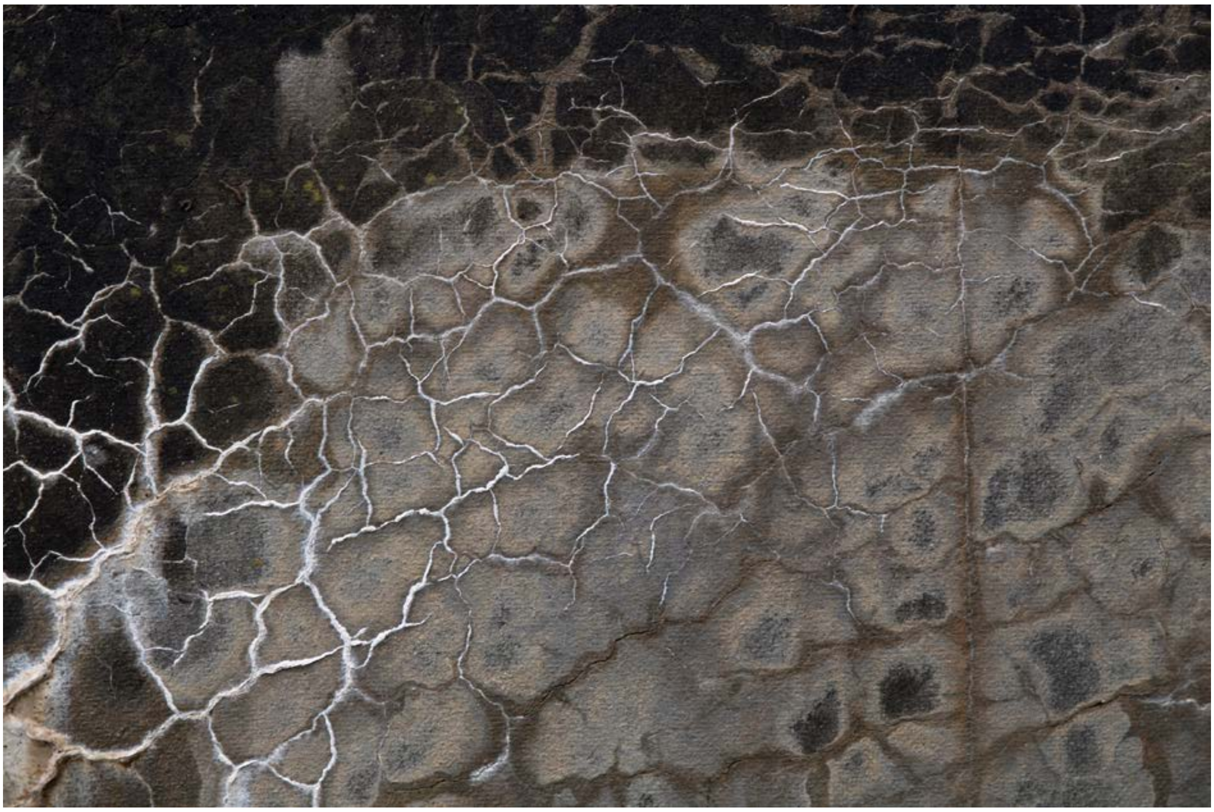
Schweizerisches Talsperrenkomitee
Comité suisse des barrages
Comitato svizzero delle dighe
Swiss Committee on Dams



CONCRETE SWELLING OF DAMS IN SWITZERLAND

08.05.2017

Report of the Swiss Committee on Dams on the state of concrete swelling in Swiss Dams

AAR Working Group **May 2017**

Index ***Page***

In 2013, the Swiss Committee for Dams (SCD) decided to set up the "Alkali-Aggregate-Reaction" working group with the objective of investigating the current situation of Swiss dams in relation to this phenomenon.

The present report has been prepared by the AAR Working Group and has been approved by the Technical Commission (TECO) of the Swiss Committee for Dams on 15.11.2017.

Members of the *AAR Working Group*:

Mr. Francesco Amberg	Lombardi SA, Minusio
Mr. Patrice Droz	Stucky SA, Renens
Mr. Raphael Leroy	Alpiq SA, Lausanne
Mr. Johannes Maier	Federal Office for Energy, Ittigen
Mr. Bastian Otto	Axpo Power AG, Baden
Mr. Roger Bremen	Lombardi SA (President of the working group)

Title page picture: Image of cracking on the surface of an AAR-damaged structure

1 Introduction and study objectives

1.1 Introduction

The Alkali-Aggregate Reaction (AAR) affecting many civil engineering infrastructures is undoubtedly one of the major challenges for today's engineers. The problem is particularly complex for dams usually because of their large size, as well as any economic implication, amongst other factors, of any major intervention.

In order to better understand the extent of the problem in Switzerland, the Swiss Committee for Dams (SCD) commissioned a Working Group (WG) in 2014 in order to study and establish a review of the current state of Swiss dams. The study provides some indications on the measures to be taken in order to control structures affected by concrete swelling (swelling caused in particular by Alkali-Aggregate reaction AAR or by a sulfatic reaction ISA).

This report summarizes the main findings of this study, which provided a relatively comprehensive framework for Switzerland. The SCD expresses its gratitude to the members of the Working Group for their time devoted to this study. Gratitude also goes to the owners and operators of the hydroelectric facilities, for their collaboration and availability in providing information on the behavior of their respective structures.

1.2 Objectives

Concrete swelling, associated with the Alkali-Aggregate reactions in particular represents without doubt the major problem affecting concrete dams worldwide. If still 30 years ago the international community was convinced that the problem was confined to certain regions of the planet, it is now accepted that the phenomenon affects all continents, and that its magnitude has been underestimated for too long.

Countless studies to better understand the different aspects of the problem have been carried out or are currently underway. All the way from the chemical analysis of the phenomenon, to numerical simulations of its influence on the stress distribution in dams, these different studies each provide additional elements to further understand the complexity of the phenomenon. It therefore cannot be expected of the WG to tackle this subject exhaustively across the full spectrum of influencing factors.

The overall objective of the study is therefore to establish a general framework to assess the situation of the phenomenon in Switzerland, and to provide operators with some indications on the measures to be taken when confronted to a structure that seems to be affected by concrete swelling. It is indeed important that the measures implemented are appropriate, both in terms of scale and time frame. Inappropriate measures may lead to unnecessary expenses, or to deterioration of the operating conditions and / or safety of the dam. The timing of any intervention is essential in order to preserve the structural integrity of any dam to the greatest extent possible. In other words, this report aims to provide support to operators in effective decision - making when considering the question "when should we do what".

The study comes to a conclusion with forecasts on the influence of this phenomenon for the Swiss dam scene during the next decades.

Except for a general topic introduction, this study does not address the causes and multitude of consequences of AAR on concrete and on dam structures. Although analyses of the various factors that trigger and maintain these reactions and their influence on swelling progression constitute the base of understanding the phenomenon, their description only would already be beyond the scope of the present study. The WG also chose not to consider the normative framework and its limitations, particularly in the area of laboratory testing. This subject also deserves to be covered in order to avoid that past mistakes occur again.

Finally, the WG preferred not to specify all dam names in the publication. As a result, the specific content of the database developed in the context of this study will not be disclosed. It is planned to update its indicative content every 10 years in order to be able to follow the medium-term evolution of the problem.

The content of this report was the subject of a workshop which took place on 13th May 2015 in Bern. The presentations given during this workshop are available at www.swissdams.ch/index.php/en/publications/workshops-2/workshop-2015 .

1 Consequences of concrete expansion at dams

1.1 Introduction

The damaging process of the alkaline aggregate reaction (AAR) was first discovered in 1940 by T. Stanton on concrete driveways, bridge girders, and retaining walls in California [1]. He reported damages caused by a chemical reaction within the concrete itself. Similar claims were later received from other parts of the USA, from Denmark, the Netherlands, Germany, England, Iceland, South Africa, France, and Japan. In Switzerland, AAR related damages were first described in 1988 where a dam was concerned.

AAR is a chemical reaction in the cement between the aggregates and the alkali. This results in AAR generated products (i.e. crystalline products in the aggregates and AAR gels in the pores) which increase in volume due to water absorption, and thus lead to concrete swelling. This can ultimately cause cracking. As a result of AAR, the mechanical properties of concrete also deteriorate [2]; in practice this is normally not problematic with dams.

Figure 1: Old Sera Dam damaged by AAR at Gondo in Zwischbergental (VS).

1.2 Alkali-aggregate reaction prerequisites

In order for AAR to occur in concrete, three conditions must be met simultaneously, as shown in **Figure 2** on the next page:

- The concrete contains reactive aggregate components.
- There is enough moisture in the concrete.
- Alkali are present.

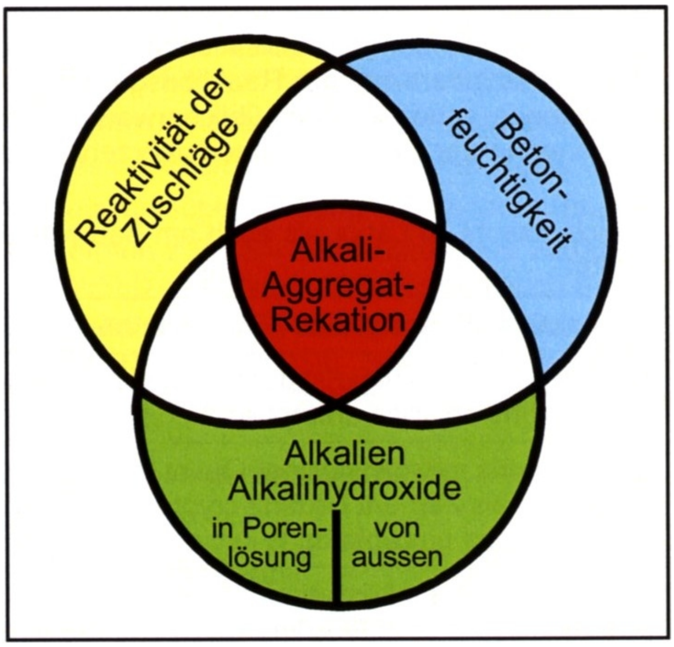


Figure 2: Concrete AAR prerequisites [3].

The alkalis are primarily OH, Na +, K + and Ca2 + ions. If the moisture content of the concrete is below 80%, AAR will not occur. However, the moisture content is always higher in concrete dam constructions, usually close to 100%. AAR is favored by moisture changes through which transport of the alkali is increased. This is often the case for downstream dam faces exposed to air, the crest, and the uppermost part of the upstream face.
Elevated temperatures accelerate AAR. A temperature rise from 10 to 40°C can increase the speed of AAR 10 fold. Therefore, sunlit, near-surface dam areas react much faster than those in shaded areas or within the dam structure. This is also a reason for occasional very severe AAR damage in tropical area dams. The increased rate of AAR reactions through elevated temperatures is also used in laboratories to accelerate experiments.
If the aggregate grains belong to the following reactive rock types, AAR may occur [4]:

Rock family	*Rocks in which reactive mineral phases can occur*	*Reactive mineral phase*
Crystalline rocks	granites, granodiorites, diorites etc	• Porous microfiber quartz
Volcanic rocks	rhyolites, dacites, andesites, basalts, obsidians, tuffs	• Unstable high-temperature forms of quartz: trydimite, cristobalite • Crypto-crystalline silicic acid: chalcedony • Amorphous, hydrated silicic acid: opal
Metamorphic rocks	gneiss, schist, mylonite, quarzite, hornblende	• Deformed, porous, weathered feldspars • Fine crystalline mica • Crypto and microcrystalline quartz
Sedimentary rocks	sandstones, greywackes, siltites, flints, pebbly limestones	• Deformed, porous, weathered feldspars • Fine crystalline clays, mica • Crypto and microcrystalline quartz • Crypto-crystalline silicic acid: chalcedony • Amorphous, hydrated silicic acid: opal

An investigation of aggregates from 79 mining areas used for the production of concrete (about 25% of all production sites) has shown that about 85% of all aggregates in Switzerland are classified as reactive (see **Figure 3**). The classification was carried out with the help of the microbar experiment [11]. Rocks were rated as potentially reactive if a microbar value above 0.11 (i.e. strain greater than 0.11%) was found.

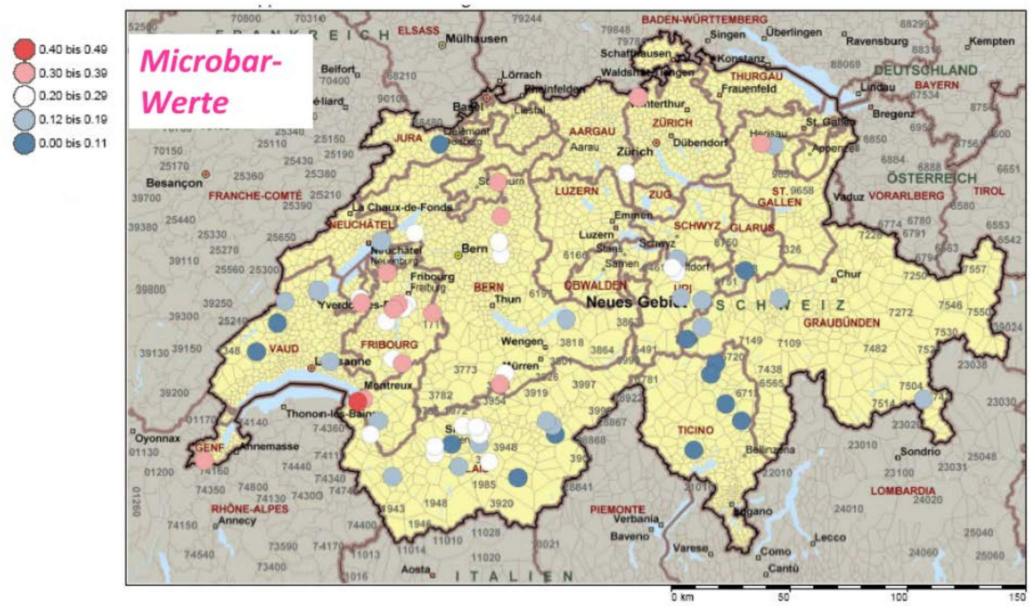
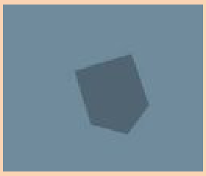
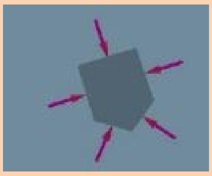
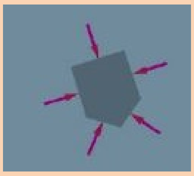
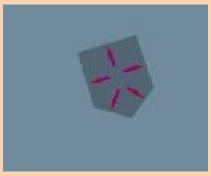
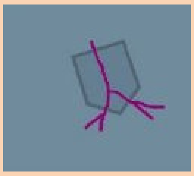
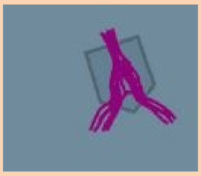


Figure 3: Assessment of the reactivity of Swiss aggregates using the microbar value (microbar value above 0.11 → aggregate is reactive) [5].

1.3 Damage mechanism of the alkali-aggregate reaction

Using the example of the alkali-silica reaction, the AAR damage mechanism can be described in detail in six steps [4]:

1 	2 	3 
Aggregate grain with alkali-sensitive silica SiO2	Surface reaction with alkali and calcium ions of cements containing non-swelling alkali calcium silicate hydrates	Diffusion of alkali ions and water inside the grain and reaction with alkaline-sensitive silica to form alkali silica gel
4 	5 	6 
Internal pressure increases due to reaction progression and water absorption	Cracking when exceeding the tensile strength of the aggregate: low gelation	Dissolution of the aggregate from within: strong gel formation

The macroscopic effect of AAR is expressed in concrete structures as follows:
The elongation of the concrete increases by 20 to 150 μm / m per year (the higher value applies to structures in the tropics):

- A 100 m high dam wall grows by 2 to 15 mm per year.
- A 250 m long dam crest expands by 5 to 40 mm per year.
- If expansion is obstructed, restraining forces or constrained cracks arise.

The concrete surface changes:

- The result is mesh or radial cracks (map cracking). At high pressure load the crack course is parallel to the pressure trajectories.
- Gel droplets form, which are initially clear or dark colored. In contact with atmospheric carbon dioxide, they turn milky and white after dehydration.

Shattering of near-surface aggregate grains (pop outs) occurs. As a result of cracking, the concrete mechanical properties diminish. For highly advanced AAR, laboratory examinations have shown the following reductions in material strength:

- Compression: The compressive strength is reduced by 25 to 60%.
- Tensile strength decreases by 50 to 70%.
- Elasticity decreases by 60 to 70%.

AAR reactions affecting dams in Switzerland are still at an incipient stage. The above measurements therefore only play a minor role and may at present be neglected.

Since 2001, the Swiss Federal Office of Energy (SFOE), together with Swiss Electric Research, has partly funded several research projects on the AAR damage mechanism in concrete dams. The École Polytechnique Fédérale de Lausanne EPFL, has supported works led by professors Karen Scrivener [6, 7, 8] and Jean-François Molinari [9]. Within the framework of kinetic investigations, the influence of parameters such as temperature, alkalinity, and size of aggregate grains on chemical reactions and their resulting macroscopic effects on concrete (especially swelling mass and mechanical properties) was studied. In addition, studies on the effect of the external conditions (in particular creep and deformation damages) on concrete behavior were investigated. **Figure 4** shows the AAR swelling test setup for concrete cylinders under 3d-compressive loading and the apparatus for the 3d-load cell for taking fiber-optic strain measurements. **Figure 5** shows the AAR ranges determined by numerical 3D computer simulations for three different load scenarios.

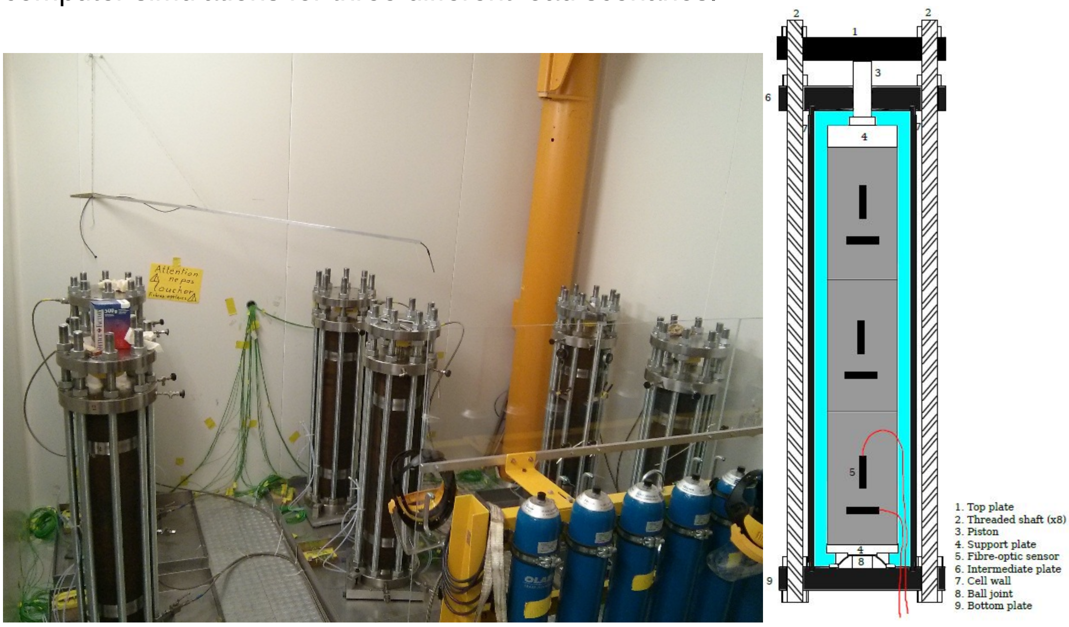


Figure 4: Test rig for AAR swelling simulations on concrete cylinders with 3d compressive load (left) and schematic of the 3d load cell with fiber optic strain measurement (right) [8].

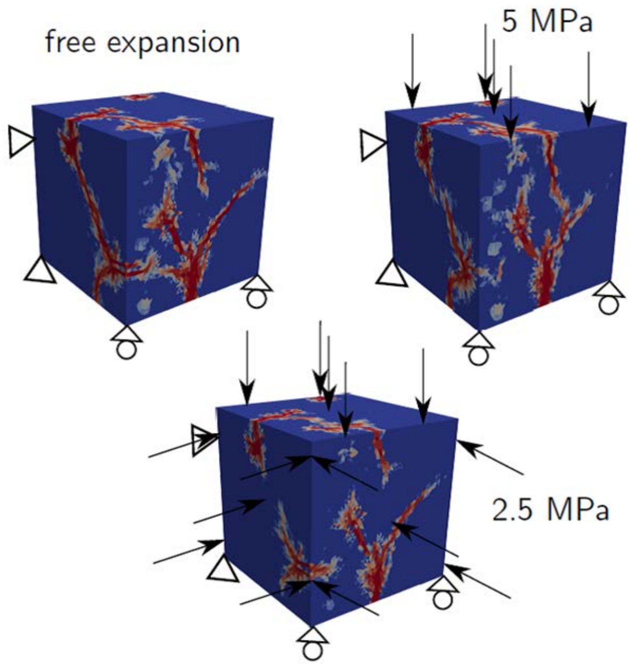


Figure 5: Load influence on AAR areas (red and gray) due to numerical 3d-computer simulation (top left without external load, top right with single-axis compressive load, and below with three-axis compressive load) [9].

1.4 Determination of alkali-aggregate reactions in concrete stocks

A progress report on the diagnosis of AAR in dams was prepared by the EMPA at Dübendorf under a research mandate from the SFOE [10]. The report deals with visual inspections of dam structures and concrete samples, the use of light and electron microscopes, as well as spectroscopy for AAR identification.

1.5 Dam structure inspections and measurements

AAR assessments of existing dams are primarily based on controls and measurements carried out directly on site:
Visual controls:

- Identification of cracks that may have gel deposits or containing moisture.
- Opening / closing of the vertical block joints on the air / upstream face.

Functional checks of the of the mechanical discharge system:

- Does the hydraulic system require higher oil pressure than at previous controls?
- Do movements of control gates take more time than during previous controls?

Geodetic measurements:

- Does triangulation give rise to an increasing drift over time (mostly in the upstream direction)?
- Do the crest and inspection tunnels levels show increasing rise over time?

Pendulum measurements in dam concrete:

- Is there any increasing drift over time (mostly in the upstream direction)?

- Extensometer measurements in dam concrete:
- Are increasing extensions detected over time?

Secondary structures (storage/guard houses, retaining walls, etc.) often consist of concrete with higher cement content but with the same additives. They are usually exposed to lower loads. AAR damages can therefore often be detected earlier in these secondary structures than at the dam structure itself.

1.6 Visual inspection of concrete core samples

For laboratory tests, cylindrical core samples can be taken from the structure using a core drilling machine. AAR damage can often easily be recognized with the naked eye on such samples (see **Figure 6**, left and **Figure 7**). For examinations under the microscope, smaller samples must be taken from the sample cores and processed as thin sections (see **Figure 6**, right).

Figure 6: Surface of a drill core d = 100 mm with cracks due to AAR (left) and concrete sample 50 × 90 mm with impregnated thin section / thin section for observation under the light microscope (right) [10].

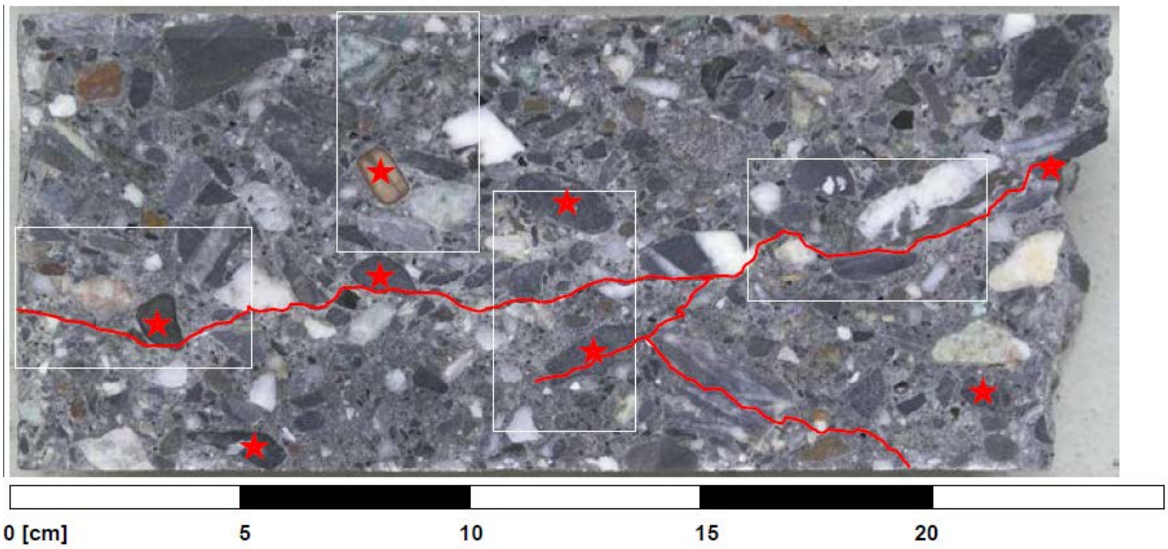


Figure 7: Longitudinal cross section of a drill core with visible cracks (red lines) and areas selected for thin sections (white rectangles) [10].

1.6.1 Observations under the light microscope

AAR generated micro-cracks and associated gel precipitations are easily recognizable under the light microscope. Observations can be further improved with fluorescent light or with the help of polarizing filters (see **Figure 8** and **Figure 9**).

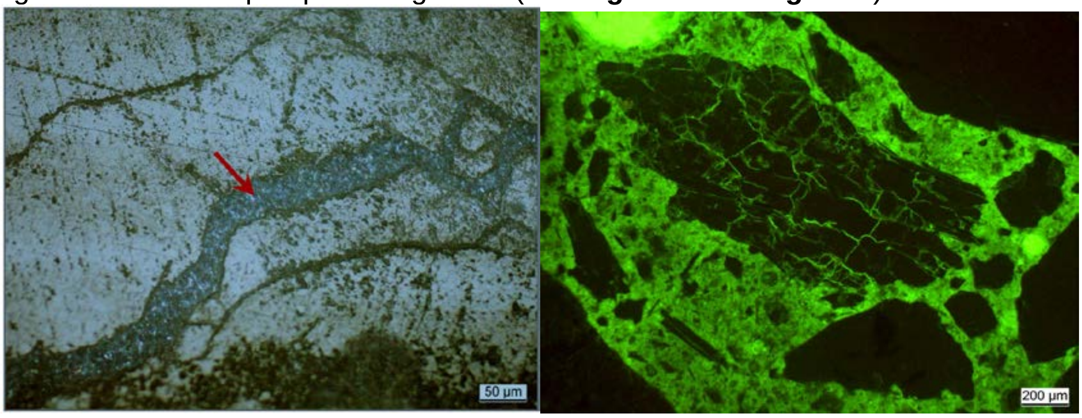


Figure 8: Micro crack filled with AAR-generated products in an aggregate grain (left, microscope image of a freshly ground moist concrete surface before impregnation) and aggregate grain containing multiple micro-cracks (right, microscope image with fluorescent light) [10].

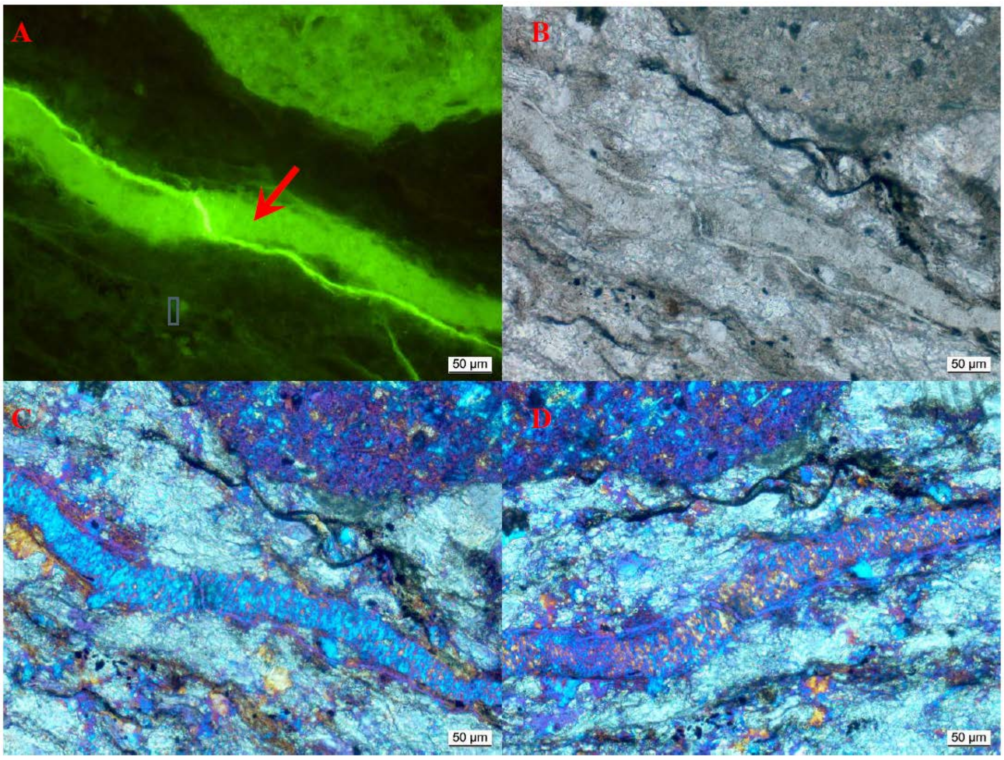


Figure 9: Thin section image of a crack completely filled with AAR-generated products (red arrow) in an aggregate grain under four different light conditions (A = fluorescent light, B = polarized light, C = crossed polarizing filter and plasterboard, D = same as C but rotated by 45°) [10].

1.6.2 Electron-microscope and spectroscope observations

The electron microscope enables greater magnification. This allows for a more detailed analysis of AAR generated products in cracks (see **Figure 10**). With the aid of energy dispersive X-ray spectroscopy (EDX), a quantitative analysis of these AAR generated products can be performed (see **Figure 11**).

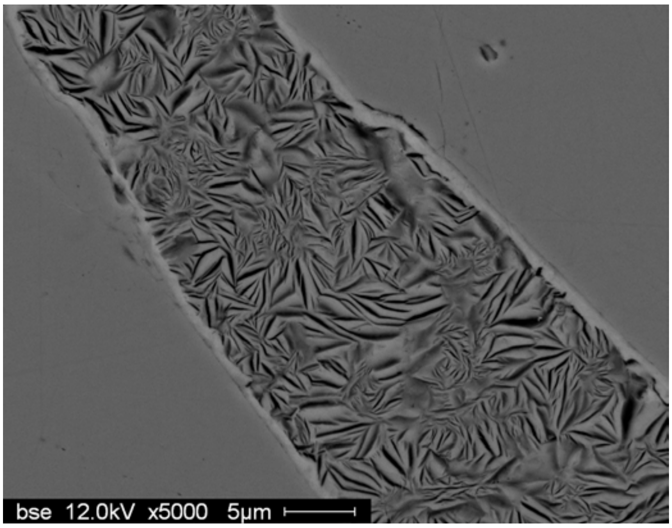


Figure 10: Electron micrograph (REM // SEM) image of a micro-crack filled with AAR generated products in the gneiss aggregate sample of a dam [10].

Element	O	Na	Mg	Al	Si	S	K	Ca	Fe	Ca/Si	(Na+K)/Si
	[Mol-%]									[-]	
BL11-2	40.5	1.9	0.3	0.9	36.5	0.3	8.4	10.4	0.7	0.28	0.28
BL16-2	40.5	1.5	0.4	2.5	36.0	0.3	8.2	9.7	0.7	0.27	0.27
BL23-6	43.3	2.0	0.3	0.8	34.9	0.4	7.6	9.7	0.6	0.28	0.28

Figure 11: Result of three quantitative material analyses of AAR generated products with energy dispersive X-ray spectroscopy (EDX) [10

1.7 Typical dam and secondary structure damage patterns

The following illustrations from **Figure 12** to **Figure 16** show AAR damage in various Swiss dams and their secondary structures.

Figure 12: Web-shaped AAR cracks at a spillway (left) and cracks at the inlet of a spillway with prevailing crack direction due to existing forces (right).

Figure 13: AAR cracks with strong air-flow efflorescence at downstream face of a stepped gravity dam (left) and shore protection in the underwater area of a weir with clearly shaped AAR cracks (right).

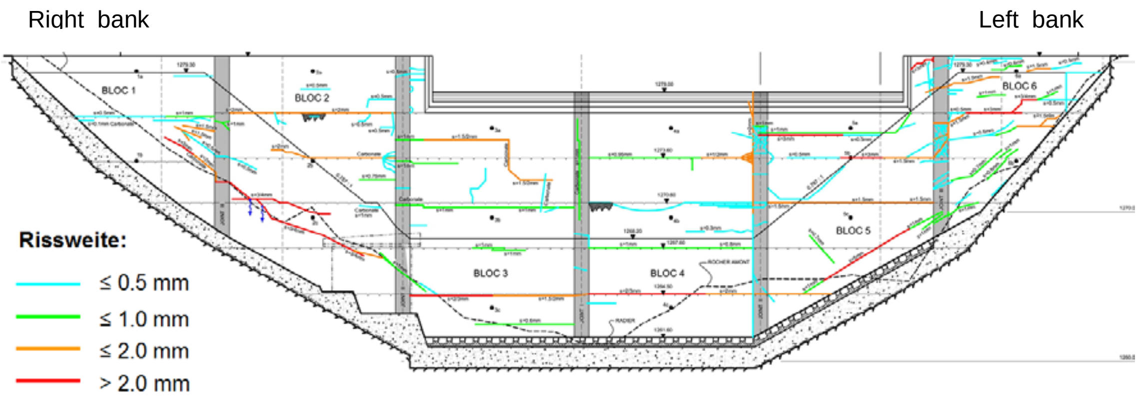


Figure 14: Concrete expansion in the old Sera Dam. Fissure pattern (structural cracks) is very similar to the one at the Zeuzier arch Dam (cracks following valley shape on the downstream face).

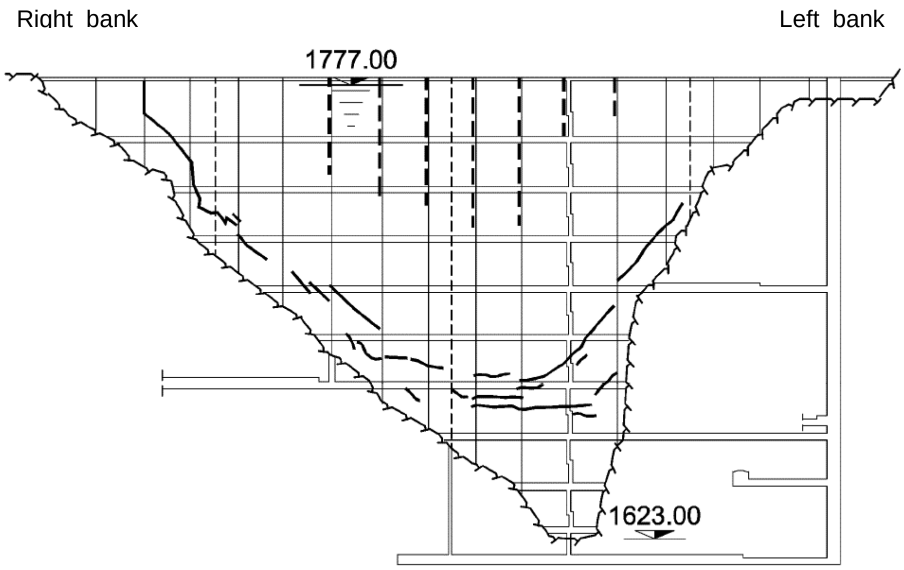


Figure 15: Crack pattern at Zeuzier arch Dam following valley closure

Figure 16: Support of a reinforcement bar with AAR cracks in the main vertical stress direction (left) and retaining wall of a surface spillway with gaping crack (right).

1.8 Safety impairment

As shown in **Figure 17** to **Figure 19** below, the safety of a dam due to AAR can be impaired in several ways (see also [13]). Blocking of gates can occur, geodetic measurements are interrupted, and structural safety may no longer be sufficient.

Figure 17: Gate mechanisms can no longer be operated due to AAR generated swelling (left) and geodetic monitoring might be interrupted due to AAR rising of the railing on the dam crest (right).

Figure 18: Determination of the depth of a structural crack on the downstream face of an AAR affected arch dam by drill core removal (left), and crack with filling on the upstream face of an arch dam.

As a result of cracking of the AAR-damaged concrete dam, the following questions must be addressed:

- How much of the cross section is still available for sustaining load stress?
- Can the applied forces still be absorbed?
- In the case of arch dams, is it still possible to redistribute stress from the vertical effect to the horizontal arch effect?

Figure 19: Provisional securing of an AAR-damaged dam crest with vertical tensioned anchors, and steel side straps in case of floods and earthquakes.

1.9 Forecasting the evolution of the alkali-aggregate reaction

To assess the future development of AAR-damaged concrete, the residual swelling mass i.e. the remaining expansion potential determined on cores taken from the structure is used. For this purpose, the cores are provided with longitudinal and transversal strain gauges and then stored in a moisture tight container with water up to the top of the core at controlled temperature (**Figure 20**).

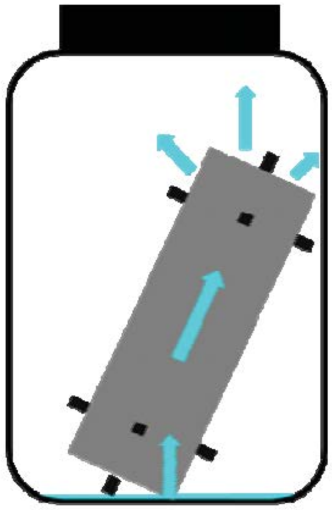


Figure 20: Cores for AAR residual swelling determination [5].

The results of the repeated strain and mass measurements are recorded in a diagram. As can be seen from **Figure 21**, three phases must be distinguished:

– Conditioning phase during which the sample is subjected to moisture and temperature expansion.

– Phase during which non-linear, then linear AAR expansion strain is applied at constant temperature and humidity.

– Drying phase with shrinkage of the sample.

AAR residual swelling mass tests can often last for more than a year!

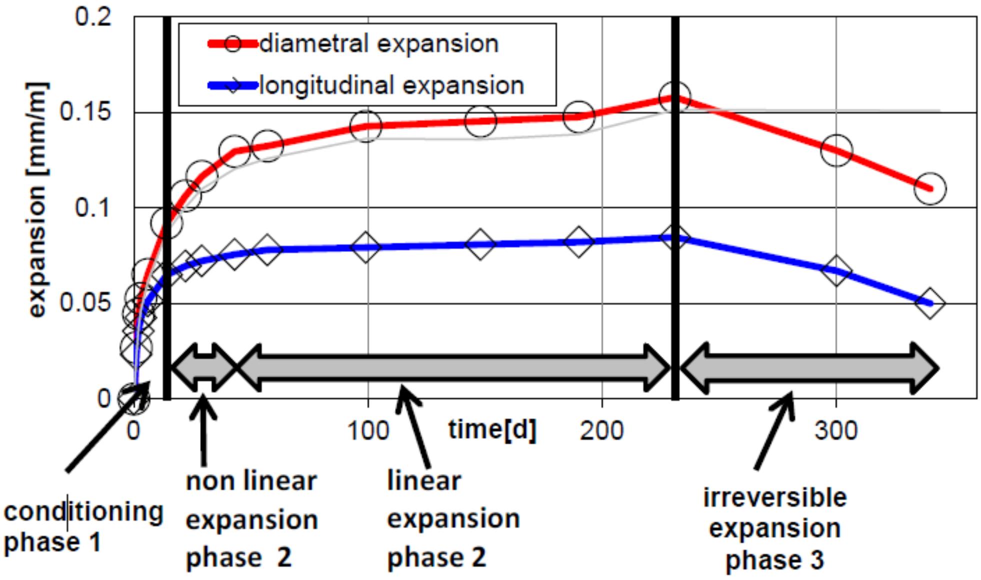


Figure 21: Diametral expansion (= red curve) and longitudinal expansion (= blue curve) [10].

1.10 Avoiding AAR in new constructions

When constructing dams, future damages due to AAR can be avoided based on **Figure 2**,

- if no reactive aggregates are used in concrete production,
- water is kept away from the concrete whenever possible,
- if the alkali content of the cement is kept as low as possible.

The alkali content of the cement is primarily determined by choice of the cement type used. It can be further reduced if silica dust, fly ash or blast furnace slag is used as partial replacement of the cement.

In theory waterproofing and drainage can help in keeping water out. In dams, however, it is usually not possible to bring the water content below the critical 80% necessary for AAR.

With regard to additives, extensive, sometimes very long material trials should be carried out before using them in new constructions:

- Petrographic examination of additives: thin sections should be examined under the microscope.
- Determination of the reactivity of the aggregate using the microbar experiment: For this purpose, the aggregate is first broken and finely ground. Thereafter, with the 0.16-0.63 mm sieved grain group, sodium hydroxide and cement CEM I 42.5 are produced in three mixing ratios 10 × 10 × 40 mm. It is then placed 4 hours in humid storage, and 6 more hours with potassium at 150 °C. The increase in length can then be measured. If the increase in all three mixing ratios is below 0.11%, the aggregate may be said to be non-reactive. The microbar test cannot be used for reactivity testing of aggregates of predominantly metamorphic or crystalline rock.
- Carrying out the concrete performance test: Test samples are prepared from the concrete mix intended for the new construction with the addition of sodium hydroxide. The samples shall be stored under controlled conditions throughout the duration of the experiment. Tensile and mass determinations are performed on test samples on a monthly basis. In order to obtain meaningful results on AAR behavior, performance tests on concrete need long test periods, often requiring one year or more.

Valuable information concerning the tests above and for the prevention of AAR generated damage in new concrete structures can be found in the SIA 2042 [11] published in 2012, or the publications of the RILEM [12] specific to this topic.

During the construction of the Sera Dam, which took place between September 2009 and December 2010, the non-reactive aggregate supplement for the concrete was transported from a quarry near Brig (VS) by lorry over the Simplon Pass to the construction site in the Zwischbergen valley near Gondo VS.

Figure 22: Newly constructed Sera Dam in Zwischbergental near Gondo (VS) with non-reactive aggregate concrete [17].

2 State of AAR in Swiss dams

2.1 Data structure basis

2.1.1 Swiss dams

The dam supervision section of The Federal Department for Energy (SFOE) is responsible for dam safety in Switzerland, and has provided the study database for Swiss dams. This database contains the name, altitude, storage volume, class, type, and year of construction of each dam. In Switzerland, 154 concrete walls and 85 dams are under federal supervision. Small dams, generally less than 10 m high and with a storage volume of less than 500,000 m^3 are under cantonal responsibility.

In this report, only the 154 concrete dams subject to federal supervision will be considered. These are subdivided into 3 classes depending on the height H and the storage volume V according to **Figure 23**:

- Class I: 74 dams (50%)
- Class II: 34 dams (25%)
- Class III: 35 dams (25%)

In the database 11 dams and weirs are not classified.

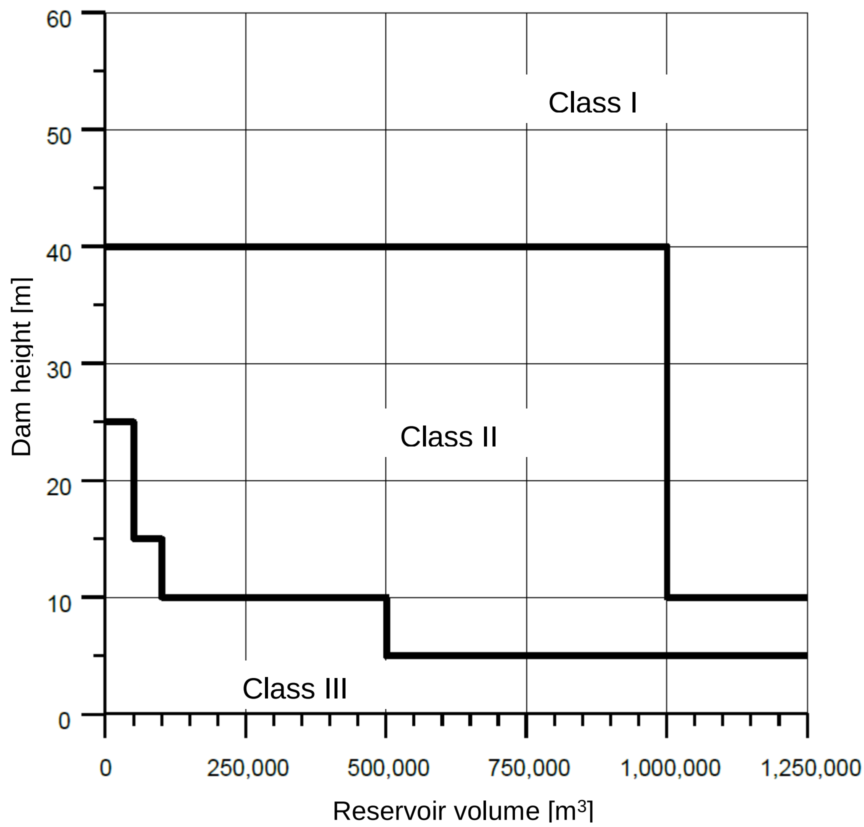


Figure 23: Definition of the three different dam classes.

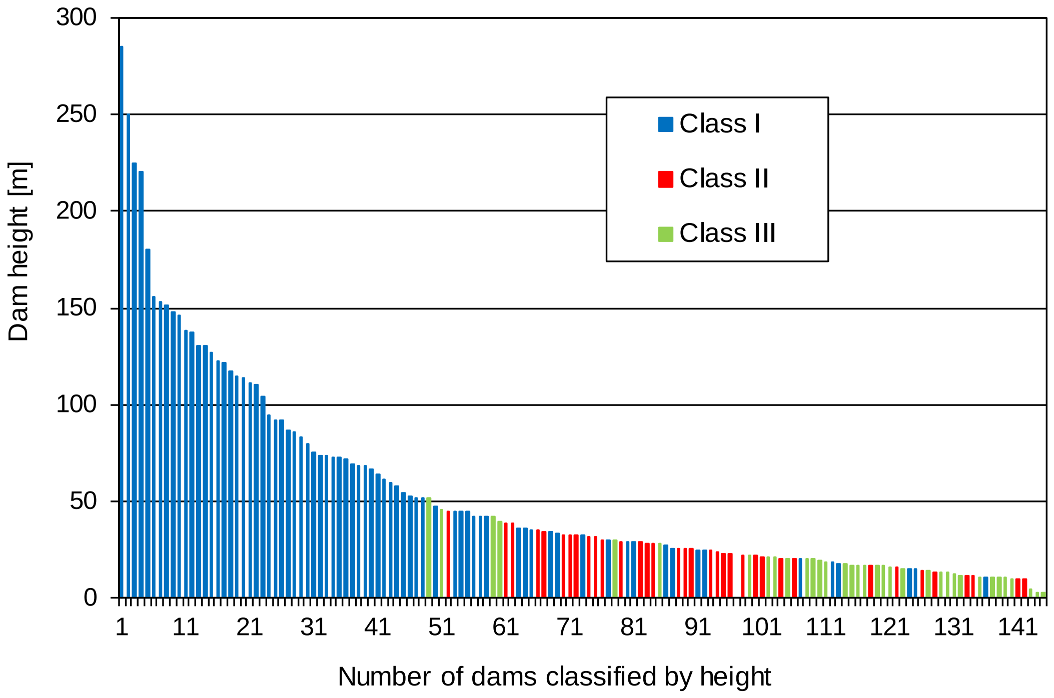


Figure 24: Dam classification according to wall height.

Figure 25 provides a representation of the type and height of dams depending on the year of construction. The concrete wall is divided into the following groups:

- 50% gravity dams (77 dams)
- 35% arch dams (53 dams)
- 12% weirs (18 dams)
- 3% others (2 buttress dams, 2 multiple arch concrete dams, and 2 arch gravity dams)

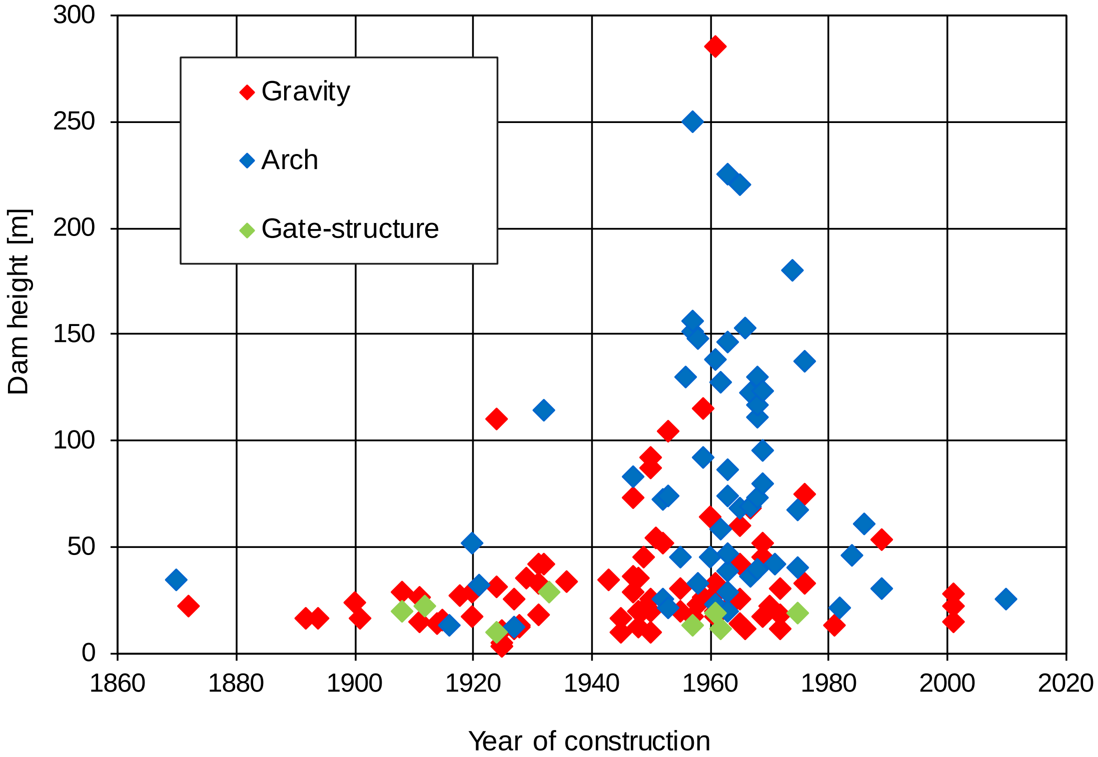


Figure 25: Dam type and height according to construction year.

2.1.2 Monitoring Systems

Dams are monitored structures. This is an essential feature that allows the early detection of abnormal behavior before swelling becomes visible and problematic for the safety of the dam. Information about the existing monitoring systems is of utmost importance in the assessment of dam behavior, and was provided by the members of the working group. A total of 4 categories of monitoring instrumentation were considered: two for horizontal and two for vertical displacements:

Horizontal displacements:

- Pendulums: Are among the most commonly used and accurate monitoring systems. Their measuring frequency enables the measurement of seasonal fluctuations.
- Other instruments, usually geodetic measurements: geodetic networks are widespread, but the frequency of measurements is usually not sufficient for measuring seasonal variations. For most dams, the measurement interval is 5 years.

Vertical displacements:

- Levelling: Along with geodetic measurements, levelling is one of the most commonly used vertical displacement monitoring system. However, the measurement interval is usually not sufficient to identify seasonal fluctuations.
- Invar wires, extensometers and other equipment: sometimes other equipment is used to monitor the vertical displacements of dams. These types of instruments have rarely been present since the beginning of operations of older dams. As a

result, there is incomplete behavior monitoring data over the entire service life of such dams.

The existing monitoring systems in relation to dam class are shown in **Figure 26** and in relation to the type of wall in **Figure 27**.

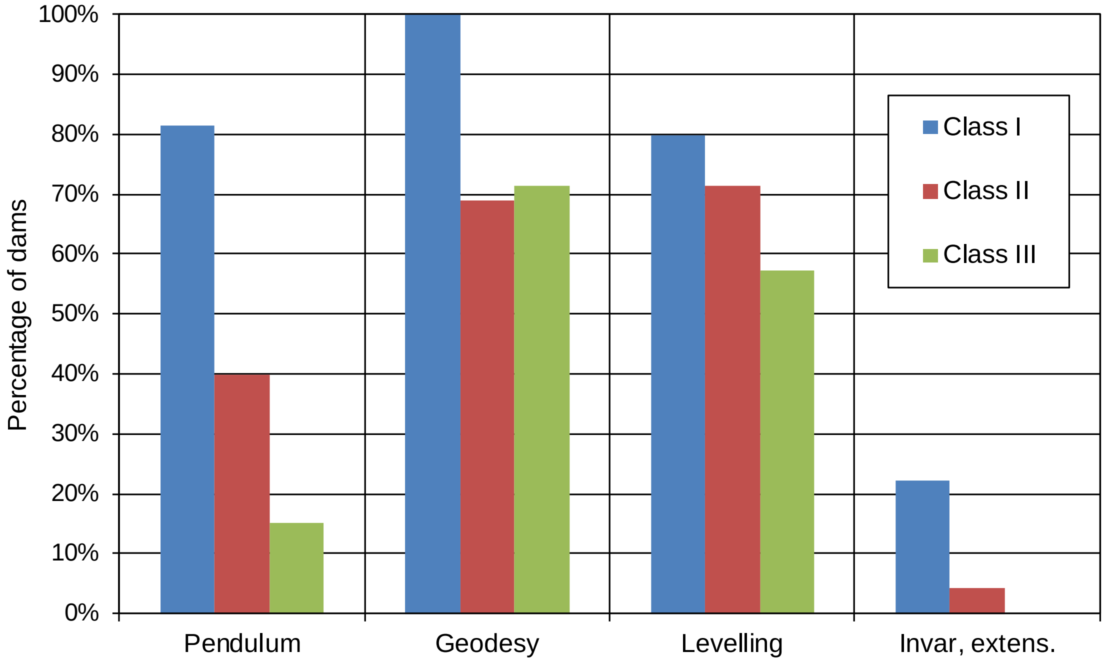


Figure 26: Available monitoring equipment in concrete dams (subdivided according to class).

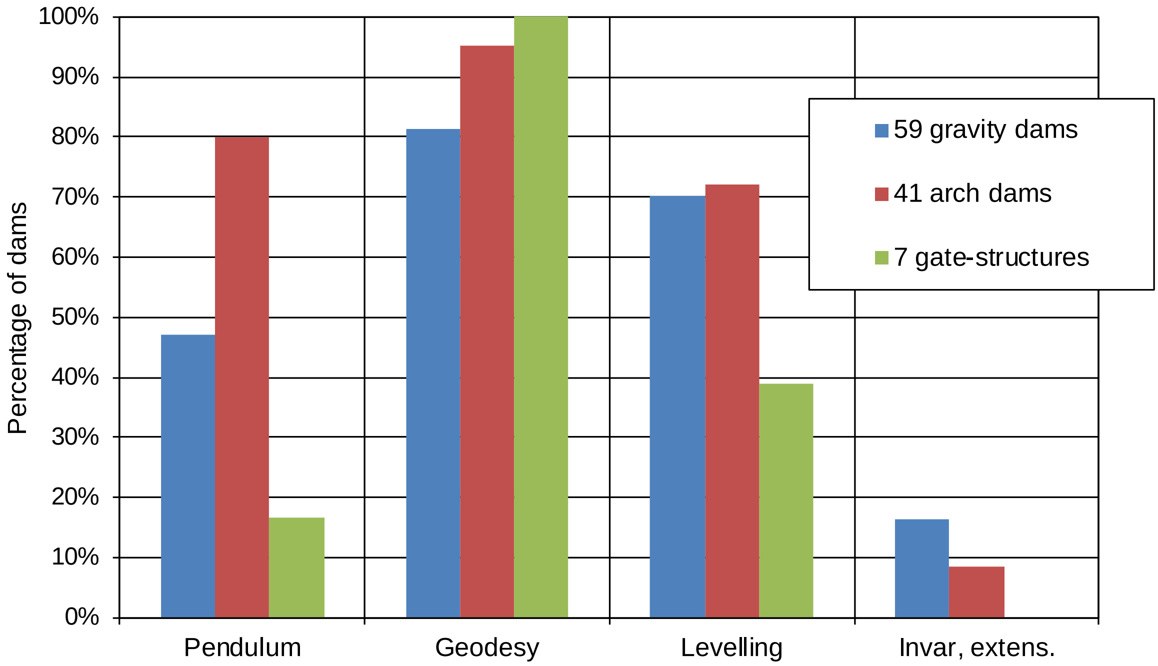


Figure 27: Available monitoring equipment in concrete dams (subdivided according to wall type).

Based on **Figure 26** one can draw the following conclusions:

- Class I dams are relatively well monitored. 80% of the constructions have pendulums, and geodetic measurements are carried out in all of them.
- Monitoring levels progressively decreases on smaller structures. About 70% of class II and III dams have geodetic monitoring with additional levelling. Pendulums however, are less common: they are installed in only 15% of Class III dams.
- Invar wires or extensometers installed vertically in the construction are seldom used in Swiss dams.

The results shown in **Figure 26** do not include all 154 dams in Switzerland. At the time this report was issued, the database only contained complete information for about 100 dams (65-70% of constructions). This percentage of completeness is comparable for all classes of dams.
As shown in **Figure 27**, weirs are generally less monitored than gravity and arch dams.

2.2 Evaluation methodology

One of the main objectives of the working group is the identification of irreversible drift in Swiss concrete dams, which can be attributed to concrete swelling. In the case of arch dams, an upstream drift and crest rise are expected, whereas with gravity dams the horizontal displacement is also possible downstream.
The first step of the study was therefore to identify the presence of such tendencies. Concerning the question on whether these types of movements can be detected, the following 5 answers are possible:

- Yes, the dam shows an increasing irreversible drift in the vertical or horizontal direction, which steadily develops in the long term (even if such movements are not connected to the concrete swelling). The relevance of the irreversible drift itself is not decisive, but the irreversible part is clearly recognizable in comparison to reversible drift movements (possibly only after processing measurements with suitable interpretation models in order to correct for the reversible part).
- Probably yes, but the measurements are insufficient or drifting is too small to confirm the phenomenon,
- Unknown situation, basically due to insufficient monitoring,
- Probably not, but monitoring is inadequate to ascertain such a conclusion,
- No, the dam shows a completely reversible behavior, without any tendencies. Had a permanent drift due to creep been detected at a young age, it was not included in this study.

The situation for Class III, II and I concrete dams is shown in **Figure 28**, **Figure 29** and **Figure 30**.

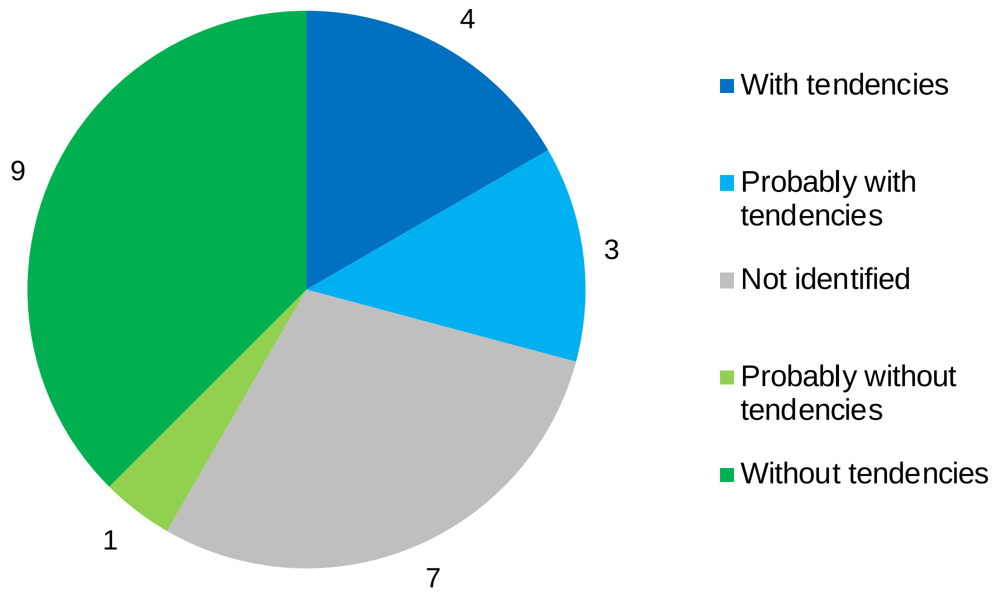


Figure 28: Presence of tendencies in class III concrete dams (pie chart values show the number of dams, the diagram considers 24 dams, i.e. 69% of the total number of structures).

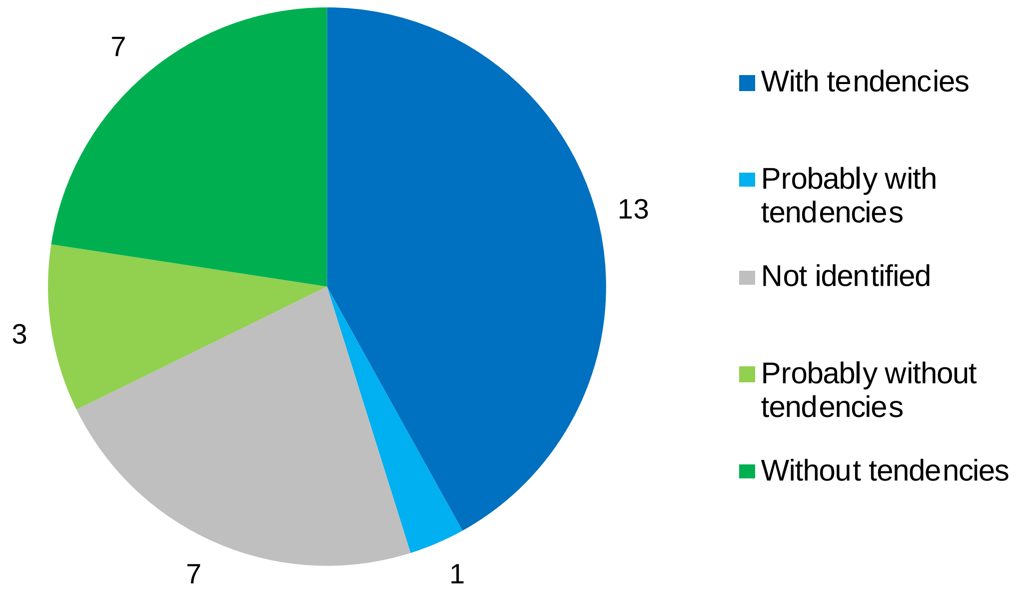


Figure 29: Presence of tendencies in class II concrete dams (pie chart values indicate the 31 dams considered, i.e. 89% of all structures).

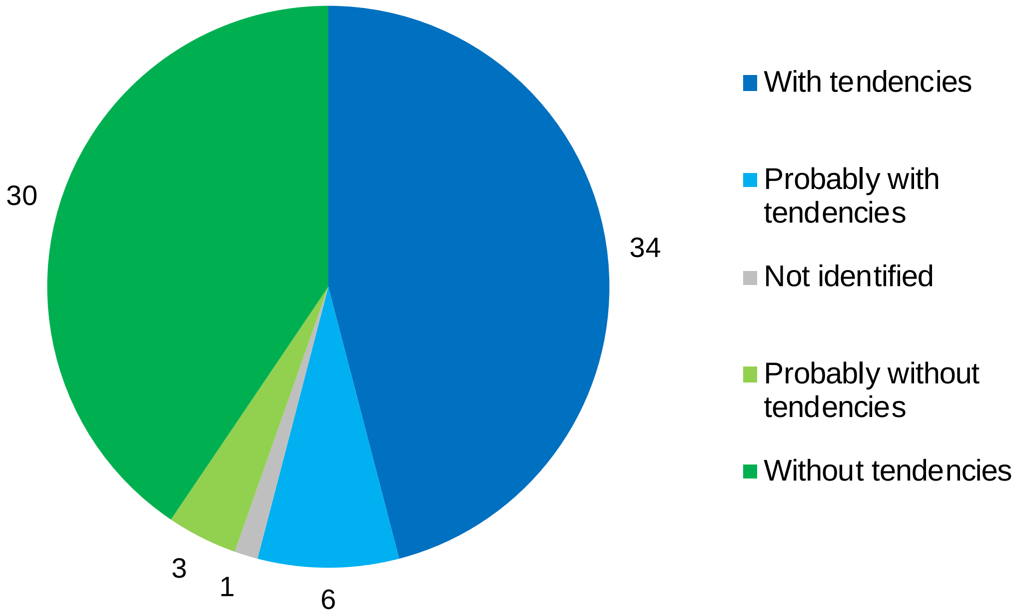


Figure 30: Presence of tendencies in class I concrete dams (the values around the pie chart show the number of dams the diagram considers i.e. all 74 class I dams).

As can be seen from **Figure 30**, knowledge about the behavior of Swiss dams is sufficient. Only one class I dam could not be rated, and the number of dams with remaining uncertainties is limited. There seems to be a large number of dams showing long-term tendencies, while 34 dams display a confirmed trend and a further 6 dams a possible trend.

Figure 31 shows a summary of trends for all concrete dams in Switzerland, including class II and III dams.

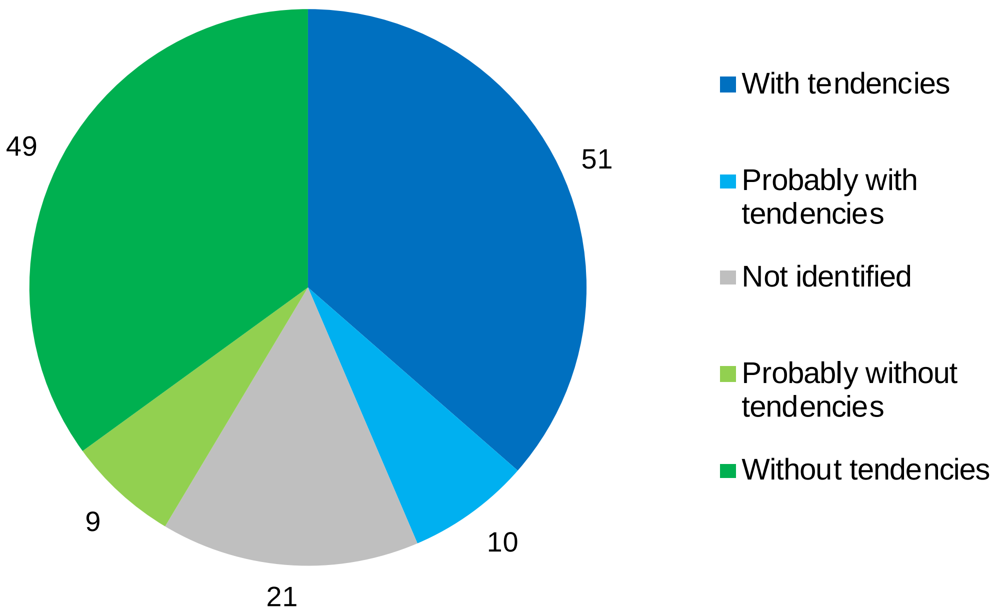


Figure 31: Occurrence of tendencies in Swiss dams (pie chart values show the distribution of 140 dams, i.e. 90% of all structures).

Compared to Figure 30, in Figure 31 the number of unknowns increases. It should be noted that Figure 31 only accounts for 90% of all concrete dams, as the database has not yet been completed. The increase in the number of dams with an unknown behavior is due to a lack of monitoring in smaller structures. In total, up to 60 dams are affected by these phenomena.

The working group also investigated a possible connection with the construction period. **Figure 32** shows that the number of dams with trends between 40 and 60% is practically independent of the year of construction.

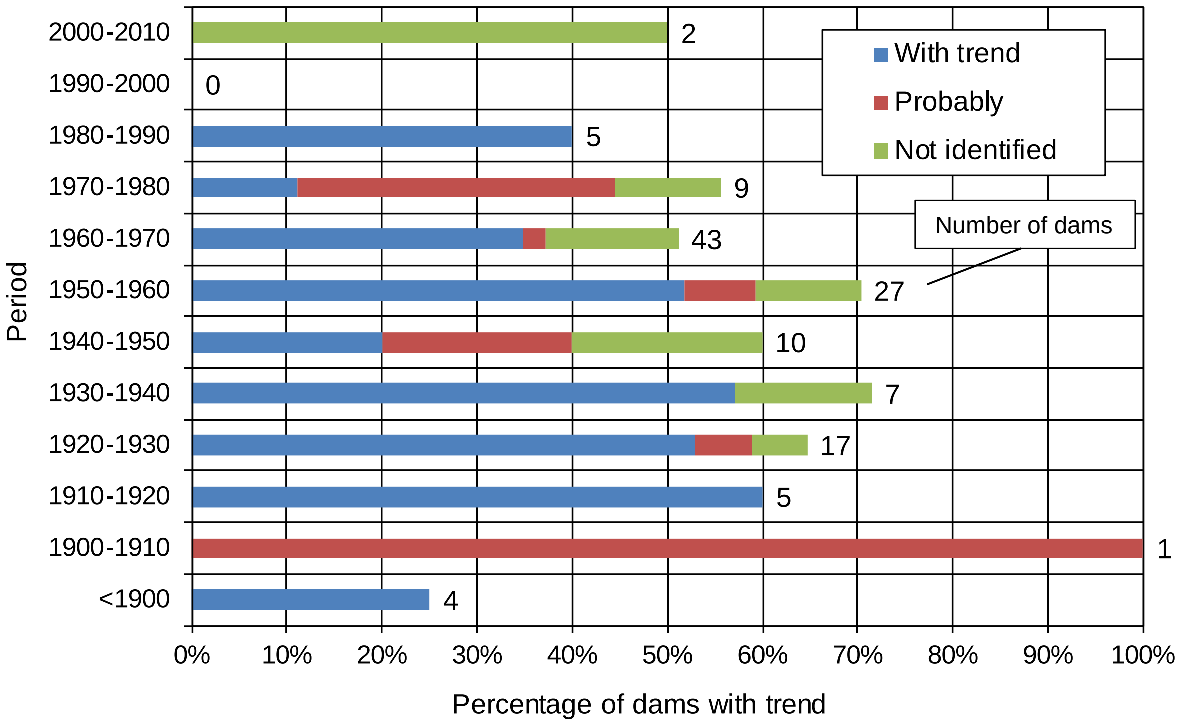


Figure 32: Percentage of dams affected by trends according to construction period.

Another statistical result can be found in **Figure 33**, which shows the existence of tendencies according to dam type. Gravity and arch dams are equally affected by irreversible phenomena, whereas in weirs the situation cannot be assessed with certainty because their behavior presents a greater number of unknowns. This is due to the complexity of these types of dams and the limited monitoring of weirs. It does not mean that weirs are less controlled than other dams. Weirs are generally complex structures, with many tunnels and passageways where visual inspections can play an important role in assessing behavior. The presence of large gates can also provide important information for the assessment of the structural condition. However, for the working group, this type of information was difficult to access, especially when compared to easily accessible data in other types of dams.

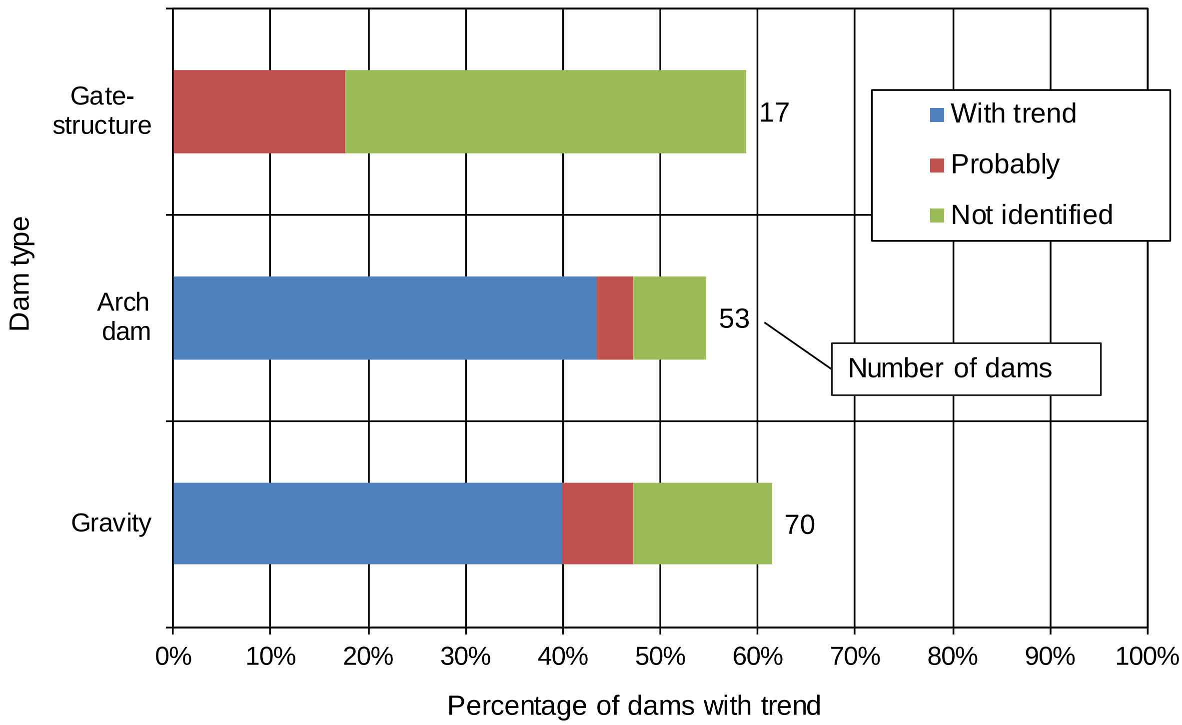


Figure 33: Percentage of affected dams according to trends, depending on dam type.

The distribution of dams showing trends associated with the geology of Switzerland is shown in **Figure 34**. The higher reactivity in the Alps (Austroalpin, Penninc and Autochton) is mostly associated with deformed and cracked quartz, whereas the Helvetian, Jura and Molasse region dams contain more sand and limestone.

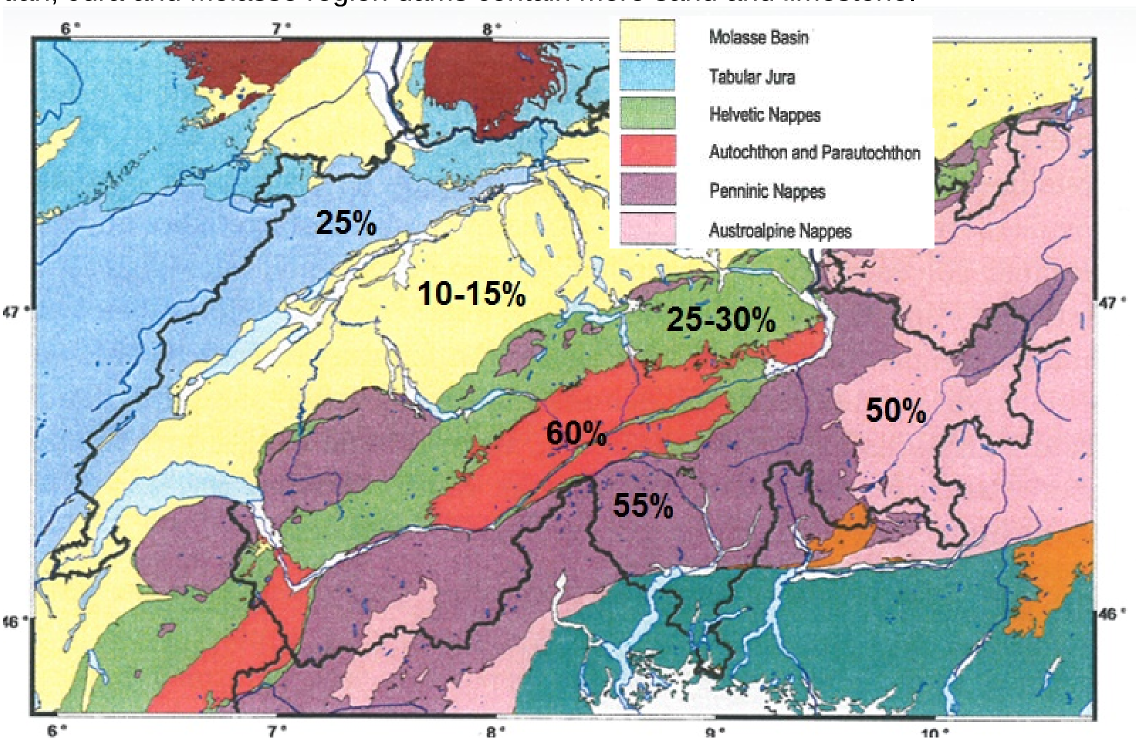


Figure 34: Percentage of dams with tendencies related to the geological areas of Switzerland.

One last interesting part of the study analyzes the causes of the identified tendencies. The working group considered the following 5 possibilities:

- Concrete swelling as a result of chemical reactions is confirmed, possibly also by laboratory tests.
- Possible concrete swelling, i.e. not confirmed by laboratory tests. The behavior is however compatible with concrete swelling (crest rise, horizontal displacement, cracks).
- Unknown or determinable causes.
- Presumably different causes: the perceived tendency is poorly compatible with concrete swelling, but the causes are not yet clear.
- Other causes: the reason of the identified tendency is known and is not associated with concrete swelling.

Figure 35 shows the situation of the 61 dams presenting tendencies (confirmed or probable from Figure 31. Apart from a few known cases, the vast majority of dams with irreversible drift may be affected by concrete swelling. In about 1/3 of the dams, the cause is clearly chemically related.

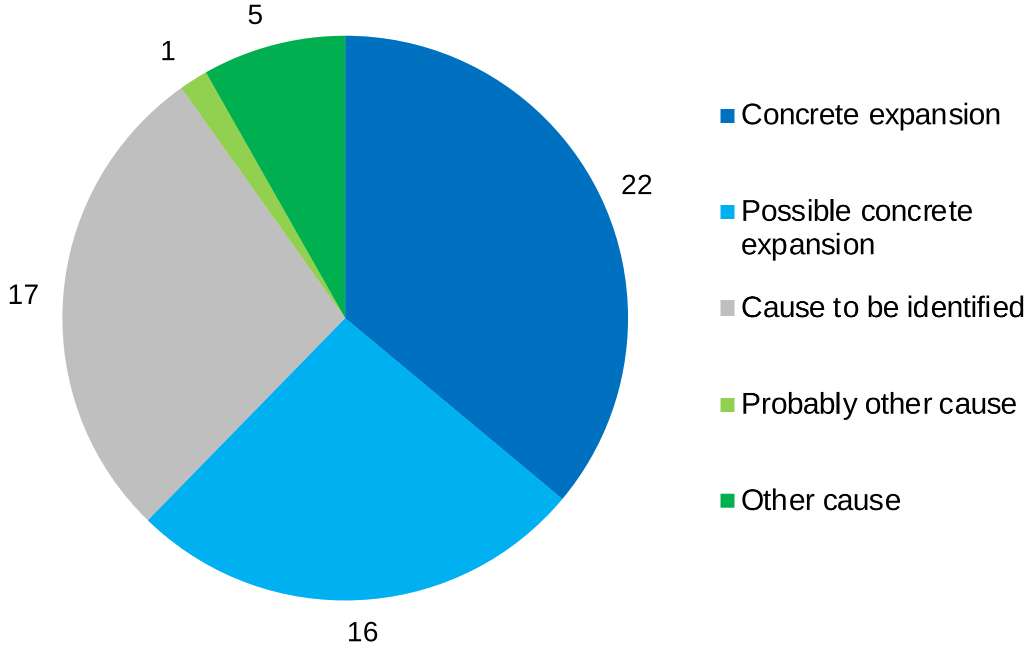


Figure 35: Distribution of dams according to tendencies

2.3 Data evaluation results

2.3.1 Initiation time

This section describes the behavior of a specific number of Swiss dams affected by concrete expansion. Dams subject to concrete expansion exhibit a normal behavior in an initial phase, followed by a phase in which horizontal drift and crest rising can be observed. The initial period of normal behavior is defined as the initiation time and is shown in **Figure 36**.

The inclined line corresponds to the age of the dam (reference year 2013). It is therefore not possible to have points above this line. However, it cannot be ruled out that dams which are not expanding today will continue to behave normally. From the figure, it can be clearly seen that in some cases, the initiation phase can be very long and may even take up to 50 or 80 years. This observation is not unique, since similar results have already been published [15]. Nevertheless, in other cases, expansion began immediately after construction. The astonishingly long initiation time of 80 years could possibly be associated with injection work in the affected wall, i.e. concrete swelling only started after injections.

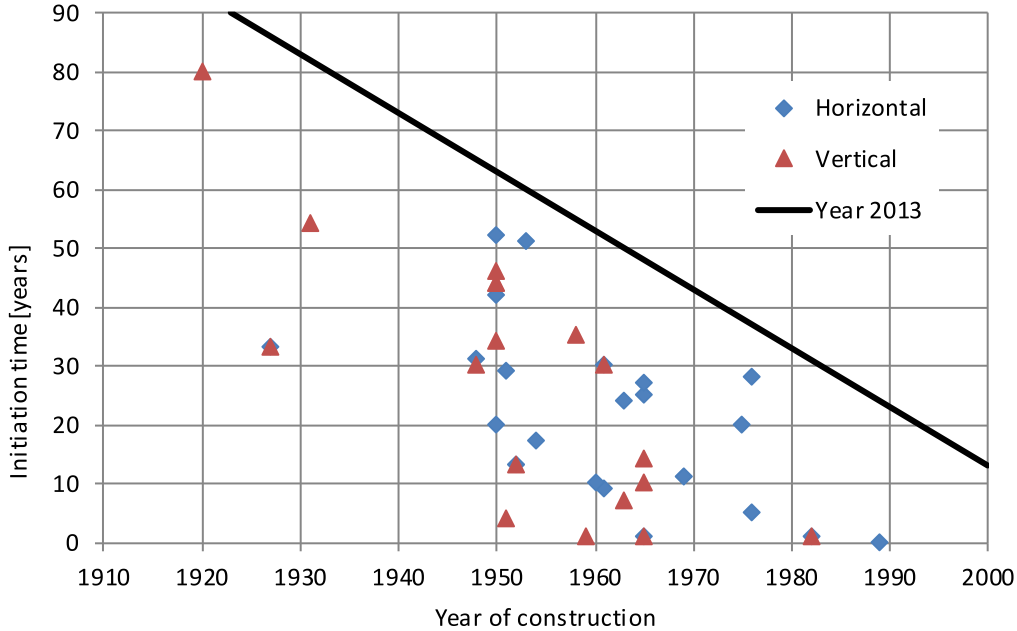


Figure 36: Time of initiation of the observed concrete expansion in various dams represented according to year of construction.

Another interesting observation is that the initiation time for the vertical direction seems to be shorter than that for the horizontal direction. In other words, vertical expansion seems to occur slightly before horizontal drifting. The average initiation time of a selected number of dams is 20 years for vertical displacements and 26 years for horizontal ones. Here, only 11 dams with an initiation time defined for both directions are taken into account.

2.3.2 Drift in horizonal direction

A certain horizontal drift and permanent displacement does not have the same relevance for a small rigid gravity dam as it would have on a large and flexible arch dam. To compare the behaviour observed on different structures, it is therefore necessary to find other parameters. Ideally, one should determine the average expansion, but based on the measured displacements in horizontal direction this is not straightforward. For this, an appropriate structural analysis should be at hand. Therefore, in order to relativize the observed behaviour with the dimension of the structure, it was decided to compare the permanent displacement with the annual variation induced by the regular operating conditions (water level and seasonal temperature variation). The permanent displacement is therefore divided by the typical annual variation of the displacement measured in the same point. The obtained parameter is illustrated in **Figure 37**. Arch and gravity dams are represented distinctly. A value of 1.0 means that the permanent displacement accumulated over a period of 10 years corresponds to the annual variation.

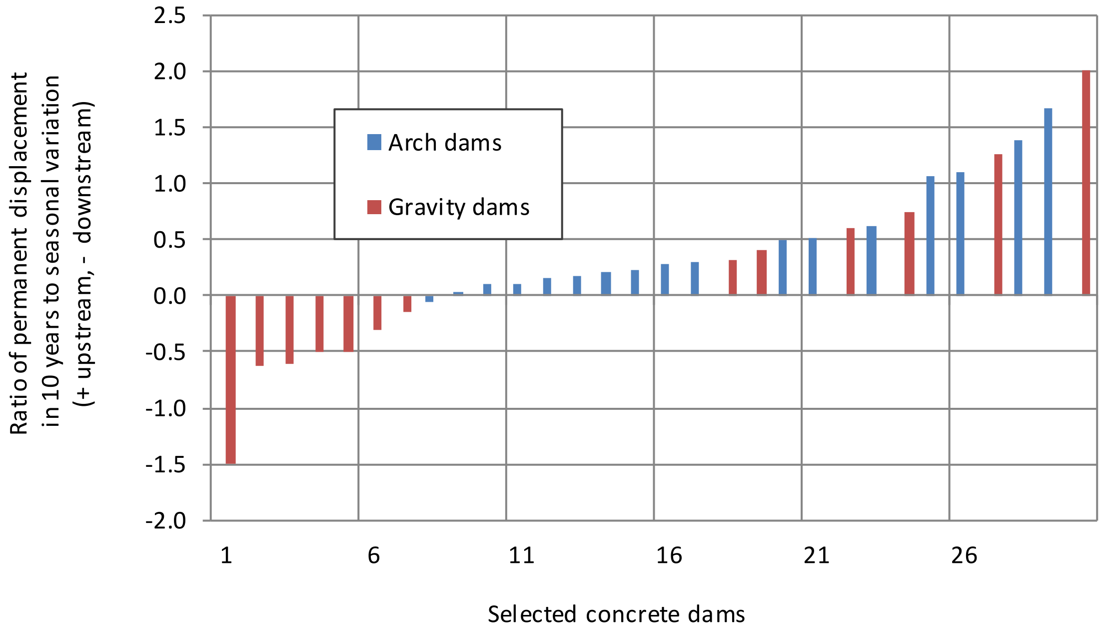


Figure 37: Horizontal drift rate compared with the seasonal elastic variation (30 dams where the required data could be reliably collected and tested).

Negative values indicates that the horizontal drift is directed in downstream direction. This might appear unusual for a dam with concrete expansion, but according to **Figure 38** there are more than one gravity dams exhibiting a drift towards downstream. The influence of temperature on the development of expansion has already been commented on and described [13, 14].
It has been shown that in gravity dams that horizontal drift is mainly determined by a differential expansion between the downstream and upstream faces. In the Alps, the temperature distribution is mostly influenced by sunshine and therefore by the orientation of the dam.

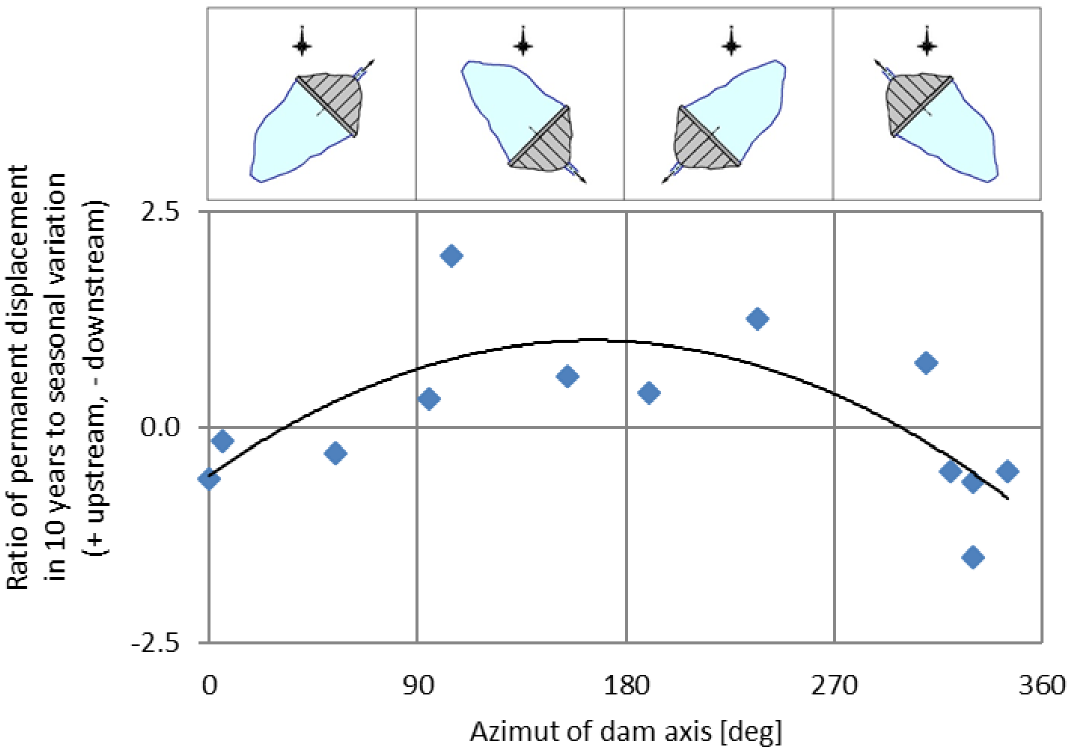


Figure 38: Horizontal drift of gravity dams in function of the orientation of the dam axis (dam axis normal to the faces and oriented towards downstream).

The result presented in Figure 38 seems to rather confirm the effect of sunshine: gravity dams with the upstream face towards the south (azimuth of dam axis between -45°, i.e. 315°, and +45°) exhibit a permanent displacement towards downstream, while for gravity dams with the downstream face towards the south (azimuth of dam axis between 90° and 270°) a permanent displacement exclusively towards upstream is observed.
By the end of 2013, the total permanent displacement in some cases exceeds the regular seasonal variation by 8 times. Repairing works have been undertaken in certain cases where the permanent displacements exceeded the seasonal variation by 5 times. For lower displacements, no works have yet been performed.

2.3.3 Vertical displacements

To assess the vertical displacement, the levelling readings at crest elevation are generally taken into account. The non-reversible component of displacement is then divided by the dam height, to obtain an average expansion in vertical direction. The corresponding values for a selection of arch and gravity dams are illustrated in **Figure 39**

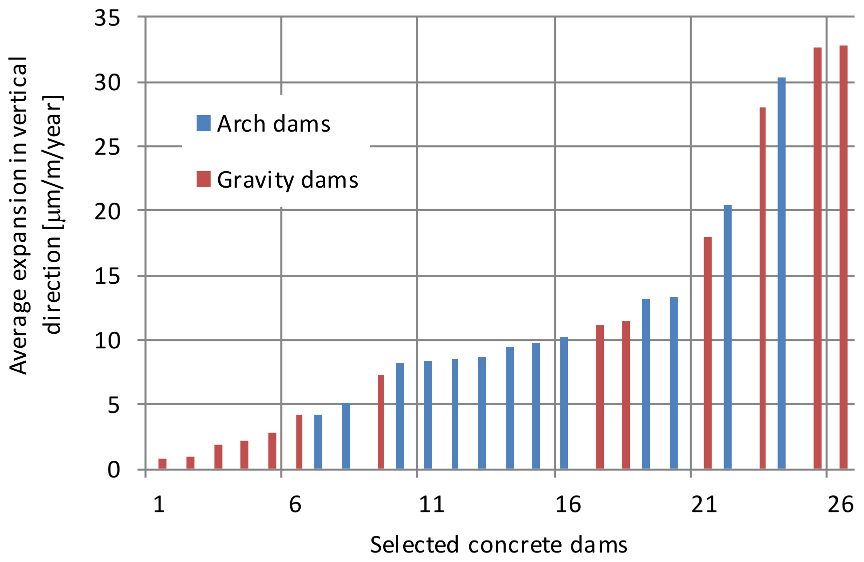


Figure 39: Average vertical expansion along the entire dam height (based on 26 dams where the required data could be reliably collected and tested). The expansion rate refers to the last 10 years.

The expansion rate reaches at maximum 30-35 µm/m per year which can be considered as still moderate. At Mactaquac an expansion of up to 140 µm/m per year has been estimated [16]. The values illustrated in **Figure 39** are quite similar to the ones presented in a recent study performed by Electricité de France [18].

Finally, **Figure 40** shows the total expansion in vertical direction developed until the end of 2013. Like before, it corresponds to the average value, which is determined by dividing the permanent rising of the crest by the dam height. The two dams with values higher than 800 µm/m have been rehabilitated.

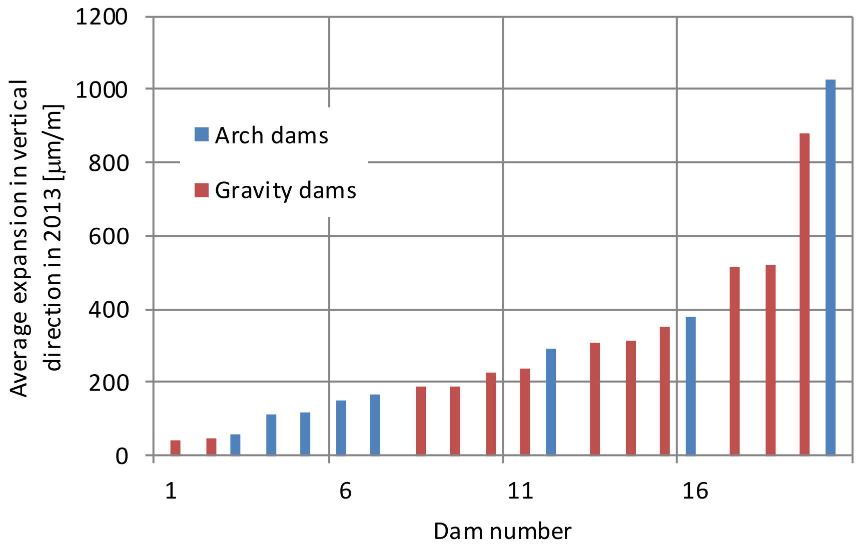


Figure 40: Average vertical extent over the entire wall height (from 20 dams where the required data could be reliably collected and tested).

2.4 Conclusions

The AAR working group analyzed the behavior of 154 Swiss dams with the aim of identifying the effects of concrete expansion induced by the chemical reactions on these types of structures. Not all concrete dams could be reliably analyzed, because in some the required data was not available or could not be analyzed sufficiently by the working group. In the end a total of 119 dams were rated.
The results show that about 50% of these dams show tendencies and permanent drift (61 dams). Of these:

- 38 dams were identified as exhibiting these tendencies which are compatible with concrete expansion (22 of which, where concrete expansion was also further corroborated by laboratory tests).
- 6 dams were identified as subject to other phenomena.
- For a further 17 dams, the situation is still inconclusive (the working group was unable to sufficiently interpret available data).

From this analysis, it can be concluded that concrete expansion affects between 35% and 45% of Swiss concrete dams. The phenomenon is thus highly relevant.

Swelling dams are generally characterized by an initial period of operation with a regular and reversible behaviour. This initiation time may vary between 0 and many years, reaching up to 50 or even 80 years in some cases. After this initial period the effect of concrete expansion becomes visible with crest liftings and horizontal drifts. Such tendencies are nearly linear with time. The mean expansion rate estimated in vertical direction varies between 1 and 30 μm/m/year. In the horizontal direction the arch dams move towards upstream, while gravity dams can also move in the downstream direction, depending on their orientation. In fact, gravity dams tend to drift towards north, possibly due to the greater expansion at the face exposed southward and thus to greater sunshine.

Repair works have been executed in a few cases, such as Illsee, Sera and Salanfe. In these dams the total concrete expansion reached roughly 500 to 1000 μm/m. In the future, repair works on further dams should be expected.

3 Research priorities and strategy

3.1 Research findings to this day

The first researches on concrete swelling in dams were evaluations of a series of long-term deformation measurements on dams. 10 to 20 years after the construction of certain dams, deformation trends manifested themselves showing early signs of unusual behavior with few exceptions, which were interpreted as concrete expansion (see **Figure 41** to **Figure 43**). Levelling measurements have shown uplift of dam cross sections while tachymetric and pendulum measurements have shown horizontal displacements. In the case of arch and curved gravity dams, these horizontal drifts are in upstream direction. For gravity dams deformation may be directed upstream as well as towards the valley (see Figure 37 in Chapter 3). Initial explanations for this increase in volume were attributed to concrete and ambient temperature increases measured in the 1990's. Only with increasing deformation was AAR, and in a few cases sulphate swelling, seen as the reason for long-term deformation trends and became the center of specific research. The relatively late recognition of concrete swelling is also due to the fact that creeping of dams under constant water pressure overshadows the effect of swelling or has also in some cases compensated for it (see Figure 43).

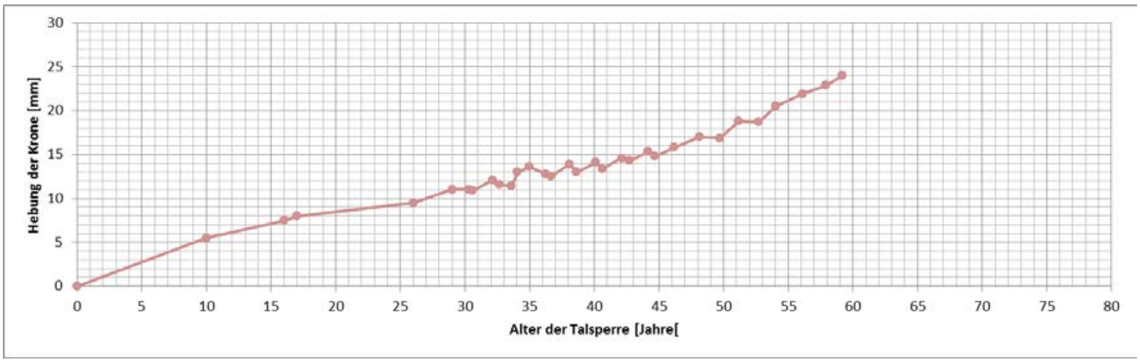


Figure 41: Long-term behavior of dam crest elevation.

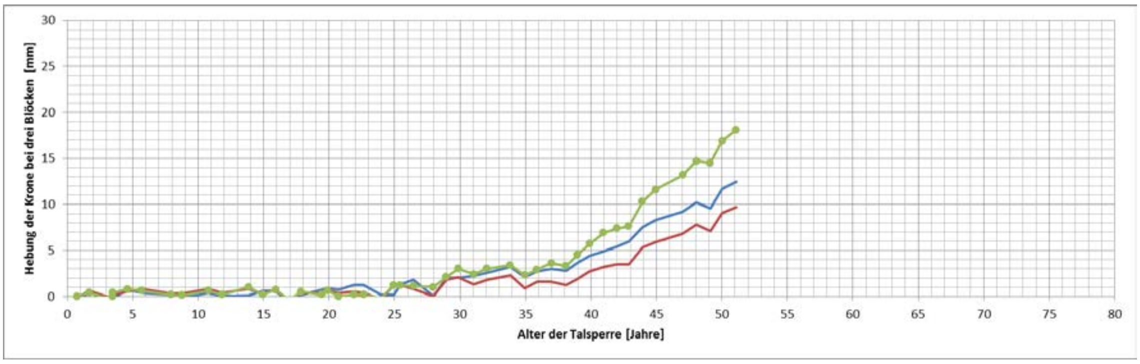


Figure 42: Long-term behavior of a rising dam at three different levels.

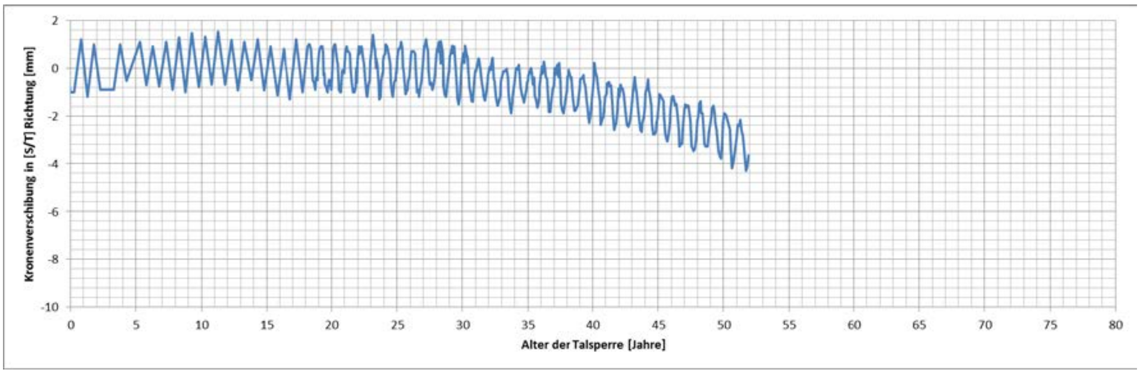


Figure 43: Long-term behavior of horizontal crest displacements in upstream / downstream direction.

Of central interest in the behavioral analysis of a swelling concrete dam is knowledge about the local distribution of swelling across the entire wall, i.e. in cross-section and along the dam's longitudinal axis. The necessary deformation strain measurements are only conditionally available. For small dams, usually without inspection tunnels, only crest levelling or simple tachymetric angle measurements exist. This is certainly sufficient for dam monitoring, but an analysis of the local concrete swelling distribution is not possible. For larger dams, there are inspection tunnels in which the dam deformation is measured, or several geodetic survey points on the downstream face (see **Figure 44**). This allows for a more detailed analysis of deformations, such as elongations on different levels of the wall.

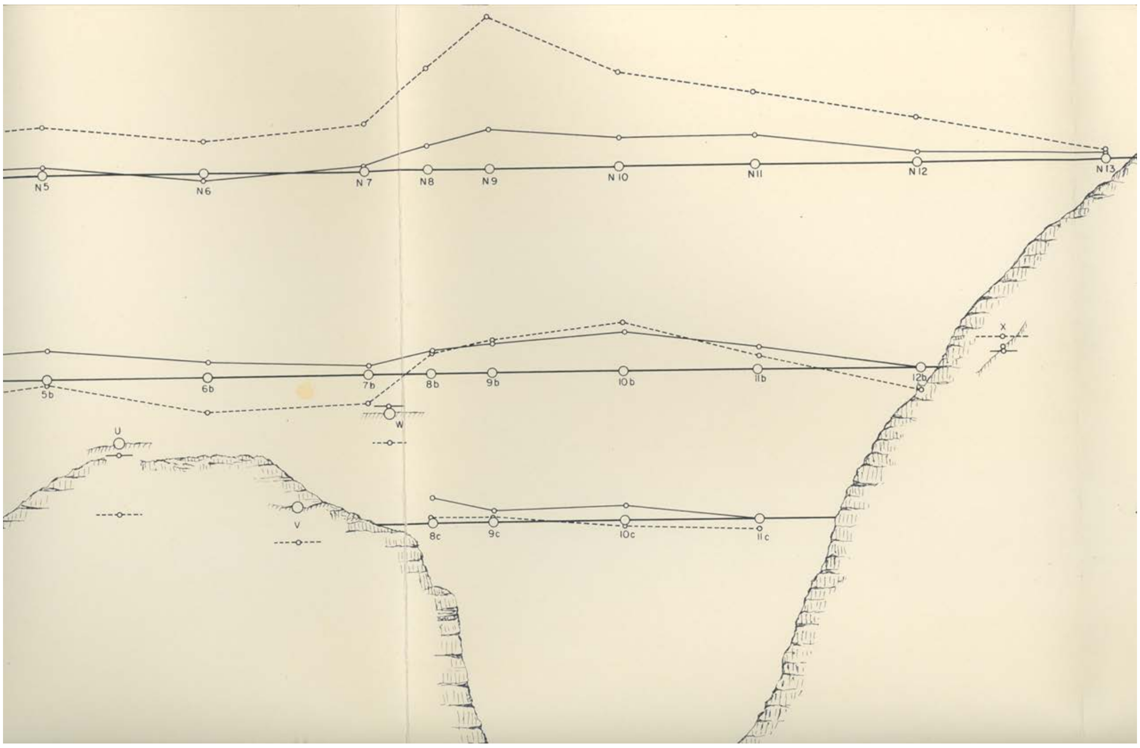


Figure 44: Tachymetrically determined elevations of a dam at three different levels and for two points in time.

Analyzing the deformation of walls over their height has clearly shown that higher dam levels are more prone to swelling. In recent years, various dams have been further equipped for measuring the local distribution of concrete elongation. For example multiple extensometers or sliding micrometers are being used, and pendulums increasingly also have the ability to measure vertical displacements. All of these tools permit a more detailed determination of local swelling.

3.2 Alkali-aggregate reaction-influencing parameters

Parameters influencing AAR swelling, and therefore also the local distribution of swelling in the dam cross-section are (see Chapter 2)

- Wall temperatures and their seasonal changes
- The petrography of aggregates, especially the presence of amorphous silicon (amorphous quartz)
- Concrete relative humidity
- The alkali content of the concrete, thus cement dosage and alkali content of the cement used
- Stress conditions within the concrete

3.3 Relative concrete humidity parameter

In the beginning of research on AAR, the importance of relative humidity in concrete was given high importance. For example, moisture sensors have been installed in the concrete and upstream face membranes have been applied to dams, on the one hand to minimize percolation of the concrete per se, but also with the intention of preventing or at least significantly reducing swelling rates. Measuring relative concrete humidity has turned out to be relatively demanding and experience with membranes has shown that the leachate can be greatly reduced, but that the AAR reaction process continues unhindered. Today, there is an overall agreement that the concrete mass of dams and their large dimensions, as well as humid weather conditions in Switzerland, always make for moist concrete dam walls. The determination of the moisture distribution in a wall therefore has little to no relevance in the analysis of AAR swelling in dams in Switzerland. The concrete is always humid enough - relative humidity > 80% - so that AAR occurs unobstructed.

3.4 Cement alkaline content parameter

The alkali content of the cement used has also been studied in the past. Given the relatively old age of many dams and the sometimes rudimentary historical data on cement composition, it is difficult to derive a local distribution of AAR based on this parameter. In general, it can be stated that cements used for Swiss dam construction have enough alkali content making AAR possible.
Tests on concrete samples with different cement dosages have not shown any very significant variation with respect to existing gels in thin sections or with swelling potential in long term swelling experiments. **Figure 45** shows qualitative thin-section analyzes for the presence of gels in 24 concrete samples. The difference between surface concrete containing 250 kg/m^3 cement, and core concrete with 190 kg/m^3 is not significant.

It is therefore usually not effective to focus on cement properties and cement dosages when analyzing dam AAR behavior. Exceptions cannot however be ruled out (see example in chapter 4.1)

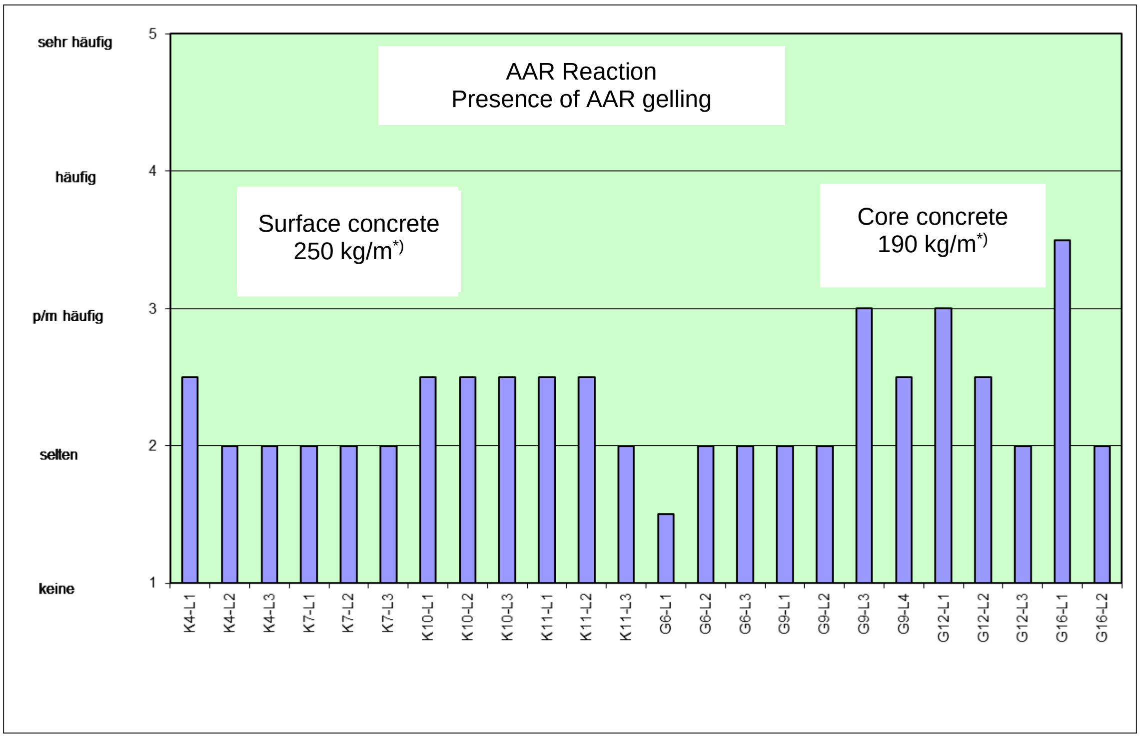


Figure 45: Description of the occurrence of AAR gelling for 2 cement dosages in the same dam.

The remaining enabling and central drivers of AAR swelling of existing dams in Switzerland are therefore the petrographic composition of additives, concrete temperature, and actual stress forces in the concrete.

3.5 Reactivity of additives parameter

The content of silicon in an additive and its reactivity determine the total swelling mass potential as well as the reaction rate. Temperature mostly controls the reaction rate. Since AAR is a chemical process, the swelling rate is heavily dependent on concrete temperature (see comments in Chapter 2.

The composition in the aggregates i.e. the origin of the rocks is usually known for every dam. The types of rocks can however be different. For example, when extracting aggregates from quarries, rock composition is often relatively uniform and the type of rock is known (e.g. pure lime or gneiss). However, when extracting aggregates from alluvial material, the composition of rocks may vary widely due to different catchment areas. Therefore, despite the mixture of individual aggregates, it should not always be assumed that the distribution of minerals prone to swelling in the dam concrete is exactly the same. A determination of this distribution in an existing dam is hardly conceivable. Under certain circumstances it can be seen from construction reports whether a slightly different aggregate was used. Otherwise, a relatively homogeneous distribution of the swelling-prone minerals must be assumed. Research on the distribution of aggregates is therefore generally not effective. One aspect, namely

the alignment of flat-oval aggregates of alluvial material during the concreting process, which may be deposited horizontally, may play a role in influencing the swelling direction, but this has not yet been investigated.
The reactivity of rocks in Switzerland has already been addressed in Chapters 2 and 3. Concrete originating from the crystalline core of the Autochthon, the Pennine cover and the crystalline Eastern Alpine region, have a greater potential for swelling than concrete made of Helvetic cover and Jurassic rocks.

Swelling potential (free swelling) can generally be determined with controlled long-term laboratory tests (see Chapter 2).

3.6 Temperature parameter

It can be inferred from theoretical considerations, laboratory tests, and comparisons between dam wall deformations and damage patterns (structural cracks), that temperature distribution and its annual variation is the most important parameter for present-day swelling in dams. Determination of concrete temperatures at various points in the dam by means of in-situ measurements as well as mathematically, is relatively simple.

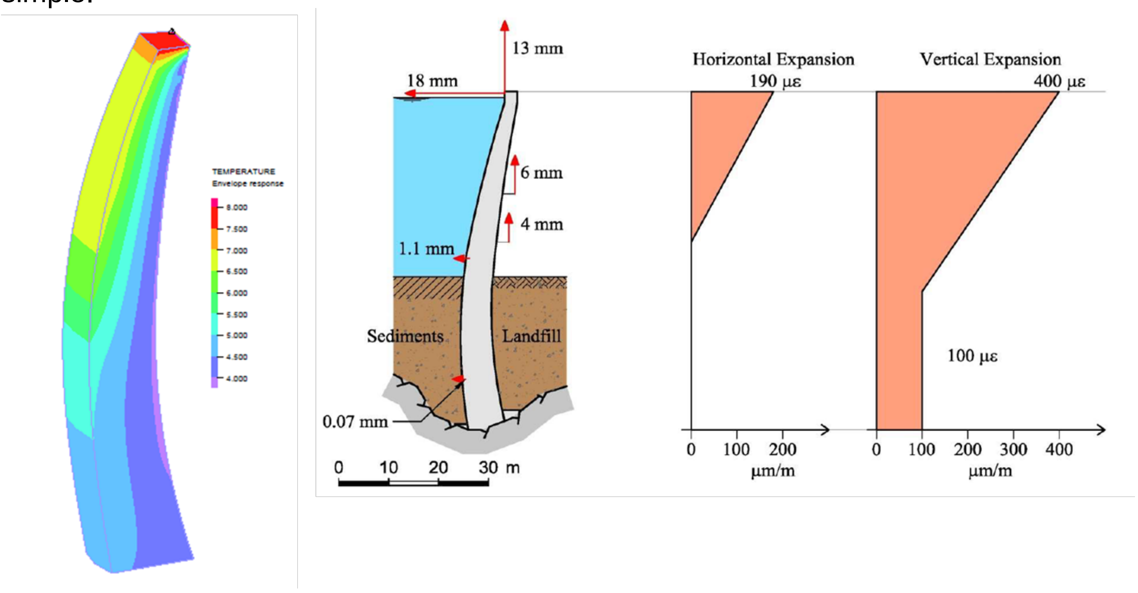


Figure 46: Temperature distribution in an arch dam with measured dam displacements and resulting strains (from [14]).

In many of the examined dams, it can be seen that the warm and slender upper wall area swells significantly more than the colder and more massive lower part. In the above-mentioned arch dam, the lower, backfilled and cool wall area has expanded vertically by 100 µm/m, the crest area by 400 µm/m, i.e. 4 times more. The distribution of structural cracks also suggests that the side facing the sun, in this case the upstream face, swells more than the downstream face. Radiant heat significantly influences swelling behavior. Concrete temperatures on sunlit areas are between 4 to 6 °C warmer than the ambient temperature.

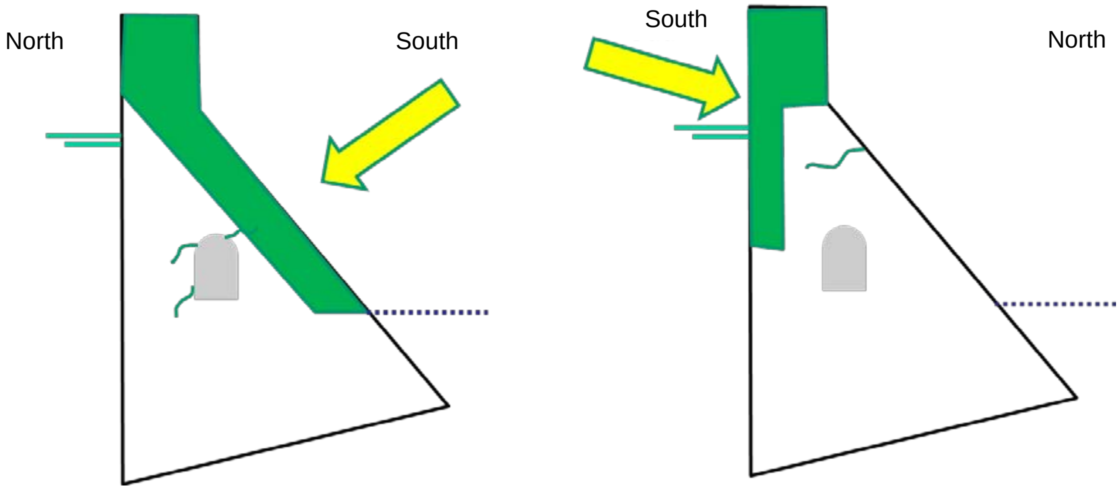


Figure 47: Greater swelling in sun-facing concrete areas, and places of first structural crack occurrence.

Dams respond with uplift and horizontal displacement mostly in the upstream direction to concrete volume fluctuations, which is geometrically dependent on the shape of the dam. Generally, it must be deduced that the dam core is less swollen than its faces because of lower temperatures. This creates internal restraining stresses. **Figure 47** schematically shows two real examples of gravity-arch dams. 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.
By analyzing crack images and taking all other parameters into account, it is possible to draw initial conclusions on the local swelling distribution within the dam.

3.7 Concrete strain parameter

Uniaxial compression tests have shown that expansion strains decrease with increasing axial stresses in the load direction. At compressive load values of 5 to 8 MPa, strains are completely suppressed. **Figure 48** shows the results of such uniaxial compression tests. It is noteworthy that axial strains influence the maximum expansion strain threshold. In arch dams, strains in the range of 4 to 5 MPa can be present, and can affect swelling behavior in dams.
However, it is also known from similar research on swelling, that axial swelling is suppressed by strains, but that the latter increase transversely in the load direction. The chemical reaction that leads to concrete expansion is therefore not stopped, and further continues.

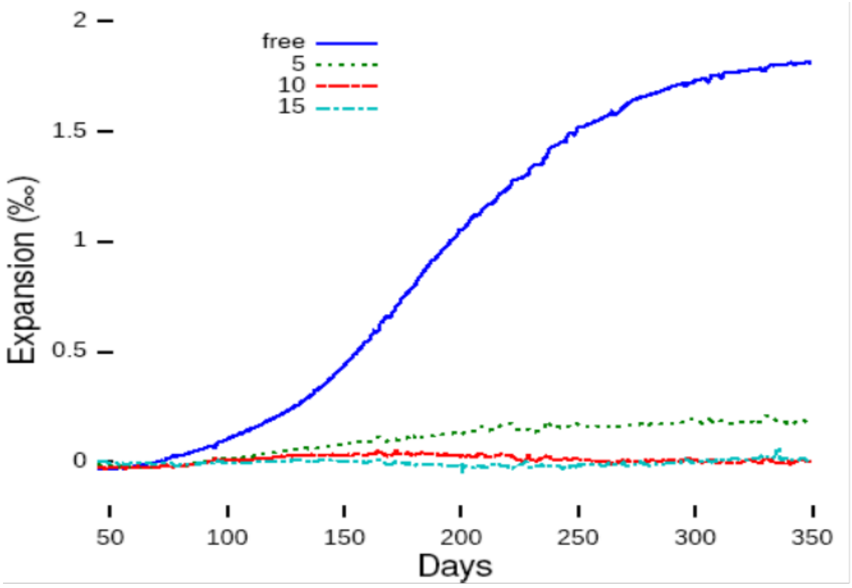


Figure 48: Swelling strains for various axial stresses in MPa (Prof. K. Scrivener, EPFL Lausanne)

The change from the axial to the radial direction of expansion is a consequence of the cracking pattern redistribution. The swelling-prone minerals of concrete aggregates, shown in **Figure 49** as yellow dots cause micro-cracks through expansion which propagate in the aggregate. These cracks are responsible for the increase in volume of the concrete and not the gel volume. The effective increase in the volume of silica minerals (gels) is only of a few percent.
Concrete strains influence the alignment of micro cracks which increase stress in the load direction, thereby reducing expansion in that direction, but reinforcing it in the transversal direction.
True tri-axial swelling tests are not yet available. Swelling behavior in tri-axial states of tension is currently part of ongoing research at the EPFL. The distribution of concrete stresses in dams is thus an important factor in understanding swelling and of concrete dam behavior.

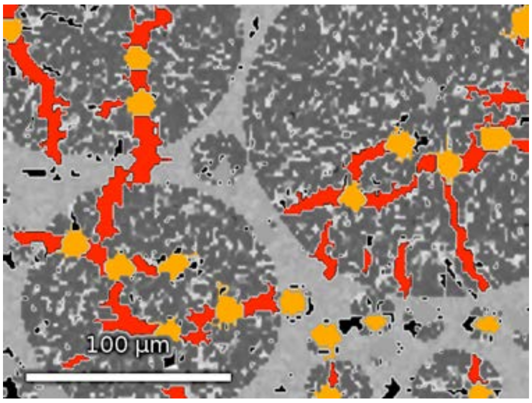


Figure 49: Schematic representation of the swelling process in concrete (Prof. K. Scrivener EPFL Lausanne).

3.8 Distribution of AAR swelling and dam geometry

Figure 50 shows the measurement of swelling strains since construction for 11 dams. There are gravity arch dams, gravity dams, and arch dams shown. In addition, the figure gives a snapshot of when the start of the deformation trend was determined [S] and when first structural cracks were observed (blue areas). Notice of the swelling phenomenon at the aforementioned dams took place 4 to 27 years after construction (an overview was already presented in Chapter 3).

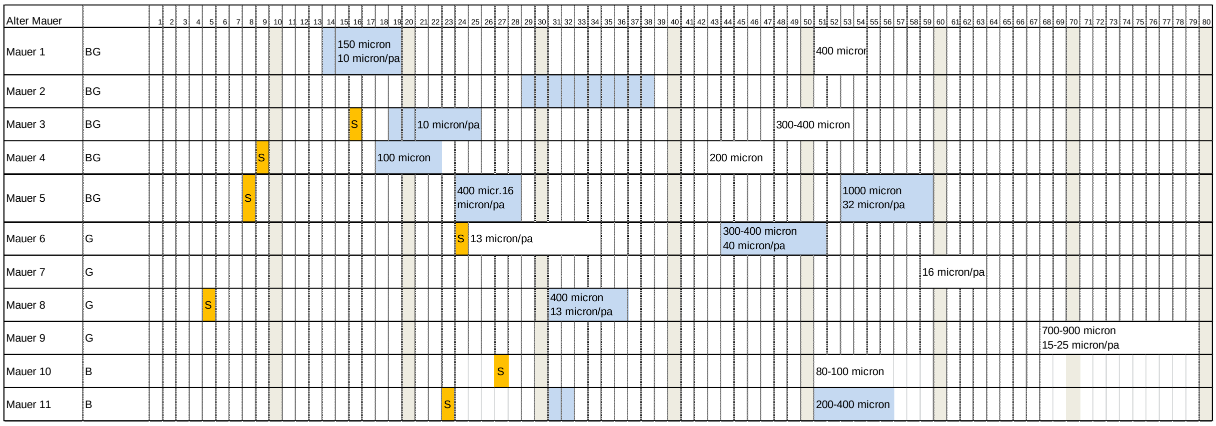


Figure 50: Temporal evolution of concrete swelling and timing of first structural cracks at 11 different dams.

Structural cracks in dams have initially and predominantly occurred in arch gravity dams 10 to 20 years after construction. Due to their compact design, initial cracks were found at strains of approx. 100 µm/m to 150 µm/m at strain levels of 10 to 20 µm/m/year. In the case of studied gravity dams, the time of appearance of the first cracks is 20 to 50 years after construction, at elongations of 300 - 400 µm/m and strain levels of 13 to 40 µm/m/year. Some gravity dams have no cracks, although concrete did expand by 400 µm/m. At the two illustrated arch dams, swelling was found to have initiated about 25 years after construction, with no signs of structural cracks, even though strains were measured at 100 to 400 µm/m.
This clearly shows that the absolute amount of swelling and its rate have an impact on the development of structural cracks in concrete dams, yet the dam construction method is the most important determining factor. The less a dam structure is able to sustain the internal swelling stresses due to deformation, the more susceptible the dams are to AAR damage.

3.9 Key findings

Tests performed to date on Swiss dams show that the properties of aggregates and temperature distribution in the dam are the two main influential parameters for the evolution of concrete swelling over time. The local distribution of swelling strains is also influenced by concrete stresses such as static loads and stress forces from swelling itself.
Although the nature of concrete additives has a significant effect on concrete swelling, the distribution of these additives within the dam is usually relatively homogeneous. This means that the local distribution of concrete expansion within a dam is not significantly dependent on additives. Concrete moisture and cement content, or the type of cement used play a minor role for AAR in existing dam structures in Switzerland.

A very important aspect is the combination of the local distribution of AAR and its evolution over time within concrete dams in combination with their design i.e. geometry [14]. This combination determines the timing of the appearance of the first structural cracks in the concrete, as well as the further process of structural damages. Depending on the configuration of the swelling and dam geometry, structural cracks on the downstream face and / or cracks within the concrete develop. Cracks on the dam surface are visible and also relatively easy to measure. Cracks within the dam can be recognized when they intersect an inspection tunnel, but internal cracks often remain unknown.

3.10 Future research focus

Understanding of AAR processes has continually improved through research over the past 5 to 10 years. At the EPFL, Prof. K. Scrivener's work has, for the first time, demonstrated a clear correlation between the proportion of silica having reacted and unconstrained expansion (**Figure 51**). In addition, it could be shown that concrete expansion is generated by micro-cracks in the aggregates. The micro-cracks, in turn, are caused by the expansion of local swelling-prone silica.

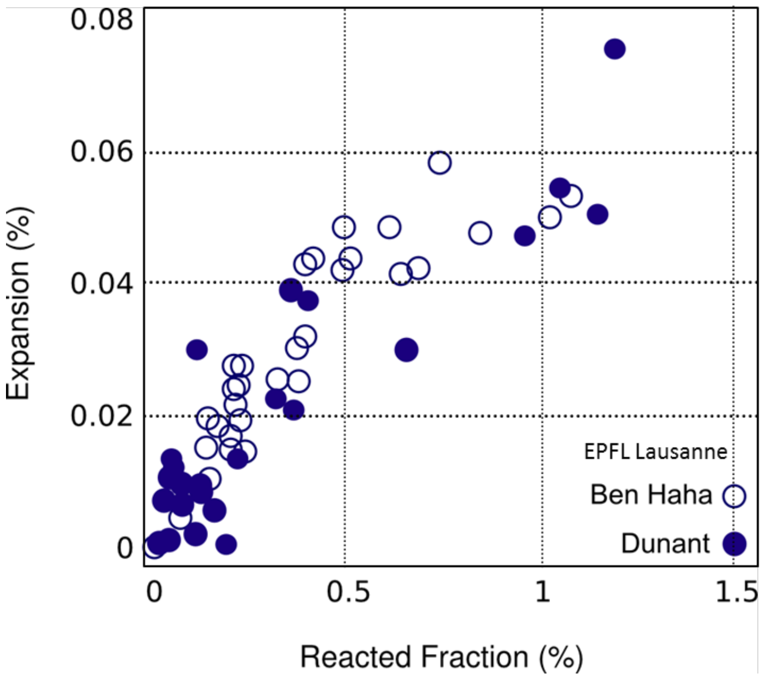


Figure 51: Proportion of silicon having reacted versus concrete expansion, from experiments by EPFL Prof. Scrivener.

After research has now revealed the fundamental chemical-mechanical process of AAR, new research needs to focus on open questions concerning the influence of concrete stresses (tri-axial) on AAR and other influencing factors such as concrete creep or pore water pressure. These findings must then be applied and implemented on the real structures. This requires findings from the micro-scale, to be transferred through laboratory tests (meso-scale), to the structures themselves (macro-scale). A research project of the Swiss Federal Office for Energy (SFOE) was carried out at the EPFL with the aim of using numerical models to reconstruct concrete dam swelling over space and time in order to be able to make predictions.

As for exploring dam swelling behavior, the focus in the future will be on determining local concrete expansion behavior, i.e. concrete expansion along the cross-section of the dam as a result of AAR, both horizontally and vertically. In addition to conventional deformation measurements, such as leveling and tachymetric measurements, additional instruments are being developed and existing systems are being further supplemented.

Another focus for future research is the measurement of structural cracks. Detection and measurement of cracks that are visible on dam faces represents the state of the art of what is being done in this sector. There is still potential for development in the exploration of internal concrete cracks, and especially their precise measurement and monitoring. Today, this is practically only possible through extensive drilling campaigns. Non-destructive tests, e.g. ultrasound or seismic imaging (tomography) either a low penetration depth in comparison to the structural dimension, or a local resolution (detection of cracks in the mm range) which is too low.
There is still an important development potential in the field of non-destructive testing of concrete structures with concrete thicknesses in the meter to decameter range.

4 Diagnostic examples

Even though several Swiss dams are subject to the Alkali-Aggregate reaction, only two examples are presented in this section. A third dam structure affected by AAR is also presented, although it does not concern a dam itself but the auxiliary structure of a hydroelectric power plant.

4.1 Châtelot Dam

4.1.1 General dam description

The Châtelot Dam is located on the Doubs River at the French-Swiss border. Built from 1950 to 1953 and operational since 1953, it is a double curvature arch structure 74 m in height. The main flood evacuation structure is a 65.50 m long spillway over the crest with two guiding walls separating it into three parts.
Figure 52 shows the plan view of the structure and its surroundings, as well as the cross section.

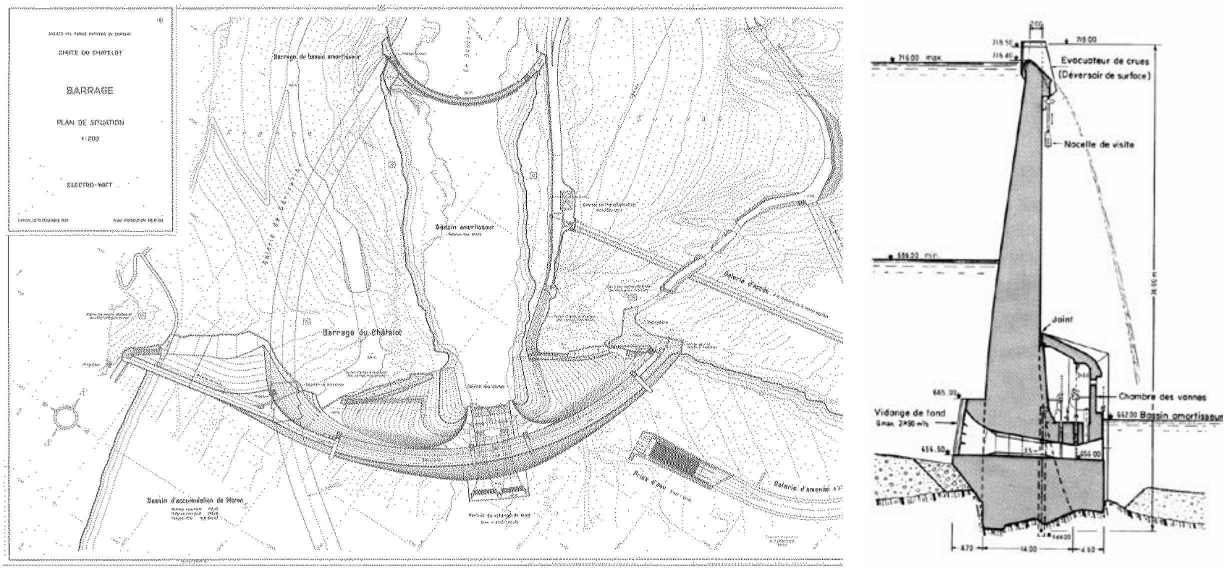


Figure 52: Plan view and cross section of the Châtelot Dam.

4.1.2 Dam behaviour

Figure 53 shows the displacements of the geodetic measurements at various points of the downstream face of the dam.

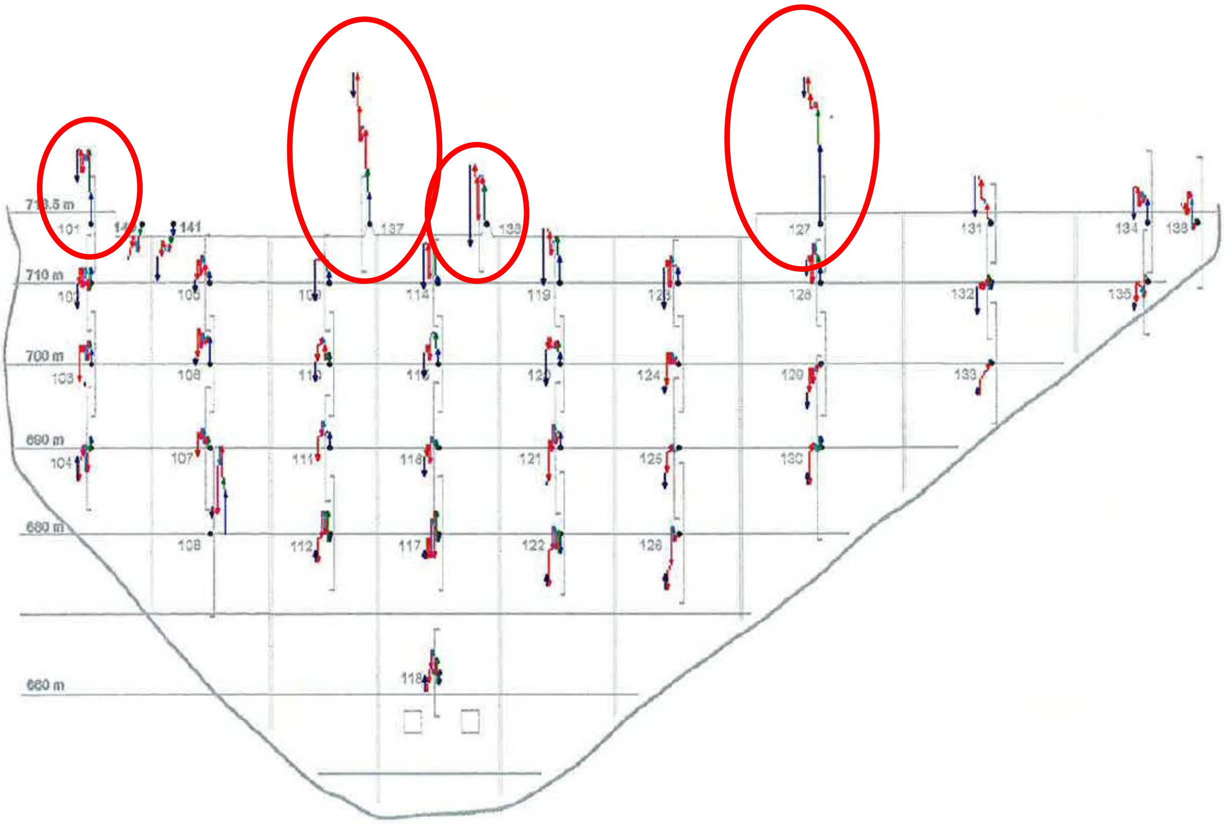


Figure 53: Displacements of the measurement points at the downstream face (1984–2008).

The greatest deformations during this period occur at the upper part of the dam. Thus, the previous figure illustrates the altimetric displacements of the targets on the dam and clearly shows that points on the dam crest, as well as those on guiding walls undergo significant vertical deformation, i.e. up to 3 mm of uplift since 1984. These vertical displacements are not clearly confirmed on the targets placed at lower levels.

This irreversible upward deformation seems to indicate an expansion of the crest material, composed of reinforced concrete, which is characterized by a dosage much higher than the dam body itself. The average annual rate of vertical deformation is 0.18 mm/year.

Expansion of the structural concrete by the Alkali-Aggregate reaction and affecting elements such as crest pavements and guide walls, is probable, as also suggest the guide walls showing a network of cracks with calcite precipitation.

Figure 54: View of the guiding walls: horizontal fissuring.

Investigations with sampling and testing are underway in order to confirm the hypothesis of Alkali-Aggregate reaction development in the upper crest elements.

4.2 Tourtemagne Dam

4.2.1 General dam description

The Tourtemagne Dam, located in the eponymous valley in Valais, is a single curvature arch dam. Built between 1957-1958, the Tourteagne Dam has a height of 33 m and a length of 115 m, of which 15 m are located on the left bank and 30 m on the right bank, corresponding to the gravity sections.
The arch portion of the structure has a constant thickness of 1.20 m, so that the total volume of concrete is only 3,200m3. Thus, to allow for such slenderness the dam has to be pre-stressed in both the horizontal and vertical directions.

Right flank
Left flank

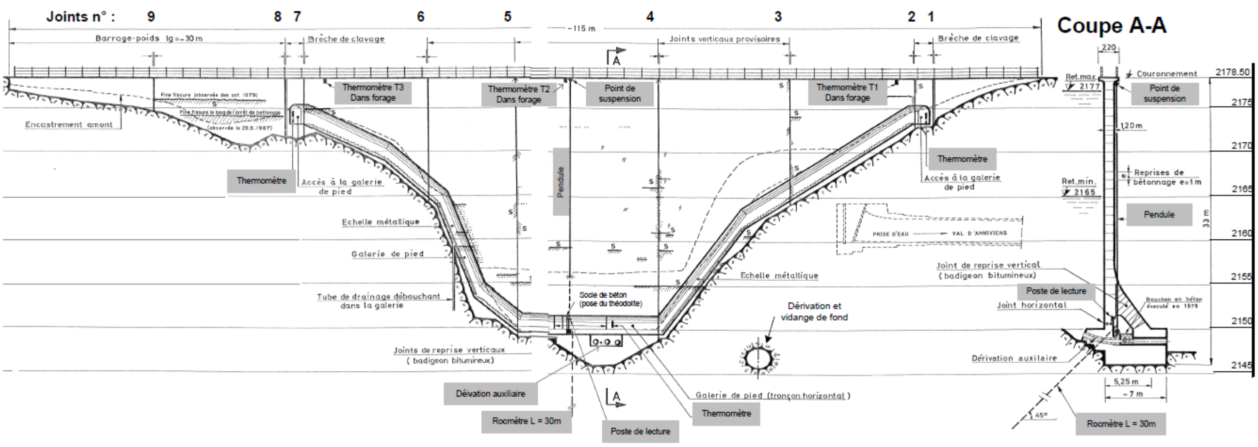


Figure 55: Downstream view of the dam – Vertical cross section.

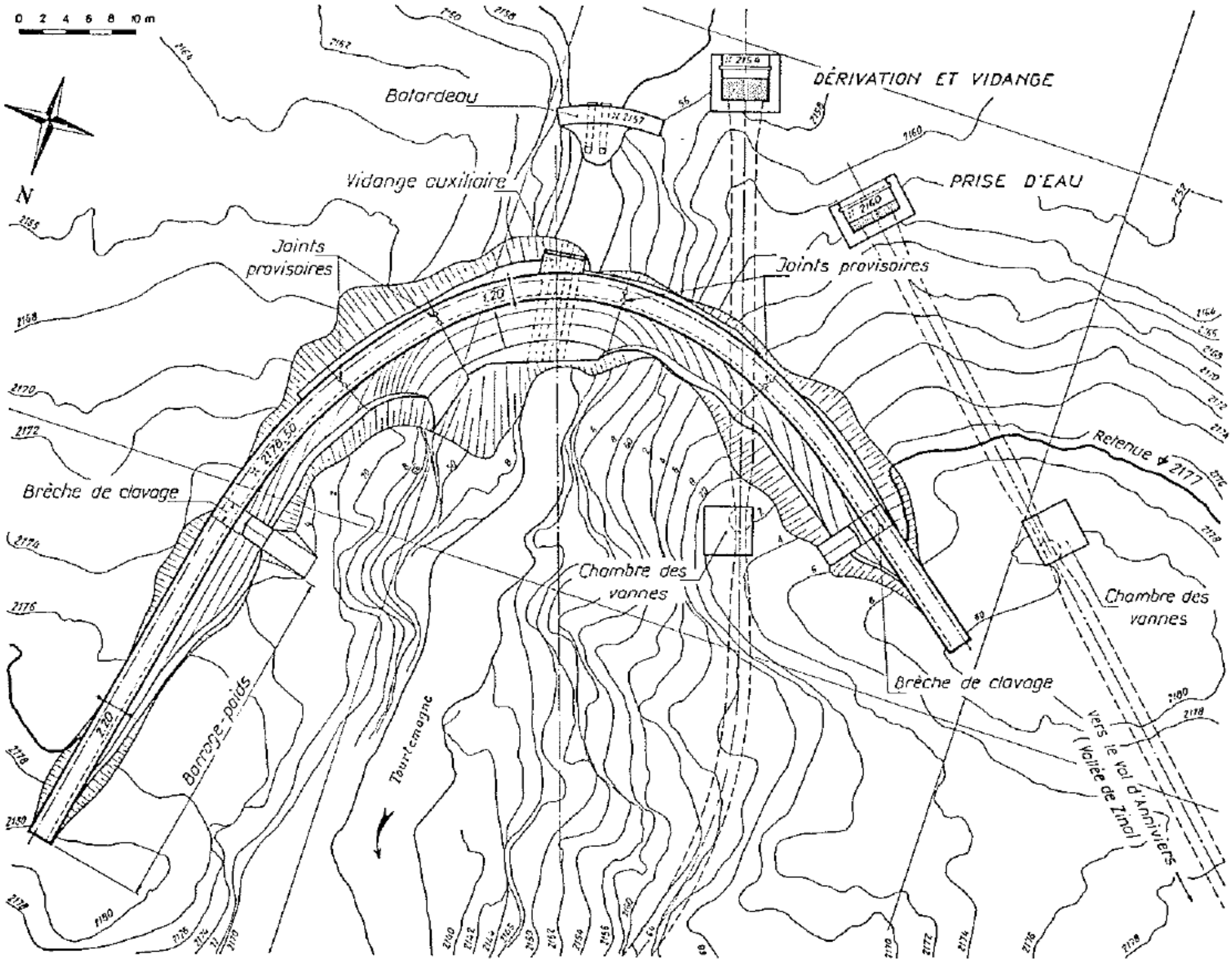


Figure 56: General plan view of the Tourtemagne Dam layout.

Figure 57 shows the pre-stressed cable network installed during construction.

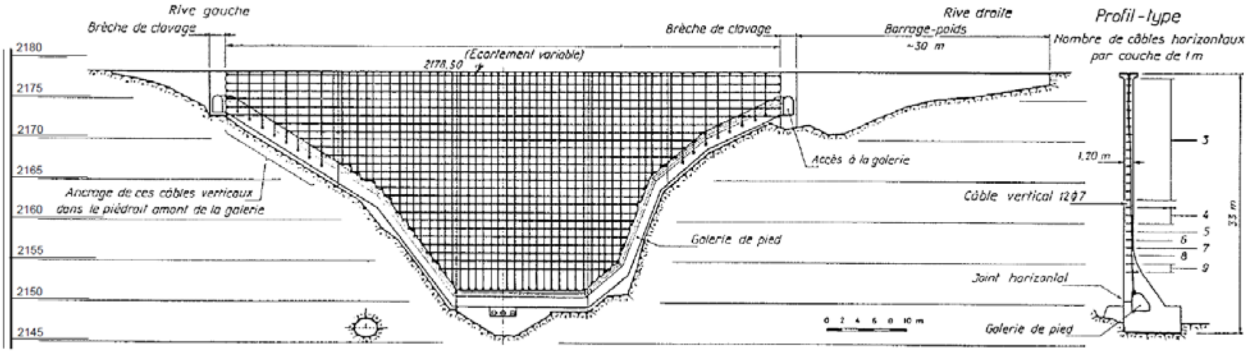


Figure 57: Upstream view showing the network of pre-stressed cables in the arch section.

The type of construction chosen and in particular the use of pre-stressing, were directly dictated by the severe thermic conditions to which the dam is subjected. The climate is rigorous and the very thin arch has a low thermal inertia. In winter, the reservoir might be completely empty so that the thin 1.20 m thick dam section undergoes intense cooling: the average temperature of the concrete can drop to -10 °C, causing

significant tensile stress in the arch. In summer, the reservoir can be emptied several times during the season, requiring the dam to alternate through cooling and heating. When the lake is empty, the vertical arches are under stress solely by the thermal effects. The arch is therefore subjected to tension during cooling. The application of a simple passive reinforcement would have brought only a questionable improvement: indeed, it would not prevent the formation of cracks in the concrete, but would only oppose their opening and despite its presence receding concrete would generate additional tensile strengths of 3 to 6 kg/cm 2 (0.3-0.6 MPa).
The chosen solution consists in exerting pre-stressing in the thinner parts of the structure with the intention of creating a normal force of permanent compression on all dam sections, whatever the reservoir level, thereby avoiding the risk of cracking and maintaining the residual traction due to thermal effects at acceptable levels.
The arch was subjected to biaxial pre-stressing through a double system of both horizontal and vertical cables, which neutralized most of the parasitic forces:

- The network of vertical cables compresses the horizontal sections of the arch and opposes bending resulting from embedding effects in the structure base.
- Horizontal pre-stressing was obtained by means of curved cables following the shape of the arches. Various disadvantages inherent to this pre-stressing method (tension on the abutments, shortening of the arches causing a general displacement downstream of the arch, and consequently bending at the base of the structure) have been ruled out by injecting four temporary vertical active joints, separating the arch into five pre-fabricated segments. These joints, equipped with flat cylinders to compensate for the shortening of the segments due to pre-stressing, were sealed after compression and once the required key temperature of +5 °C was reached.

Figure 58: Aerial view of Tourtemagne Dam and flood discharge structure on the right bank.

4.2.2 Arch behavior

Movements of installed pendulums along the downstream face of the structure, in the central section, show a slight irreversible displacement over the years. However, unlike irreversible upstream displacements that one can observe on arch dams subject to swelling, in the case of the Tourtemagne Dam these displacements occur downstream, at least at the pendulum level. The cause is probably due to concrete swelling which leads to an increase in tension in the pre-stressing cables which hold the dam downstream at its center. Irreversible displacements upstream are detected by geodetic measurements on both sides of the central section.

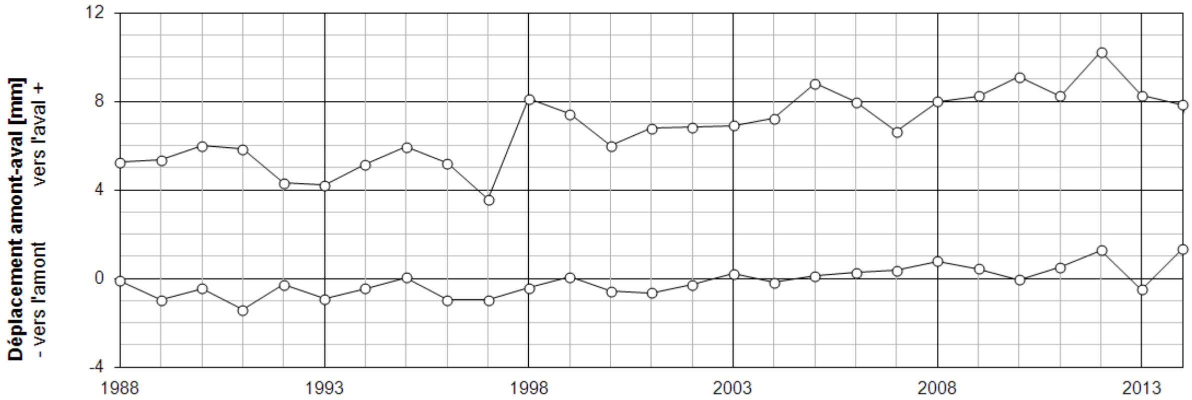


Figure 59: Extreme displacements (min and max) upstream-downstream of the central pendulum.

On the other hand, dam crest levelling measurements show a vertical displacement towards the top of the crest.

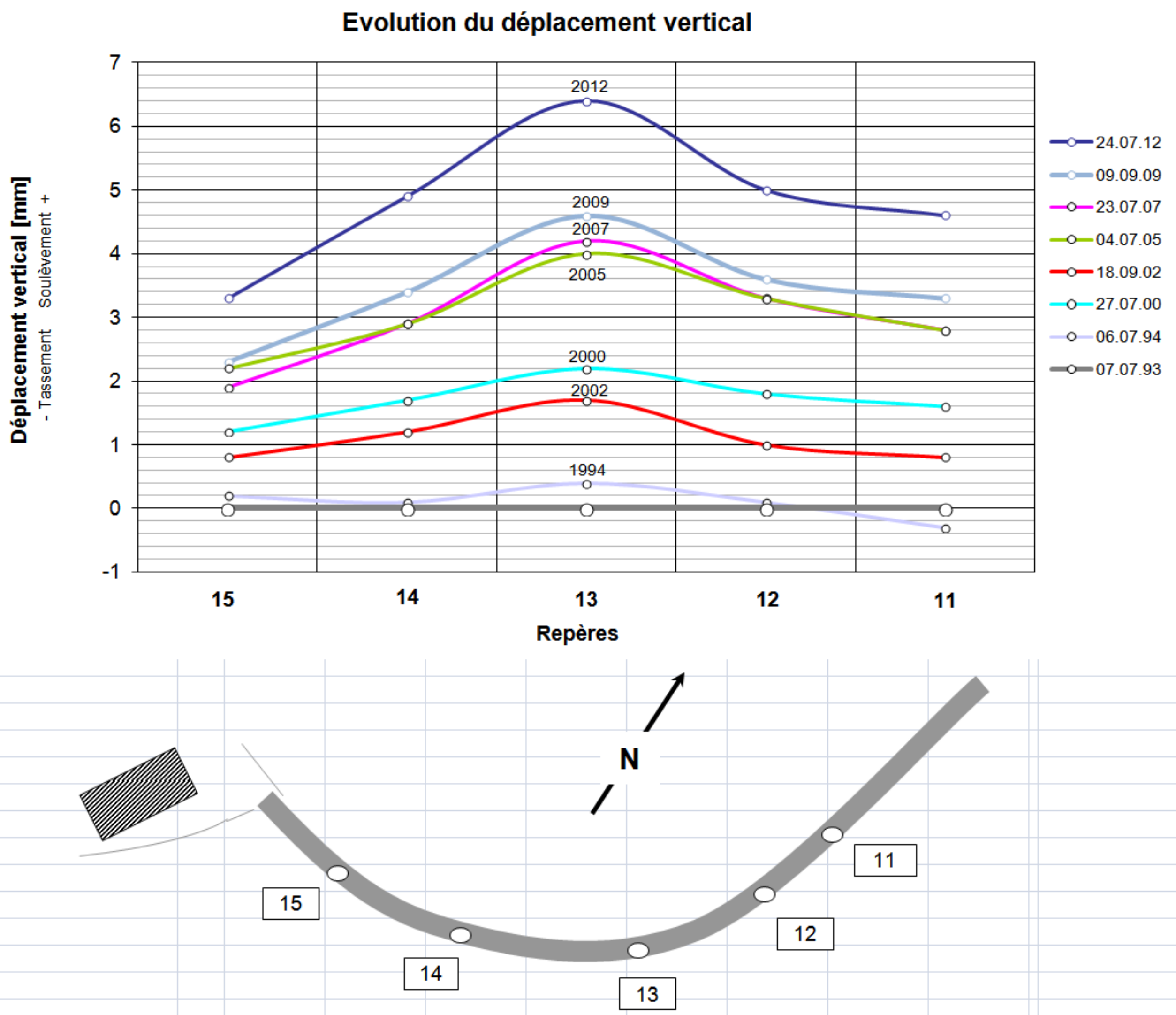


Figure 60: A quasi-linear evolution of displacements of various leveling points makes it possible to estimate the rate of displacement relative to the maximum height of the dam structure at 10.3 µm/m/year.

4.2.3 AAR identification

Such displacements seem to cause an internal concrete swelling reaction. It should also be noted that some associated dam structures show typical Alkali-Aggregate reaction surface conditions of (cracking fissuring).
Sample and laboratory tests are underway to confirm the presence of AAR.
In addition, numerical simulations using the thermal expansion analogy were undertaken. This approach did not perfectly reproduce observed behaviors i.e. minimal upstream-downstream displacements in the center, and notable vertical displacements, whereas the numerical results showed similar displacements in the vertical and horizontal directions. Using a thermal analogy of the AAR effect to reproduce the state of deformation is acceptable when the anisotropy of the state of stress is not too marked.

4.3 Veytaux pumped storage plant

The Veytaux power plant is part of the Hongrin-Léman project. The power increase developments are being finalized.
The Veytaux plant was completed in 1971. Parts of the plant are constantly submerged such as the outlet channels or the pits with the installed pumps. It is in these parts that the typical surface conditions of AAR have been identified twenty to thirty years after being operational. A systematic survey campaign was undertaken in the early 2000s, accompanied by sampling and testing.

Figure 61: Surface condition of certain walls of submerged parts of the power plant.

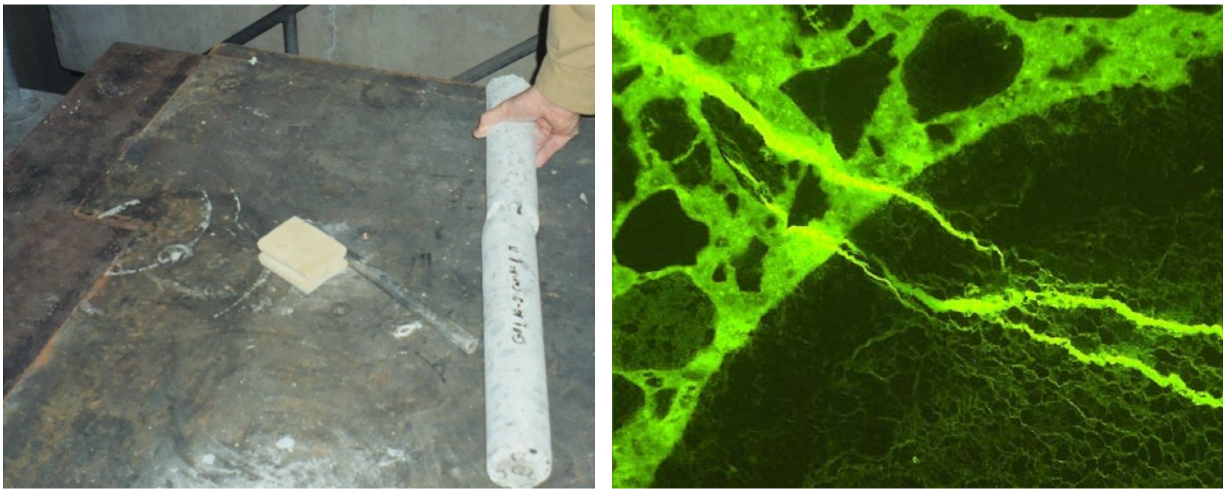


Figure 62: Sample collection - Microscopic analysis.

In addition to the mineralogical and microscopic analyses, accelerated concrete swelling tests have been carried out. The purpose of these tests was to confirm the presence of AAR, to assess its progress and to estimate the residual swelling potential.

Moreover, given the influence of moisture and temperature on AAR swelling, continuous temperature and relative humidity measurements were made over several years, in parallel to some internal structure deformation monitoring with invar wires.

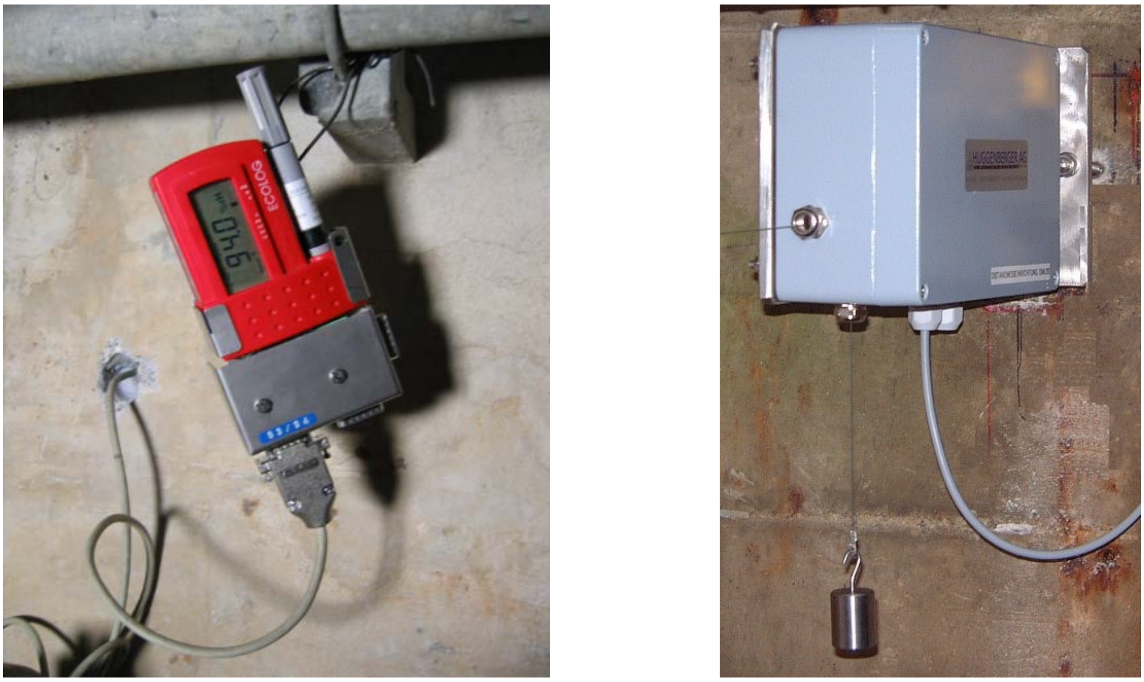


Figure 63: Continuous temperature and relative humidity measurements - invar wire deformation measurements.

Collection of thermal and water data, as well as sample analysis results made it possible to model past and future deformations of the plant building blocks, after adequate calibration of a chemo-mechanical model (see [19] et [20]).

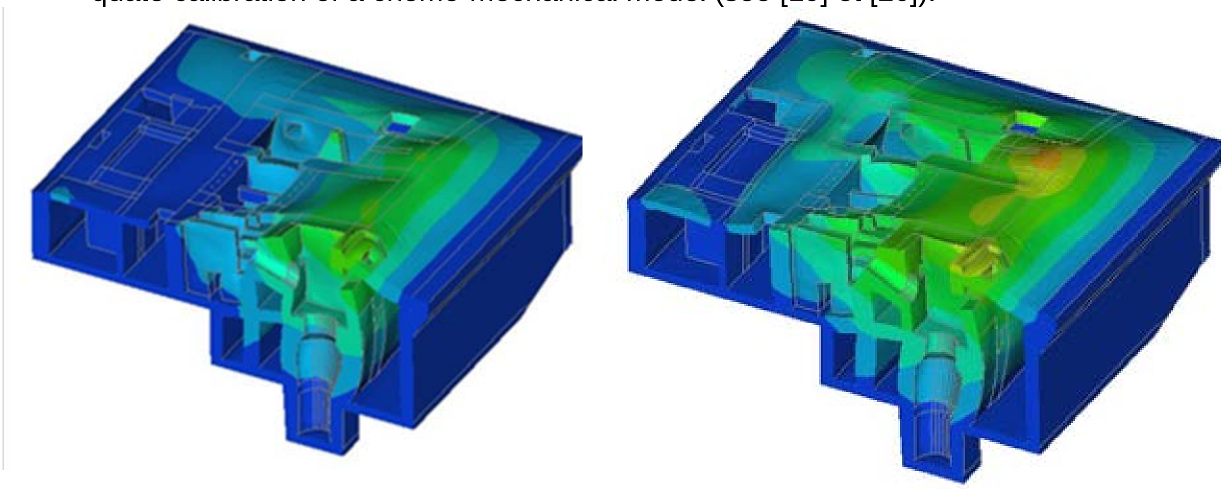


Figure 64: Deformation of block 1 of the plant in 2014 and 2070.

Based on the obtained results, stress relieve measures of the hydro-mechanical manifold and the unit pipes were carried out in 2014.

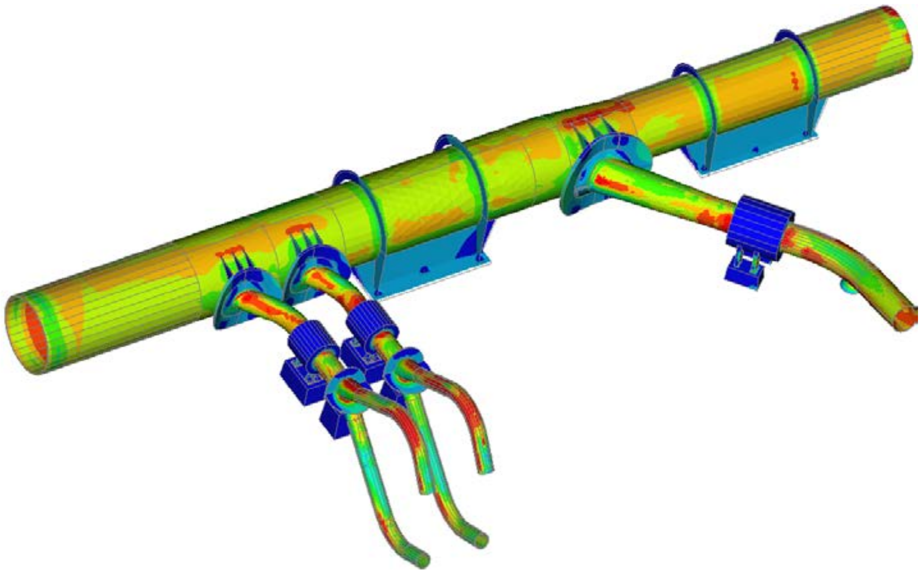


Figure 65: Stress state of the load distributor and the pipes of the group 1 calculated from AAR-generated deformations.

4.4 Diagnostic conclusions

In general, demonstration of irreversible dam structure displacements that are not attributable to causes related to the behavior of the structure base for example, is the first indicator leading to the suspicion of the presence of internal concrete swelling. The observation of fissuring or the occurrence of significant cracking generally takes place only after irreversible displacements have been detected. This confirms the need for rigorous monitoring using appropriate means for detecting dam displacements. The redundancy of the means of surveillance, for example through pendulums and geodetic measurements ensures more accurate monitoring results. Sampling and petrographic analysis are required only at later stages in order to confirm or refute the presence of AAR.

5 Intervention examples

5.1 Sera Dam

5.1.1 General dam description

The "first" Sera arch Dam near Gondo in the Zwischbergental, operational since 1952, creates a compensation reservoir with a capacity of 175,000 m^3. The reservoir provides daily flow regulation used by the Gondo power station, owned by the “Energie Electriques du Simplon” (EES).

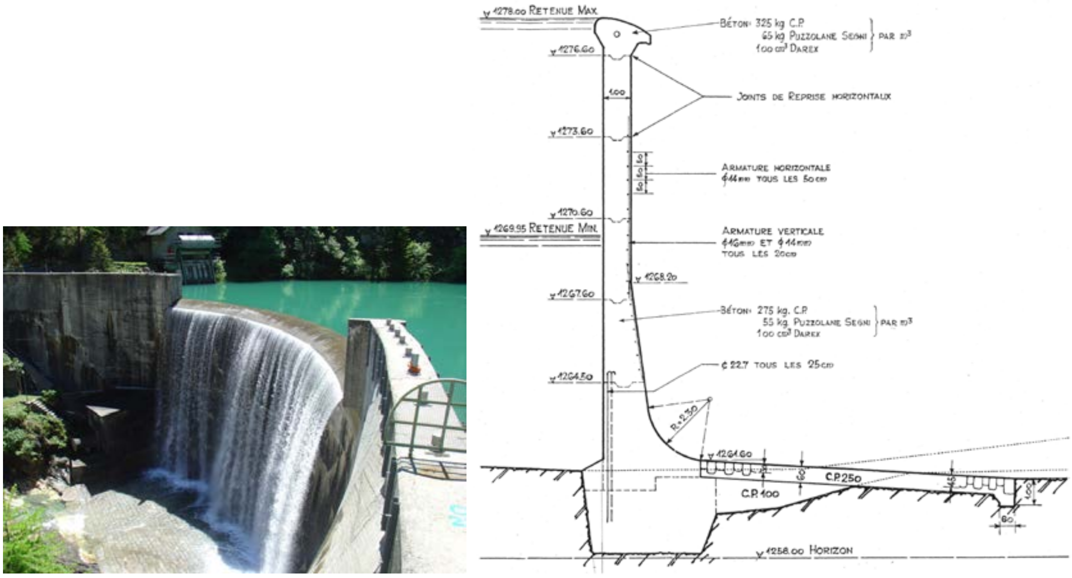


Figure 66: Sera Dam – cross section

Height: 20 m
Crest length: 75 m
Concrete volume: 2‘300 m^3
Crest elevation: 1’280.4 m a.s.l.

5.1.2 Behavior and operation of the structure

Since its commissioning in 1952, monitoring of the dam's behavior has mainly been done using geodetic and levelling measurements. Concrete from the Sera Dam affected by an Alkali-Aggregate reaction leading to concrete swelling, has caused irreversible deformation upstream, accompanied by dam heaving and diffuse cracking. The maximum upstream displacement in the order of 60 mm was much higher than seasonal effects. The upstream deformation has led to the appearance of several structural cracks on the downstream face, corresponding to concreting times in the right foundation part.

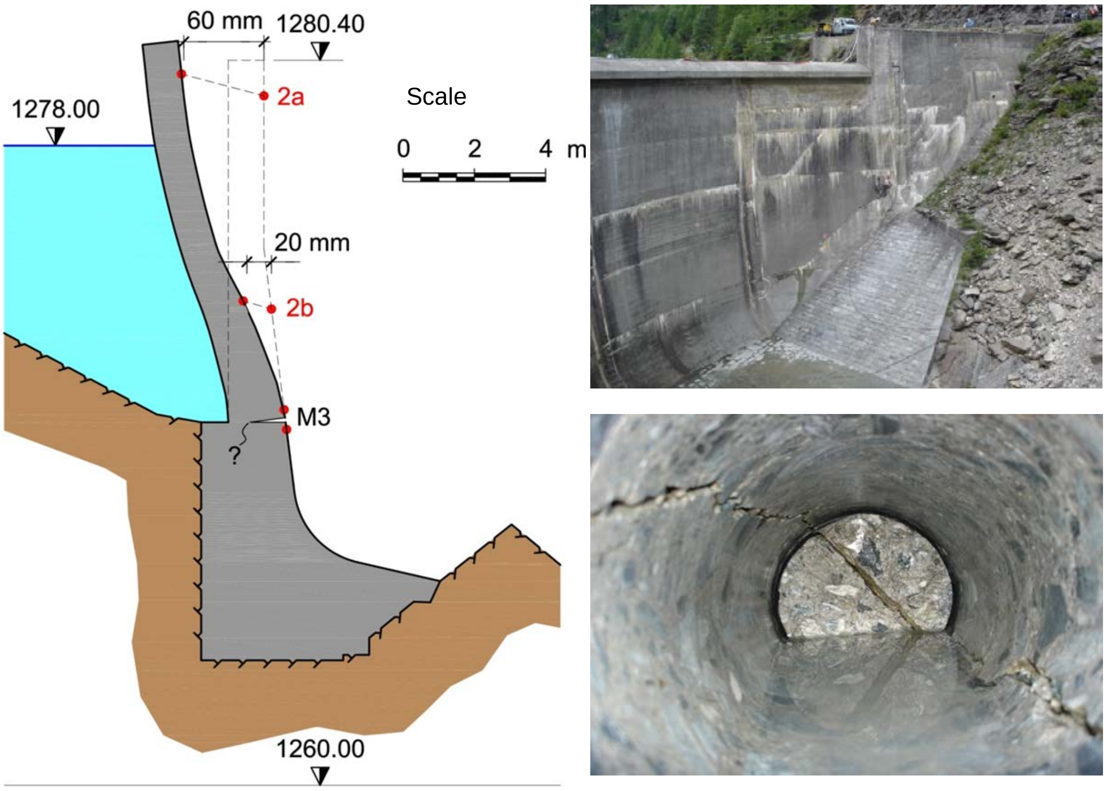


Figure 67: Cross section of vertical displacement towards upstream – development of cracking at the dam base.

On the basis of this observation the EES has evaluated the condition of the dam and its operational readiness in 2006. The results and conclusions of this evaluation showed a gradual deterioration of the operational conditions as well as the safety of the structure. However, with provisional reinforcements and adaptation of the operational procedures, the operational conditions remained satisfactory in the short term. Temporary reinforcement works consisting essentially in the installation of vertical anchors from the dam crest were carried out in autumn 2006.

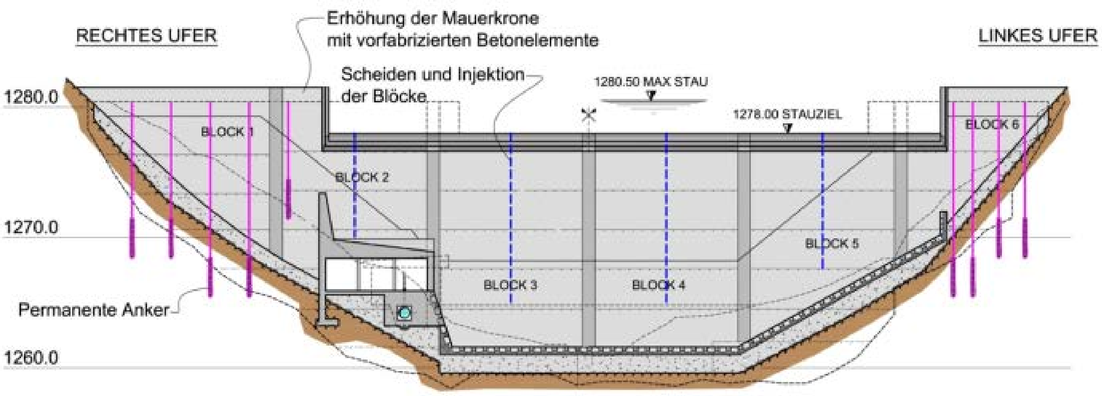
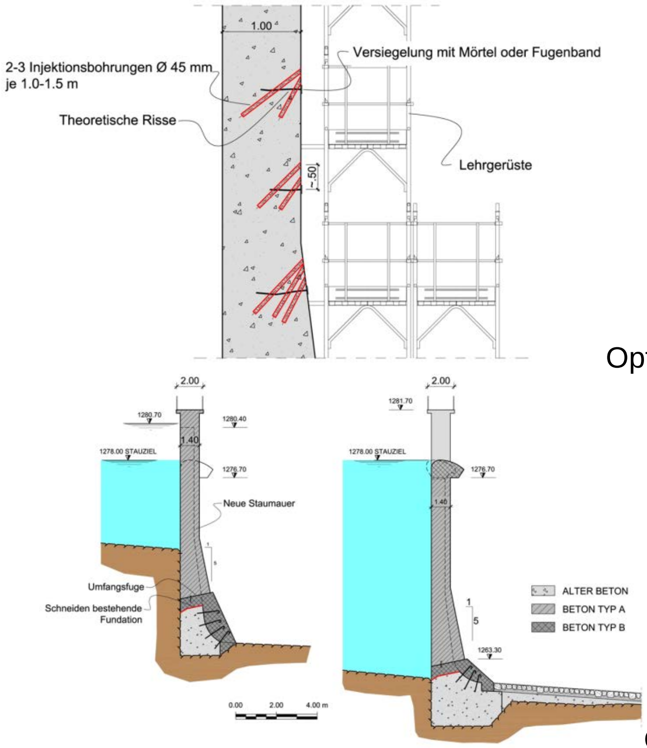


Figure 68: Temporary reinforcement – set-up of the vertical tension anchors.

The evaluation recommended a definitive reinforcement of the dam for the period 2008/2009. In December 2006, the EES launched a definitive rehabilitation project.

5.1.3 Rehabilitation

Three variants have been proposed. The first consisted of a reorganization of the dam aimed at recreating a monolithic structure by injection. This measure was accompanied by a reinforced inspection. The second variant involved the reconstruction of a new dam on the existing foundation; the latter being shrink fitted and adapted to allow for future swelling at the level of the seal between the dam and its foundation.



Option 1

Option 2 sloping

Figure 69: Rehabilitation options.

The selected rehabilitation solution consisted in creating a new dam downstream of the old dam. Several configurations and locations of the new downstream dam from the former one have been analyzed. The construction of a new dam support located a few meters downstream of the old one does not pose difficulties on the left bank, whereas the sloping topography of the right bank is less favorable.
The new dam was built entirely downstream from the old structure. In order to avoid having to position the foundations on the right bank at an unfavorable location (with very low angles of incidence and a very steep slope), the new dam partially intercepts the old structure. Prior to construction, partial demolition of the downstream base of the old dam was necessary.
Given a new foundation was constructed it was possible to choose a new more favorable geometry, compared to the one of the old dam structure. From a structural point of view, the Sera dam is similar to a double curved arch dam. Its shape is optimized so as to avoid reinforcement using additional armatures.
The final geometry results from a compromise between several requirements:

- A constrained area, with the old dam upstream and a micro cliff downstream making the river bed drop by 10 m;
- The sloping topography on the right bank, which significantly reduces the angle of incidence of the arch support structures on the rock;

The arches of the new dam, whose axis is inclined 7 degrees in a clockwise direction relative to the axis of the old structure, are elliptical, so that their curvature varies gradually from the center towards the supporting structures.
The new dam crest was raised by 1.30 m compared to the old dam, reaching a height of 1'281.70 m a.s.l.. This raise was necessary to ensure sufficient safety in the case of flooding. The normal reservoir level of 1'278 m a.s.l. remains unchanged from the

old structure. The overflow section is equipped with a 2.0 m wide footbridge making it easy to connect the two banks of the dam.
The dam is composed of six independent blocks of varying lengths between 12.0 and 16.5 m. The structure is made monolithic through joint grouting between the blocks.

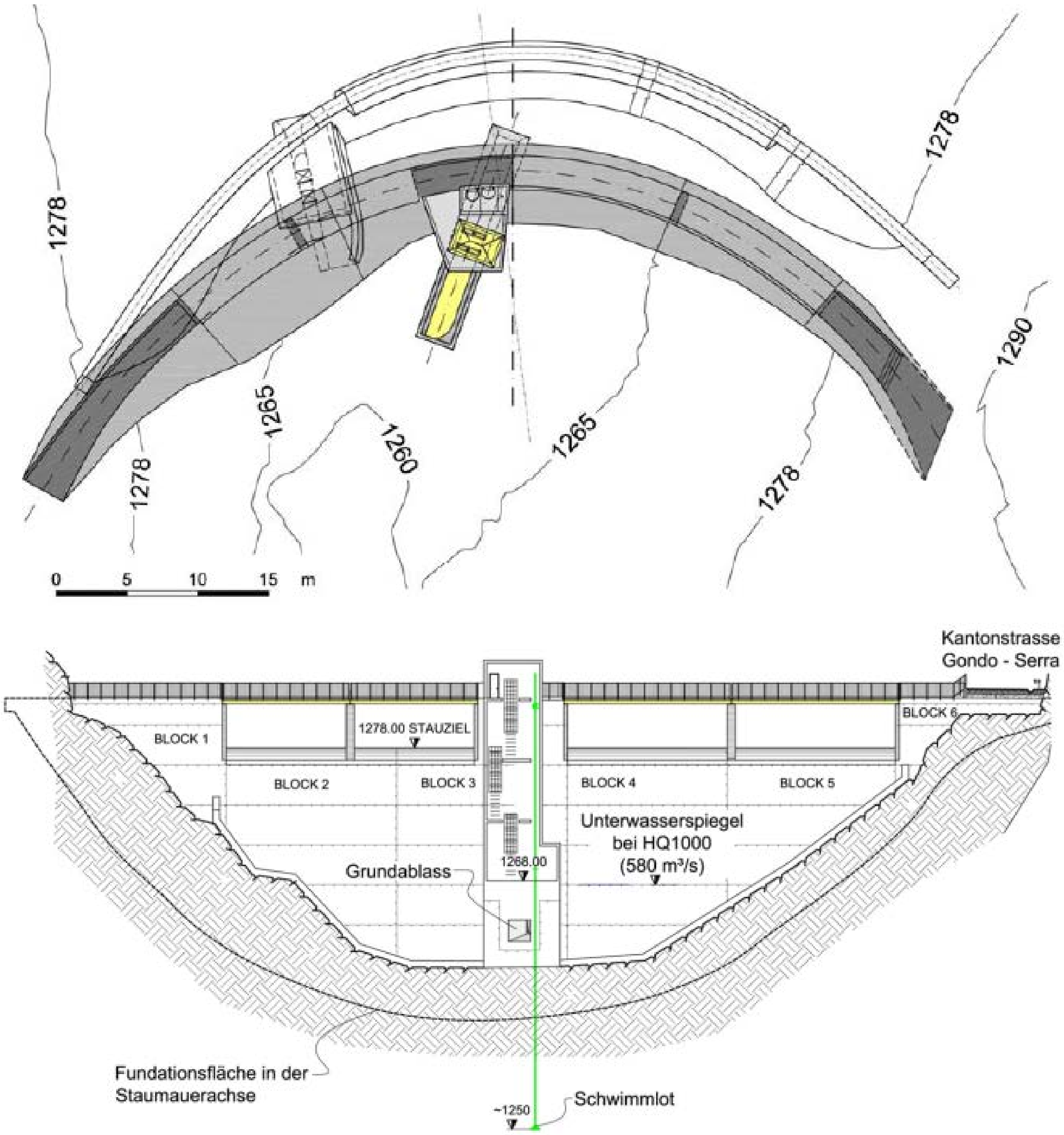


Figure 70: Plan view and new Sera dam raise.

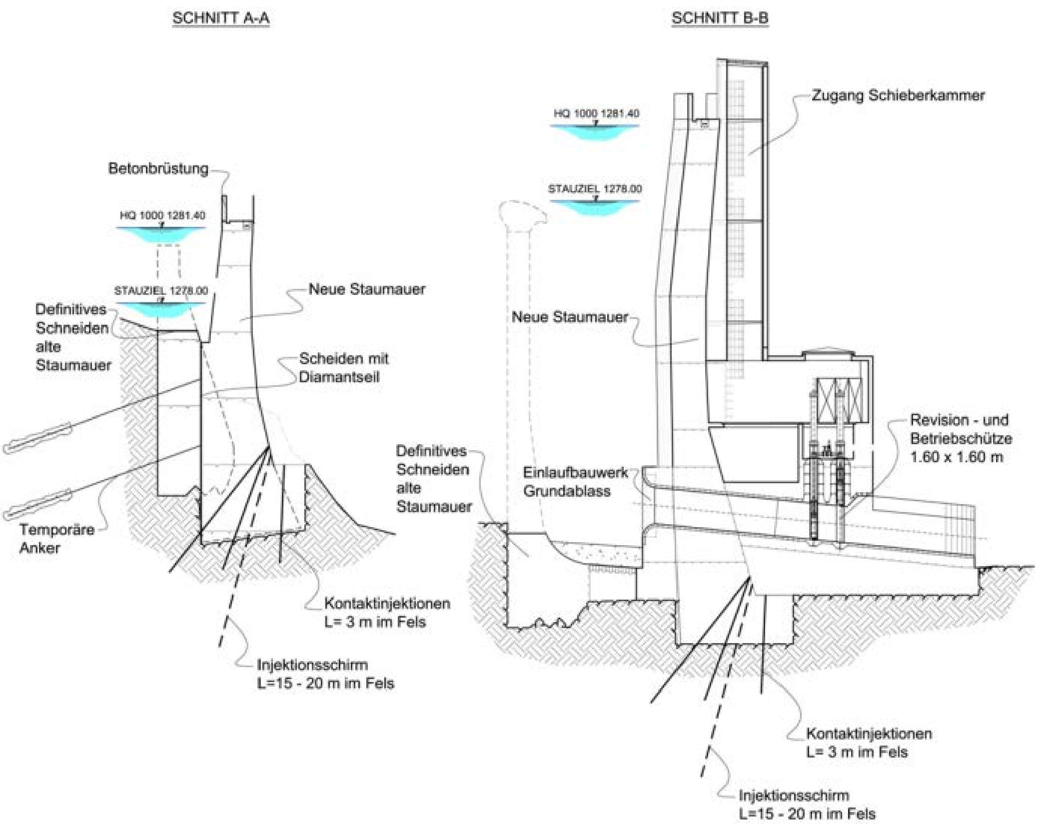


Figure 71: New Sera dam cross section.

Figure 72: Construction work on the new Sera Dam, just downstream from the former structure.

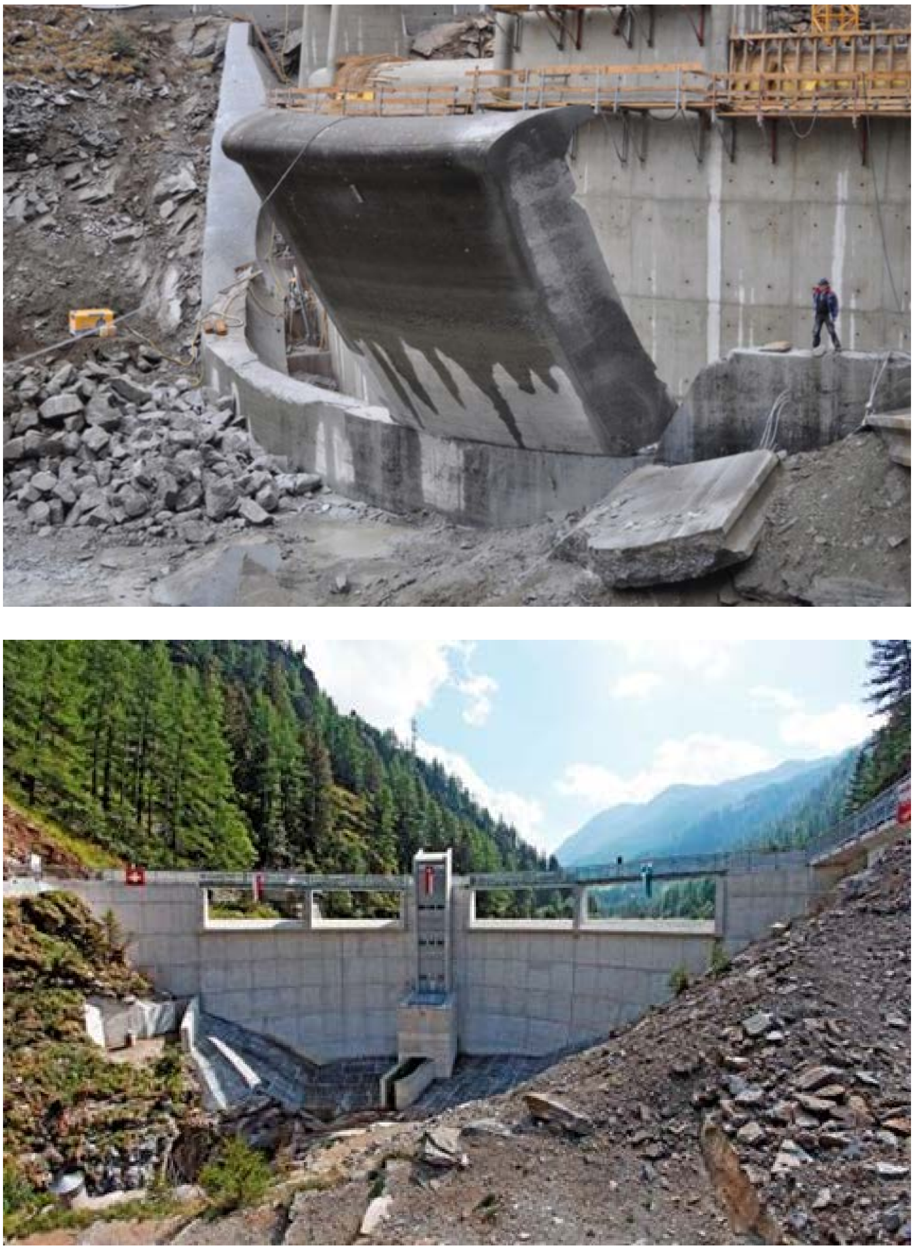


Figure 73: Demolition of the first dam and downstream view of the new Sera dam.

5.2 Salanfe Dam

5.2.1 General dam description

The Salanfe Dam is located at 1'925 m a.s.l. on the southern flank of the Dents du Midi in Valais (CH) and is of the gravity dam type. The structure is divided into four straight sections: a central section 260.65 m long, a straight part in two sections of 72.5 m and 76 m, and a left part of 189.50 m.

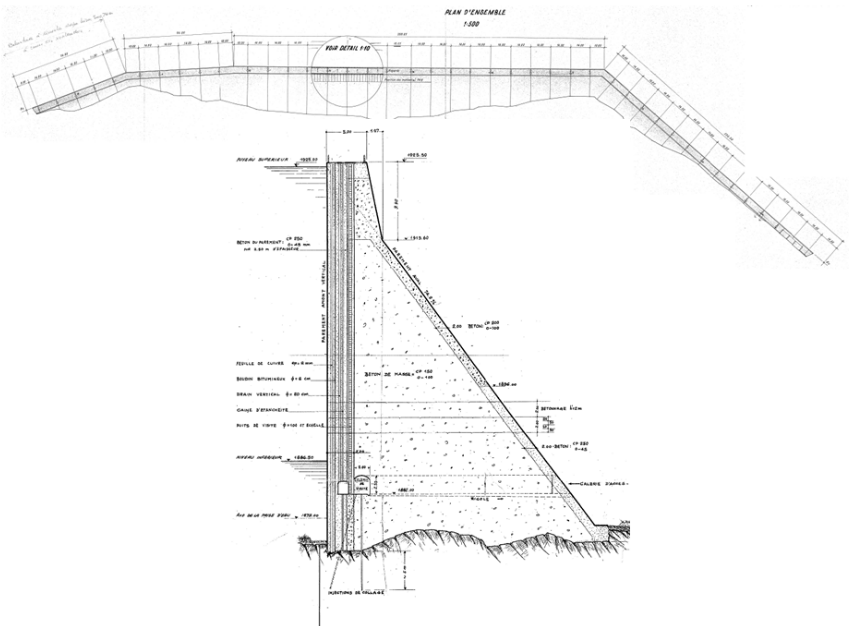


Figure 74: General plan view and vertical cross section of the Salanfe Dam.

- Number of blocks: 41
- Block width between 13 and 20 m
- Max height on base: 52 m
- Crest edge: 1’925.50 m
- Total crest length : 598.65 m
- Crest thickness : 5.00 m

5.2.2 Observed behavior

Pendulum measurements carried out at the end of the construction of the dam show a crest displacement of the structure upstream, which is getting increasingly marked since the 1970s. In order to refine the understanding of displacements of the dam other instruments of which additional pendulums were installed in 1993-1994 and in 2007. Levelling of the crest has also shown a progressive rise of the structure. In addition, visual inspections of the dam revealed the appearance of cracks, particularly in the lateral blocks on the left and right banks of the structure and horizontally in the first 5 meters under the crest. Cracking of the crest concrete also appeared. These clues have led to the suspicion that the Salanfe Dam was subject to concrete swelling related to the Alkali-Aggregate reaction process before 2000.

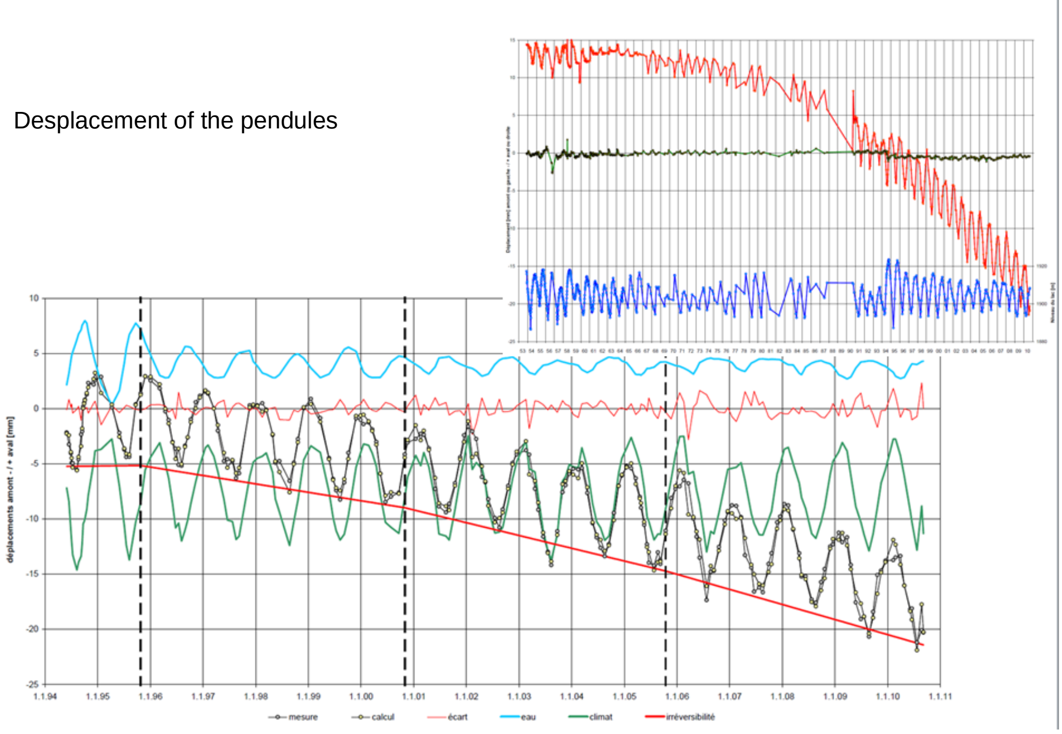


Figure 75 : Longitudinal and transversal displacements. Statistical analysis of the pendulum behavior.

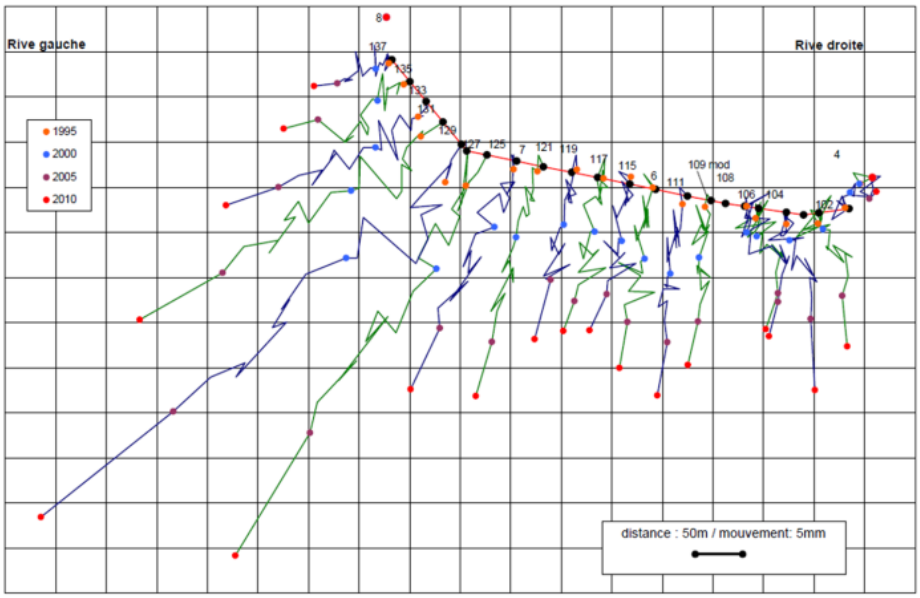


Figure 76: Planimetric crest displacements through geodetic measurements .

5.2.3 Research, concrete diagnosis - conclusions

Several sampling and analysis campaigns have been undertaken in order to:

- identify the swelling causes of the dam concrete
- verify the mechanical concrete properties and the effect of its degradation
- to characterize the state of the reaction and its development potential
- to define complementary inspection measures

Research has brought to light the development of an alkali-aggregate reaction, the presence of numerous micro cracks in the cement and the interfaces between the binding agent and the aggregates.

Following the proven diagnosis results, the residual concrete expansion potential of the structure was evaluated. A distinction between the two concretes (facing and mass) by attributing different behavioral trends in relation to AAR, with different reaction kinetics, or different final expansion and degradation potentials was not required. Based on visual observations, 60-year pendulum displacement measurements, and dam sample testing, one can attest that the Alkali-Aggregate reaction is active, and the reaction potential is still present. The heterogeneity of the results obtained is attributable to the different concretes, and to a dispersion of the swelling.

5.2.4 Rehabilitation principles

Concrete swelling generated by the Alkali-Aggregate reaction leads to significant deterioration of the structural and mechanical properties of the dam structure, and causes irreversible cracks and deformations. In order to limit these negative effects, it has been planned to unload the concrete by vertically sawing it in the upstream-downstream direction. Although not definitive, this method is the only viable one that can extend the life span of the structure. Given the complexity of involved phenomena, including the mechanical properties of materials, their physical-chemical behavior, material damage, and creep, it became necessary to resort to detailed modeling of the different phenomena. It was agreed to carry out a sawing testing campaign one year prior to the main works, not only to validate implementation, safety and environmental protection procedures, but also to calibrate some parameters of the numerical model.

Figure 77: Tunnel at the base – Fissuring.

5.2.5 Modeling

The objective of the numerical model of past and future behavior (after sawing) was to:

- Understand dam behavior
- Estimate the future swelling effects in configurations with and without sawing
- Define involved parameters and their sensitivity
- Define various characteristics of interventions
- Provide decision-making support regarding the rehabilitation strategy

Behavioral laws:
The model is based on the work undertaken at LCPC (IFSTTAR) integrating the swelling kinetics law of the sigmoid type, including concrete thermo-hydric parameters. Concrete swelling takes into account the average stress, and the stress deviator to determine the chemical expansion in each of the directions in space. Specific developments have been made in order to take into account all of the long-term behavior of structures, particularly those enduring major permanent loads such as dams. It was therefore necessary to consider deferred material deformations, namely the effects of creep, its interactions with the development of swelling and mechanical effects. Moreover, in order to model the effects of sawing, such interventions had to be modeled by developing contact elements simulating the sawing width. Finally, in order to account for elasticity degradation in relation to the chemical reaction progress, a damage variable indexed on AAR-induced expansion is introduced [21].

Total deformation is defined by:

$$\varepsilon = \varepsilon_e + \varepsilon_{flu} + \varepsilon_\chi$$

- Elastic deformation:

$$\varepsilon_e$$

- Deformation due to creep:

$$\varepsilon_{flu} = \phi(t, t_0) \,.\, \varepsilon_e$$

- Deformation due to AAR:

$$\varepsilon_\chi = \varepsilon_\infty \cdot \frac{1 - e^{-t/\tau_c}}{1 + e^{-(t-\tau_l)/\tau_c}}$$

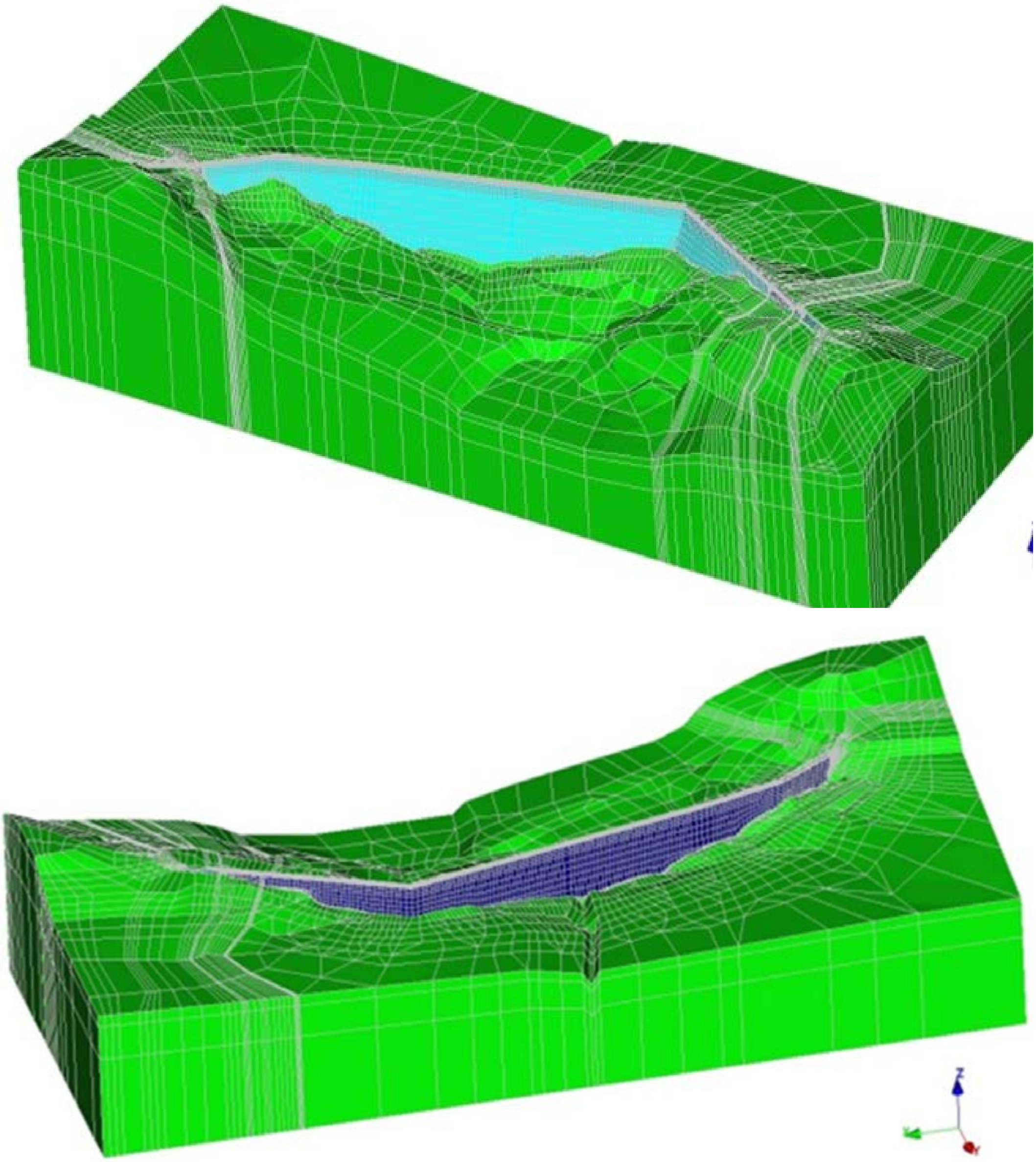


Figure 78: Modeling.

5.2.6 2012 testing campaign

Since sawing works are generally not being carried-out, a test campaign was launched in summer 2012 with the aim of:

- Identifying difficulties and risks
- Defining operational methods
- Verifying dam behavior after the first sawing cuts
- Validating the model
- Confirming the strategy

The sawing tests were carried out in 3 vertical planes. The thickness of these 3 sawing cuts was 15 mm, a height of 16 m and made from bottom to top, horizontal drilling having been done beforehand. This test campaign, conducted in 2012, not only allowed the engineer to stall his digital model, but also the entrepreneur to test his methods and equipment. Following this campaign, the main sawing operations were carried out in 2013 (see [22] and [23]). The secondary works (crest restoration, bottom outlet ...) were partially completed in 2014.

5.2.7 Works undertaken between 2013–2014

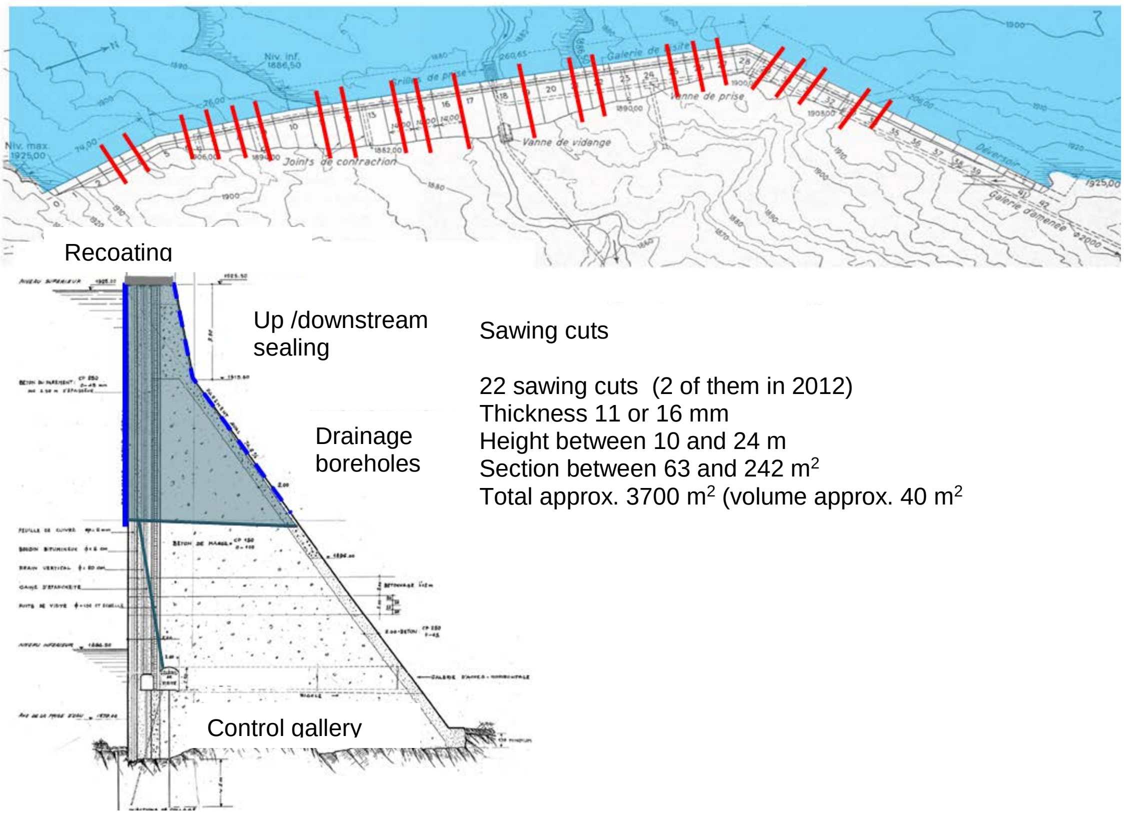


Figure 79: Plan view of saw cuts - typical cut.

Thus, the works undertaken on the dam were not solely limited to sawing, but also addressed:

- 21 sawing cuts (2 of which deepened cuts made during the 2012 testing campaign);

- Sealing of both the upstream and downstream faces;
- Core drilling of drainage holes behind the upstream seal from the horizontal tunnels;
- Restoration of the dam crest (including the installation of a new railings in compliance with prevailing standards);
- Demolition and construction of a new covering structure for the bottom outlet shaft;

Figure 80: Sawing.

Figure 81: Preparation and installation of upstream sealing.

6 Development forecasts in Switzerland

6.1 General considerations

Any dam that demonstrates non-reversible behavior must be given special attention in order to identify the reasons causing it. **Figure 82** illustrates the typical monitoring cycle of a dam affected by concrete swelling.

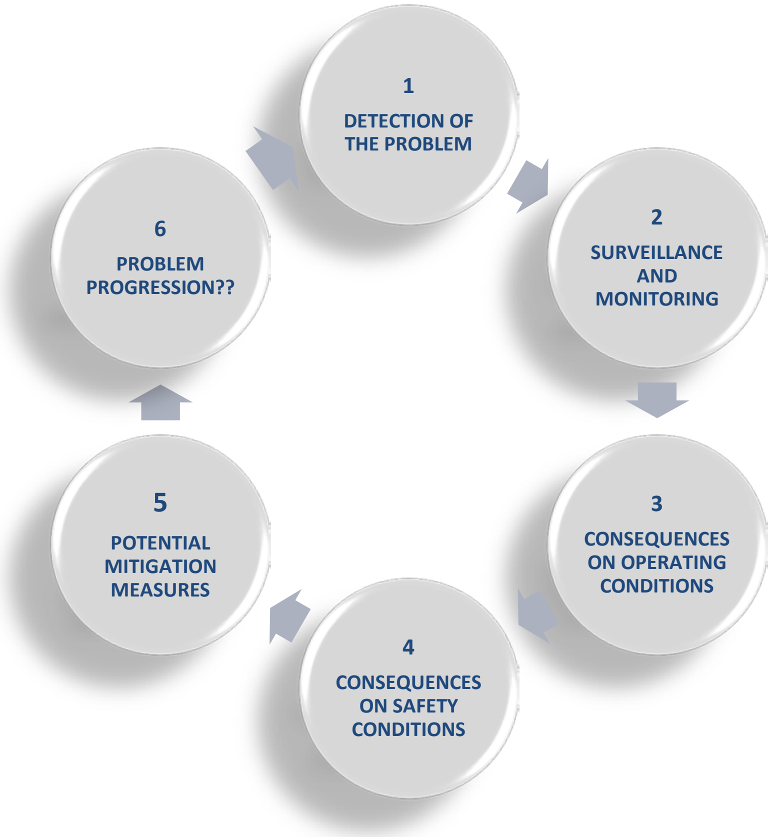


Figure 82: Typical monitoring cycle of a dam affected by concrete swelling.

It is usually possible to distinguish between 6 main stages:

- Stage 1: Detection of the problem.

  Initial detection or suspicion of concrete swelling is almost always linked, with very few exceptions, to the demonstration of non-reversible deformation. It is important to note, however, that the causes of non-reversible deformation can be multiple and very diverse. For this reason a very careful analysis of the dam monitoring measures is necessary in order to identify the causes of non-reversible behavior. After excluding other causes such as bank instabilities, movements at the foundation etc. it has often been concluded that concrete swelling was the cause of such deformations.

- Stage 2: Surveillance and monitoring.

  If dam monitoring and observation analyzes seem to reveal concrete expansion, it is usually necessary to consolidate this hypothesis with complementary measurements to identify essential characteristics. Identifying additional monitoring and specific investigation measures that quantify concrete swelling are therefore necessary at this point.

- Stage 3: Consequences on operating conditions.

  Once the reasons for non-reversible dam behavior are well established and studied, it is possible to make some assumptions about its evolution in the short and medium term, especially with regard to non-reversible deformation. On the basis of past behavior, it is generally possible to estimate the progression of non-reversible deformation over an indicative period of 5-15 years. This process first has consequences for the operating conditions before affecting the safety of the structure, and in particular it can affect the state of concrete cracking, operation of moving parts, water infiltration etc. Once the consequences on the structure have been identified, it may be appropriate to intensify specific control measures that allow for detailed monitoring of the evolution of each specific aspect.

- Stage 4: Consequences on safety conditions.

  The consequences of swelling progression on dam safety conditions are much more difficult to establish. Indeed, safety depends on a large number of factors, and requires a modeling of the cracking scenarios (partial or total) of the dam and / or of secondary structures, among others. Safety issues can have a multitude of origins (structural, hydraulic, mechanical, etc.). Determining the stress state in a dam that is subject to swelling produces relatively uncertain results and modeling of the cracking scenarios is not an easy task, in particular for arch dams. If the static loads are added to the dynamic loads, the problem increases in complexity. Assessing the influence of swelling on dam safety conditions is therefore a task that cannot be tackled solely by an analytical approach, but also requires sensitivity and specific experience with dams. An essentially analytical evaluation of the safety conditions of a dam can lead to a partial assessment that overlooks the more difficult elements to be quantified, especially concerning dams subjected to non-reversible deformation.

- Stage 5: Mitigation measures

  Following previous studies, specific mitigation measures against the effects of swelling can be envisaged. There are multiple reasons and objectives for these interventions, but it is important to distinguish whether the measures are aimed at improving the operating conditions or the safety of the dam. As indicated in previous chapters, these measures may be very different in nature and are dependent on the characteristics of the structure.

Figure 83: Examples of mitigation measures: a) carbon fiber protection; b) protection with PE membrane.

- Stage 6: Progression monitoring.

  Except in the case of demolition and reconstruction of part, or of the entire structure, interventions usually focus on the consequences of swelling, and only in exceptional cases on its causes. Mitigation interventions are therefore usually temporary, and their effectiveness is progressively reduced over time.

6.2 Future approach to the problem

Analysis of the AAR problem has made significant progress, particularly during the last decade. A great deal of research has come into being mainly to better understand the chemistry of the reaction and its consequences on concrete. Numerous models have been developed in order to simulate various processes among which deformations induced by concrete expansion.

Despite these important progress milestones, the problem is far from being satisfactorily under control for dam operators. The latter would like to know if, when, and how they should intervene on dams affected by chemical concrete swelling. A reliable answer to these questions is currently not available, particularly in the medium to long term. From a practical point of view, the main limitations of current approaches can be summarized as follows:

- Current laboratory tests do not permit the actual detection of the phenomenon, but provide at best, confirmation of its potential.
- The overall consequences on a dam are usually easily identifiable. Determination of the state of non-visible parts on the other hand is way more complex. For instance, there are no non-destructive solutions that make it possible to detect the presence of cracks within concrete (with a thickness greater than about 50 cm). Determining the current state of dam structures can be complex.
- Numerical models make it possible to satisfactorily predict future dam deformation trends. On the other hand, an evaluation of the development of the state of the stress is much more complex and often impossible.

Overcoming the current major limitations is unlikely in the short term. However, experience gained during the last decade has made it possible to make some recommendations in order to improve our approach to this problem in the future.

- It is unwise to ignore the problem. This attitude is less common nowadays but hasn't completely disappeared, and is usually explained by a certain fear of the unknown. Precautions in this study reflect the persistent reluctance to share current available data. A more active exchange of information would make it possible to better take advantage of the acquired experience by various actors in this field.
- Concrete expansion phenomena are progressing slowly in Europe, with very few exceptions. Dam structures with annual expansion rates greater than 25-30 μm/m are rare. Degradation of structures is therefore not imminent, except in special cases, but usually the process takes place gradually over several decades. There is thus enough time in order to study the specifics of each dam structure.
- Experience gained over the last decade, particularly with respect to the implementation of mitigation measures must be taken into account. There are many examples of measures that have not achieved expected results. Defining possible interventions can only arise from the combination of numerical analyzes together with practical knowledge. The risk of implementing unnecessary, or premature interventions should not be underestimated.

6.3 Need for further research & development

The above-mentioned points highlight progress that is absolutely necessary to better deal with the problems posed by concrete swelling in dams. In order to present dam operators with real progress and solutions, research and development should focus on the following main focus areas:

- Better knowledge of the chemical process and of the influence of surrounding conditions. The search for solutions which make it possible to inhibit this reaction must be pursued. However, only economically viable options are truly of interest.
- Technological advancements to better determine the state of the mass concrete (cracking, stress) at depth are essential for any reliable consideration of the conditions of a dam structure. Significant progress in this area would undoubtedly be an important benefit for reliable assessments of dam safety.
- Experience with the effectiveness of some mitigation interventions are currently very limited and ad hoc. A more systematic analysis of various approaches, of for example surface interventions, whose evolution would be monitored for at least ten years on different structures could help optimize future interventions.
- Finally, any progress made with numerical modeling of swelling phenomena are certainly welcome. Efforts must be directed towards reliable modeling of the state stress forces and not only of deformation. It is well known that an infinite number of stress distributions can give rise to the same deformation outcomes. Deformation modeling on its own, therefore, is by far not sufficient.

Finally, as has already been mentioned, the unrestrained pooling of the experience gained in various countries, and on different dam structures, would undoubtedly lead to a better approach to this problem.

7 Conclusions

Information collected by the Working Group (WG) makes it possible to estimate that between 35% and 45% of the 154 concrete dams in Switzerland show permanent deformation compatible with concrete expansion. This percentage is higher than the original expectations of the WG, and shows the scale of the phenomenon. Reversible behavior of a concrete dam is therefore not the rule. It should be noted that laboratory tests confirm the presence of concrete swelling (Alkali-Aggregate Reaction AAR or sulfatic ISA) in only about one-third of dams affected by non-reversible deformation.

It is however essential to clearly distinguish between different situations affecting a dam structure. Expansion rates are very variable, and the influence of non-reversible deformation on operating and safety conditions depend on a dam's specific geometry, which of course varies for each structure. Hence, we must learn to live with this phenomenon, but this does not necessarily mean that an affected dam has to be decommissioned in the short or medium term.

Once the presence of concrete expansion is confirmed, it is advisable to evaluate the consequences on the dam structure by clearly distinguishing between the ones affecting the operating aspects and those having an influence on dam safety. Subsequently, possible mitigation measures need to be identified. However, the examples of this study indicate that these interventions must be well thought-out in order to produce expected results on the dam structure. Except for demolition and some other very rare exceptions, all other measures only mitigate the effects of concrete swelling on dam behavior. These interventions therefore generally have a limited effect over time and are not a solution per se to the overall problem.

Experience from Switzerland and elsewhere has shown that residual concrete swelling potential evaluations are not of much practical use for operators, because the problem is always only linked to the dam structure, and not to the concrete itself, which is not the sole cause of the problem. The structure will deteriorate due to excessive deformation, well before degradation of the mechanical properties of the concrete. There are no dams in Switzerland, for the moment, with a slowing down swelling process.

Finally, as far as future progress of this process is concerned, it seems unlikely that the number of dams affected will increase significantly. However, deadlines for major interventions on some important dam structures are slowly approaching. Better knowledge of the process, especially with regard to the influence of surrounding conditions (i.e. exposure to sunshine, temperature, humidity), as well as consequences on the distribution of stress forces, is certainly desirable in order to be able to improve future interventions.

8 References

1. T. Stanton: Expansion of Concrete through Reaction between Cement and Aggregates, Proceedings of the American Society of Civil Engineers, 1940
2. T. Wälchli, E. Brühwiler: Tragverhalten von Kunstbauten mit Alkali-Aggregat-Reaktion (AAR) — Sachstandsbericht, Bundesamt für Strassen, Bericht 584, 2005
3. F. Hunkeler, C. Merz, P. Kronenberg: Alkali-Aggregat-Reaktion (AAR) — Grundlagen und Massnahmen bei neuen und bestehenden Kunstbauten, Bundesamt für Strassen, Bericht 8213, 2007
4. C. Merz, J.-G. Hammerschlag: Alkali-Aggregat-Reaktionen, Cementbulletin, 2000, Heft 5
5. C. Merz, A. Leemann: Validierung der AAR-Prüfungen für Neubau und Instandsetzung, Bundesamt für Strassen, Bericht 648, 2012
6. M. Ben Haha: Mechanical Effects of Alkali Silica Reaction in Concrete Studied by SEM-Image Analysis, École Polytechnique Fédérale de Lausanne, 2006
7. C. Dunant: Experimental and Modelling Study of the Alkali-Silica-Reaction in Concrete, École Polytechnique Fédérale de Lausanne, 2009
8. A. B. Giorla: Modelling of Alkali.Silica Reaction under Multi-Axial Load, École Polytechnique Fédérale de Lausanne, 2013
9. A. I. Cuba Ramos: Multi-Scale Modelling of the Alkali-Silica Reaction in Concrete, École Polytechnique Fédérale de Lausanne, 2017
10. A. Leemann, M. Griffa: Diagnosis of Alkali-Aggregate Reaction in Dams, State of the Art Report, EMPA, 2013
11. SIA-Merkblatt 2042: Vorbeugung von Schäden durch die Alkali-Aggregat-Reaktion (AAR) bei Betonbauten, Schweizerischer Ingenieur und Architektenverein, 2012
12. RILEM: Outline Guide to the Use of RILEM Methods in the Assessment of the Alkali-Reactivity Potential of Aggregates, Special Issue RILEM TC219-ACS, Materials & Structures, 2014
13. F. Amberg: Performance of Dams affected by Expanding Concrete, Dams in Switzerland, Source for Worldwide Swiss Dam Engineering, Baden-Dättwil, buag Grafisches Unternehmen AG, 2011, p. 309-314.
14. F. Amberg: A review of expanding concrete cases and consequences on dam performance, Proceedings of the Hydro International Conference, Bilbao, 2012
15. R.G. Charlwood, Z.V. Solymar: A review of alkali aggregate reactions in dams, Dam Engineering, Canada, Vol 5, Issue 2, Jul. 1994, p. 31-62.
16. D.G. Hayward, G.A. Thompson, R.G. Charlwood & R.R. Steele: Remedial measures at the Mactaquac generating station, 17th ICOLD Congress on large dams, Vienna; Q. 65 R. 47, 1991
17. R. Leroy, G. Micoulet, F. Tognola: Rehabilitation of Sera dam (Switzerland) affected by ASR, Dam maintenance and rehabilitation, Proceedings 2nd International Conference, Zaragoza, 2010

18. J. Sausse, J.P. Fabre: Diagnosis of Dams Affected by Swelling Reactions: Lesson Learned from 150 Monitored Concrete Dams in France, Dam Engineering, Vol XXIII, Issue 1, 2013, p. 5-17
19. J.-F. Seignol, L. Boldea, R. Leroy, B. Godart, J.G. Hammerschlag: Hydro-power structure affected by alkali-aggregate reaction: a case-study involving numerical re-assessment, Proceedings, First International Conference on Computational Technologies in Concrete Structures, Jeju, Korea, May 2009.
20. L. Boldea, J.-F. Seignol, R. Leroy, A. Dubas: Numerical re-assessment of the serviceability and safety of an hydraulic power-plant affected by AAR, Hydro 2011, Prague, Oct. 2011.
21. P. Droz, J.-F. Seignol, R. Leroy, L. Boldea: Strategy for the rehabilitation of Salanfe Dam, 14th International Conference on Alkali-Aggregate Reaction (ICAAR), Austin, Texas, Mai 2012
22. P. Droz, O. Vallotton, T. Menouillard, R. Leroy: Slot cutting an AAR-affected dam: case study of the Salanfe Dam, Hydropower & Dams, Issue 5, 2013
23. P. Droz, O. Vallotton, T. Menouillard, R. Leroy: Monitoring of an AAR affected dam, Q99 - R33, ICOLD, Stavanger, June 2015
24. R. Leroy: Salanfe Dam – Evolution of the monitoring system related to exploitation and ASR swelling needs, Bali, June 2014