Deterioration processes of dams affected by concrete swelling reactions. The Portuguese experience in monitoring and rehabilitation

António Lopes Batista (1)

(1) National Laboratory for Civil Engineering (LNEC), Lisbon, Portugal, albatista@lnec.pt

Abstract

In Portugal there are about six dozen large concrete dams with continuous observation, among which about a third are affected by concrete swelling reactions of internal origin. One of them, Alto Ceira dam, was replaced in 2013, since its rehabilitation was not feasible. Other three dams present average values of accumulated expansions greater than 1000x10^{-6} (Pracana, Santa Luzia and Fagilde dams). Pracana dam had major rehabilitation works in the 1980s. Currently, only Fagilde dam has annual expansion rates above 100x10^{-6}. The expansion process of Santa Luzia dam appears to be levelling out. Although in other affected dams, the deterioration processes are still of low to moderate magnitude, in some the expansion rates have been increasing in recent years.

A summary of the main observation activities (monitoring, inspection and tests) and the results of some of the studies carried out, within the scope of the behavior assessment of these dams, are presented.

***Keywords**: Concrete dams; Swelling reactions; Structural deterioration; Monitoring.*

1. INTRODUCTION

1.1 Portuguese dams affected by concrete swelling reactions

Most common chemical reactions of internal origin of concrete, which generate expansions, can be grouped into two large types, the alkali-silica reactions (ASR) and the internal sulphatic reactions (ISR). These two types of reactions need water to develop, so concrete dams are particularly vulnerable structures. In Portugal there are 20 dams where the effects of internal expansion of the concrete were identified, based on the results of monitoring and of visual inspections, namely: Alto Rabagão, Bemposta, Caniçada, Miranda, Penide and Picote dams, in the north; Aguieira, Bouçã, Cabril, Coimbra, Covão do Meio, Fagilde, Fratel, Penha Garcia, Pracana, Raiva and Santa Luzia dams, in the center-north; and Caia and Monte Novo dams, in the center-south. Alto Ceira dam is also included, as it is the first and only Portuguese dam replaced and demolished, in 2013, due to the enormous structural degradation caused by ASR. Figure 1.1 shows a map of mainland Portugal with the dominant lithologies and the geographic location of these dams, also indicating the respective structural type and current owner.

The affected dams are located in the north and south interior, in areas of predominance of granitic and schistose rocks, and in the central zone, where the Schist-Greywacke Complex (SGC) occurs. Ordovician rocks, comprising thick layers of massive quartzite, discordantly overlie the metasedimentary rocks of the SGC and form discontinuous elongated folds oriented NW-SE, which form steep valleys. Granitic or quartzite coarse aggregates were then used in the concrete compositions of all these dams, except in the case of Fagilde dam, where micritic limestones were used.

The 20 large concrete dams affected by expansive reactions (shaded in Table 1.1) represent 34% of the 58 concrete dams with continued observation.

The affected dams were all completed between 1942 and 1984, corresponding to 57% of the 35 dams built in this period. There are no signs of swelling processes in the dams built in the last 35 years, probably due to both the change in composition of concrete mixes and improvements in the construction processes.

Four dams present average values of accumulated expansions greater than 1000x10^{-6}, namely: Alto Ceira (replaced in 2013), Pracana, Santa Luzia and Fagilde dams. Fagilde dam, built with ready-mixed concrete, is the only case in which the ISR is largely predominant. Alto Ceira, Pracana and Santa Luzia dams, all located in the Schist-Greywacke Complex, were built with strained quartzite aggregates, so they experienced severe deterioration due to the ASR. For these four cases, the relevant aspects of the behaviour observed, that led to the detection of the expansive phenomena, are presented in Section 2. Some aspects of the studies carried out to characterize the expansive reactions and their structural effects are also addressed.

Structural type	Dams affected by swelling reactions	Current owner
Gravity	Coimbra	APA
	Fagilde	APA
	Fratel	EDP
	Monte Novo	APA
	Penha Garcia	AdP
	Penide (1)	Aguia Enlica
	Raiva	EDP
Arch and arch-gravity	Alto Ceira (2)	EDP
	Alto Rabagão	EDP
	Bemposta	Engie
	Cabril	EDP
	Bouçã	EDP
	Caniçada	EDP
	Covão do Meio	EDP
	Picote	Engie
	Santa Luzia	EDP
Buttress and multiple arch	Aguieira	EDP
	Caia	Ass. Ben. Caia
	Miranda	Engie
	Pracana (3)	EDP

(1) Cyclopean concrete dam covered with granite stone masonry
(2) Replaced and demolished in 2013
(3) Rehabilitated between 1988 and 1992

Figure 1.1: Location of the Portuguese dams affected by concrete swelling reactions.

For the remaining 16 dams, the expansions still have low to moderate magnitudes (accumulated expansions not greater than 500×10^{-6}), so only the most significant results of the monitoring are presented in Section 3.

This paper considers some elements of the research program recently concluded at LNEC concerning the deterioration and rehabilitation of dams affected by swelling reactions of internal origin in concrete [1] and updates, in many aspects, the 2012 inventory of Portuguese dams subject to this pathology [2].

Table 1.1: List of large Portuguese concrete dams with continuous monitoring, the rows in grey indicate those affected by swelling reactions

Dam	Structural type	Year of completion	Height (m)
Santa Luzia	Arch	1942	76
Alto Ceira	Arch	1949	36
Penide	Gravity	1949	18
Castelo do Bode	Arch-gravity	1951	115
Pracana	Buttress	1951	60
Venda Nova	Arch-gravity	1951	97
Belver	Gravity	1952	21
Covão do Meio	Arch	1953	28
Salamonde	Arch	1953	75
Cabril	Arch	1954	132
Bouçã	Arch	1955	65
Caniçada	Arch	1955	76
Bravura	Arch	1958	41
Picote	Arch	1958	99
Miranda	Buttress	1961	80
Alto Cávado	Gravity	1964	29
Alto Rabagão	Arch	1964	94
Bemposta	Arch-gravity	1964	87
Caia	Buttress	1967	52
Roxo	Buttress	1968	49
Carrapatelo	Gravity	1972	57
Odivelas	Multiple arch	1972	55
Vilarinho Furnas	Arch	1972	94
Fratel	Gravity	1973	43
Régua	Gravity	1973	42
Valeira	Gravity	1975	48
Varosa	Arch	1976	76
Penha Garcia	Gravity	1980	25
Coimbra	Gravity	1981	40
Raiva	Gravity	1981	36
Aguieira	Multiple arch	1981	89
Monte Novo	Gravity	1982	30
Pocinho	Gravity	1982	49
Fagilde	Gravity	1984	27
Fronhas	Arch	1984	62
Crestuma	Gravity	1985	65
Ranhados	Gravity	1986	41
Torrão	Gravity	1988	70
Corgas	Gravity	1991	30
Funcho	Arch	1991	49
Alto Lindoso	Arch	1992	110
Caldeirão	Arch	1993	39
Touvedo	Gravity	1993	43
Sordo	Gravity	1997	36
Catapereiro	Arch-gravity	1999	48
Alqueva	Arch	2002	96
Bouçoais-Sonim	Gravity	2004	43
Rebordelo	Gravity	2004	36
Pedrógão	Gravity	2005	43
Ferradosa	Gravity	2008	34
Olgas	Gravity	2009	35
Pretarouca	Gravity	2009	29
Alto Ceira II	Arch	2013	41
Baixo Sabor	Arch	2014	123
Feiticeiro	Gravity	2014	45
Ribeiradio	Gravity	2015	83
Ermida	Gravity	2015	35
Foz Tua	Arch	2016	108

1.2 Structural effects of concrete swelling in dams

Macroscopic evidence of the effects of concrete expansive reactions in dams, which are generally similar for the ASR and the ISR, are as follows:

i) surface cracking, diffuse and generalized (map), due to the local heterogeneity of the expansion, sometimes with concrete spalling and formation of small craters, and linear cracking due to the global structural response to prescribed strains;

ii) in structural contraction joints and in decks above the spillways, generalized closing and slippage between blocks (in particular the misalignments at the crest), with crushing of infills and spalling of edges, due to absolute and relative displacements;

iii) in spans and power plants, namely in ducts, orifices and dischargers, ovalization of the ducts section, misalignment of parts and respective fixings, and closing and distortion of holes and shutters, causing problems in supports and in equipment clearances and jamming the gates;

iv) discoloration of concrete surfaces affected by the ASR;

v) efflorescence and exudation of gel (ASR) on the surfaces, generally in cavities, crevices and joints; and

vi) leakage of water through cracks and rupture of reinforcements for expansive reactions of greater magnitude.

The effects of the expansive reactions are reflected in the results of the monitoring of concrete dams, specifically in the evolution of the following parameters:

i) absolute displacements, i.e. vertical displacement upwards and radial displacement towards upstream (except in buttress dams);

ii) relative displacements between blocks, i.e. closing of contraction joints, elevation of higher blocks in relation to the lower ones, and horizontal displacement towards upstream of the central (higher) blocks in relation to the lateral (lower) ones;

iii) concrete deformations, in active and in no-stress strain meters; and

iv) stress field.

The diagnosis and prognosis of concrete swelling reactions and its structural effects are based, in general, by: i) the interpretation of the observation results; ii) laboratory, physical and chemical analyses and tests on concrete samples extracted from the dam's body; iii) "in situ" tests; and iv) mathematical modelling.

1.3 Interventions in dams affected by concrete swelling

The effects of the ASR and the ISR in concrete structures are difficult to deal with, since there are no fully effective methods either to mitigate the development of the expansive reactions, or to repair deteriorated structures. However, engineering has developed a set of measures that, when used judiciously, allow to limit, in many cases, the unfavourable effects of expansive reactions, and adequately manage the functionality, safety and durability of the structures.

The measures to mitigate the concrete swelling reactions mostly consist in limiting the development of these chemical reactions by reducing the involved ingredients. Techniques for repairing and reinforcing deteriorated structures, which often include structural modifications, are often associated with mitigation measures, so they are addressed in an integrated manner.

The physical limitation of the ingredients of the reactions can be accomplished by: i) preventing the access of water, in its liquid and vapor forms; and ii) allowing to change, with advantage, the characteristics of the alkaline solution (only to control the ASR). When it is not feasible to prevent access to water, drainage can be done, although this measure is less effective. Regarding other techniques that can mitigate expansive reactions, the following ones should be mentioned: i) controlling the temperature of the setting and hardening of concrete; and ii) increasing the confinement of the concrete, through appropriate construction of elements and/or imposed compression. These types of measures, often used together, usually produce very beneficial effects.

Preventing the access of water to the concrete can be done by using coatings and surface protections, waterproofing membranes and/or protection panels. The drainage of concrete generally consists in creating specific holes. Changing the characteristics of the alkaline solution, to mitigate the ASR, can be done by injecting chemical compounds, even though this is currently only feasible (due to costs) in small structural parts. Controlling the temperature of the concrete, although not common in the operation stage of dams, can be achieved using specific coatings and panels. Increasing the confinement of concrete can be accomplished, as a rule, using concrete, steel, or synthetic fiber elements (steel and synthetic fiber elements can be passive or prestressed).

Regarding the reparation of deteriorated structures, mass regeneration can be undertaken through injection and replacement of parts or volumes considered irrecoverable. The demolition and total reconstruction, in extreme situations of deterioration, often involves, in the case of concrete dams, new structures with different and optimized structural solutions.

Structural reinforcement, when necessary, generally comprises the addition of linear or laminar elements of steel and carbon fibers, passive or active, as well as concrete lining, which can be simple, reinforced and/or pre-stressed. In all these cases, the confinement is also increased, resulting in the mechanical limitation of the expansion.

The structural changes are intended to endow the structures with the capacity to minimize the mechanical effects of the expansion. The most common interventions are the slot cutting of the concrete, which allows to release the stresses and to reverse the deformations caused by expansion, and the modification of supports and connections of equipment (generator units, gates, etc.)

2. PORTUGUESE DAMS WITH THE GREATEST MAGNITUDE OF SWELLING

2.1 Alto Ceira dam

Alto Ceira dam, built in 1949 and deactivated in 2013 (through demolition of its central section), was a slender arch with a height of 36 m (Figure 2.1). The dam was founded on a schistose rock mass. In its construction, concrete produced with quartzite aggregates was used. Since the 1960s, the behavior observed of the dam has been characterized by progressive displacements, vertically upwards and radially towards upstream. The concrete cracking became more accentuated over time, reaching a very advanced stage at the beginning of this century, with leakage through the cracks mainly located in the bearing zones, at mid-height (Figure 2.2).

Figure 2.1: Alto Ceira dam. Downstream view and central cantilever cross-section

The first laboratory, physical and chemical tests, carried out on concrete samples extracted from the dam's body in 1986 confirmed the seriousness of the deterioration of the concrete and its high remaining potential for expansion [3]. In 1990, a new campaign of sampling and testing of the dam's concrete was carried out. The results of the accelerated expansion tests, carried out by immersing the concrete specimens in a saturated solution of sodium chloride and in distilled water, in an oven at 50ºC, are shown in Figure 2.3 [4]. In specimens produced from samples extracted from the left bank, maximum remaining expansions of about 1400×10^{-6} were then obtained.

Figure 2.2: Alto Ceira dam. Diffuse cracking at the crest (on left) and weirs on the downstream face to collect water passing through cracks (on right)

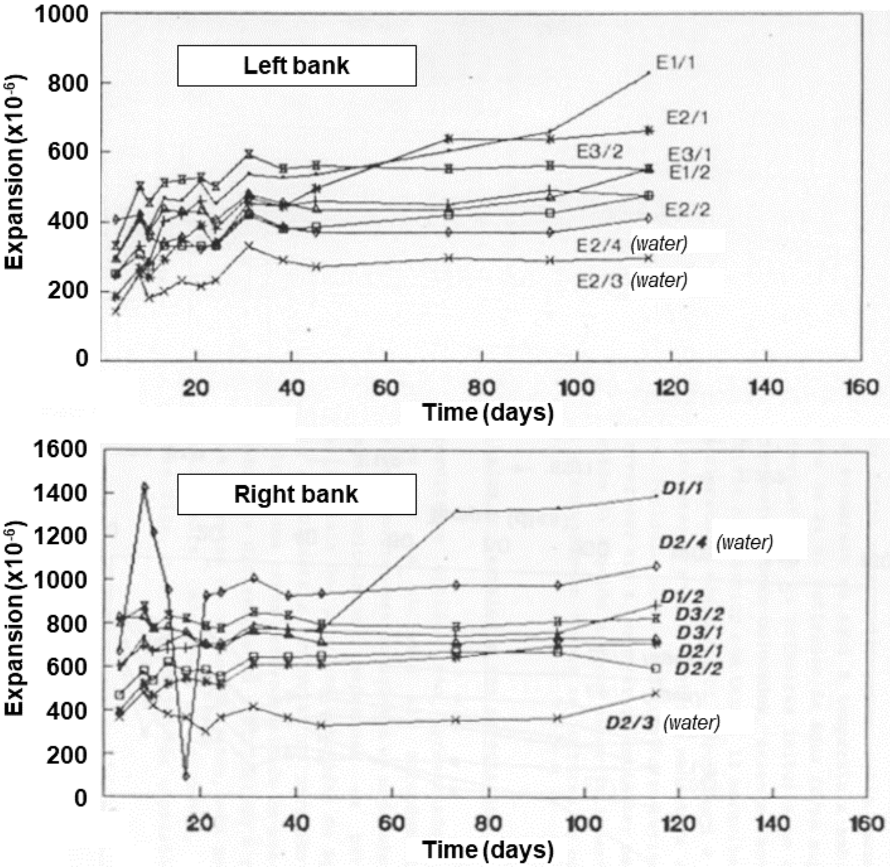
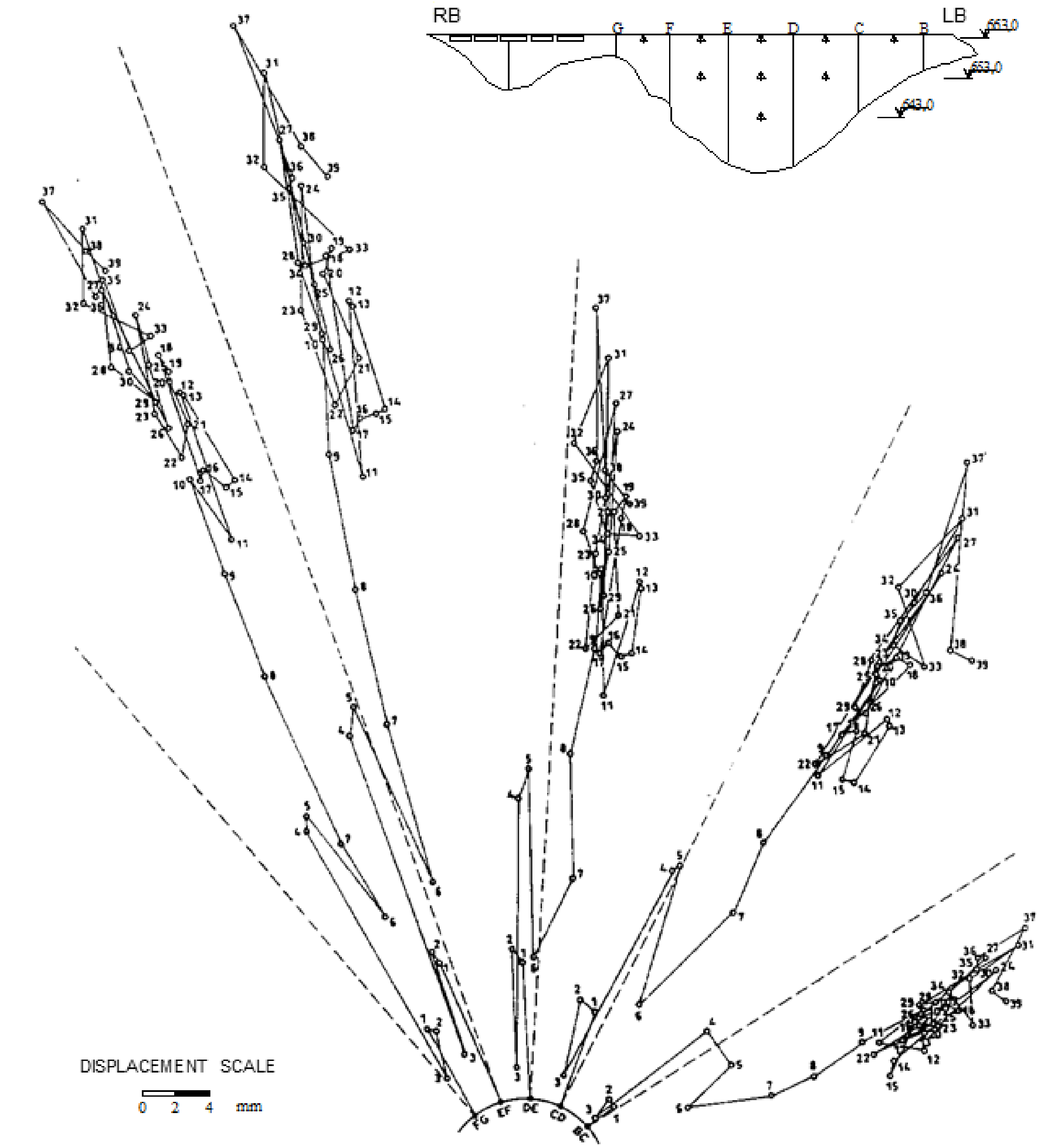


Figure 2.3: Alto Ceira dam. Main results of the accelerated expansion tests of concrete specimens (extracted in 1990) immersed in a saturated solution of sodium chloride and in distilled water, in an oven at 50ºC (adapted from [4])

The absolute displacements of the dam were monitored by geodesic methods since the first filling of the reservoir, in 1950. The radial displacements showed, over time, a great progressivity towards upstream. Figure 2.4 shows the evolution of horizontal displacements observed by geodesic methods in the upper targets of the arch (elevation 663.00 m), between 1950 and 1993. Figure 2.5 presents the evolution of these displacements at the point where they had the greatest values (next to the crest of EF block), including a function adjusted to the values observed between 1984 and 2011. The irreversible value of the radial displacement at this point reached, in this period, about 50 mm.



Geodetic survey	Date	Water level in the reservoir (m)	Geodetic survey	Date	Water level in the reservoir (m)	Geodetic survey	Date	Water level in the reservoir (m)
1	15-09-1950	659.0	14	15-01-1986	659.0	27	8-09-1988	658.4
2	24-09-1950	630.0	15	16-01-1986	659.0	28	12-10-1988	658.4
3	29-11-1950	658.6	16	24-03-1986	655.6	29	10-01-1989	659.2
4	22-08-1963	657.5	17	25-03-1986	658.8	30	4-05-1989	658.7
5	29-08-1963	630.0	18	7-10-1986	658.2	31	22-08-1989	658.0
6	20-11-1963	659.0	19	22-10-1986	630.0	32	20-09-1989	658.3
7	18-05-1970	656.1	20	23-10-1986	630.0	33	19-02-1990	658.7
8	26-09-1974	654.9	21	3-12-1986	658.3	34	15-03-1990	658.7
9	28-09-1978	654.9	22	7-01-1987	658.6	35	18-10-1990	659.4
10	10-08-1983	655.2	23	11-06-1987	658.5	36	10-04-1991	658.7
11	17-05-1984	658.2	24	11-08-1987	655.0	37	25-07-1991	658.5
12	24-01-1985	658.0	25	8-10-1987	658.6	38	19-10-1992	658.4
13	25-10-1985	654.8	26	8-03-1988	658.7	39	27-04-1993	658.9

Figure 2.4: Alto Ceira dam. Evolution of the horizontal displacements monitored by geodesic methods in the upper targets of the arch (elevation 663.00 m), between 1950 and 1993 (adapted from [5])

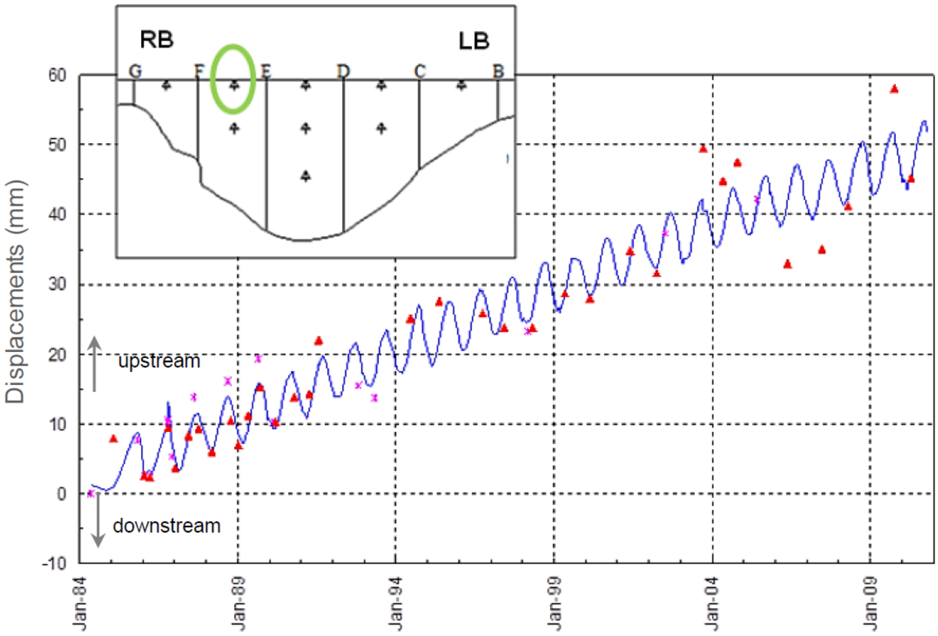


Figure 2.5: Alto Ceira dam. Function adjusted to the radial displacements monitored by geodetic methods in the upper target of the EF block, located near the crest, from 1984 to 2011

Figure 2.6 shows the evolution of the vertical displacements at the crest, obtained by precision geometric levelling, between 1950 and 2012. A tendency to blistering has been observed since the early years of operation, but rates have increased substantially since 1983. In the central zone (block DE), the accumulated vertical displacement was about 20 mm (average vertical expansion, including concrete and cracks, of about $500x10^{-6}$), but in block FG, on the right bank, the displacement reached about 70 mm, which corresponds to an average vertical expansion (concrete and cracks) of about $4000x10^{-6}$.

Regarding the leakage on the downstream face, Figure 2.7 shows the evolution of the discharges in the lower section of the BC block (collected at the CE1 weir, see Figure 2.2, on the right side), from 1997 to 2013. In the last years of operation, the discharges reached maximum values of about 5 l/min, with seasonal average values between 2 l/min and 4 l/min. The passage of water through cracks enhances the degradation of the concrete.

Figure 2.8 shows the main cracks on the downstream face, identified during the surveys carried out in 1994 and 2001. The main cracks had a horizontal direction on the face, being parallel to the insertion surface in the bearing zones at mid-height. Block BC, on the left bank, and blocks FG and GH, on the right bank, were the most cracked ones.

In 2005, a mapping of occurrences was carried out on the downstream face, using georeferenced digital photography (Figure 2.9) [7, 8]. It can be noted that the main cracks were clearly identified, as well as leakages and carbonate deposits.

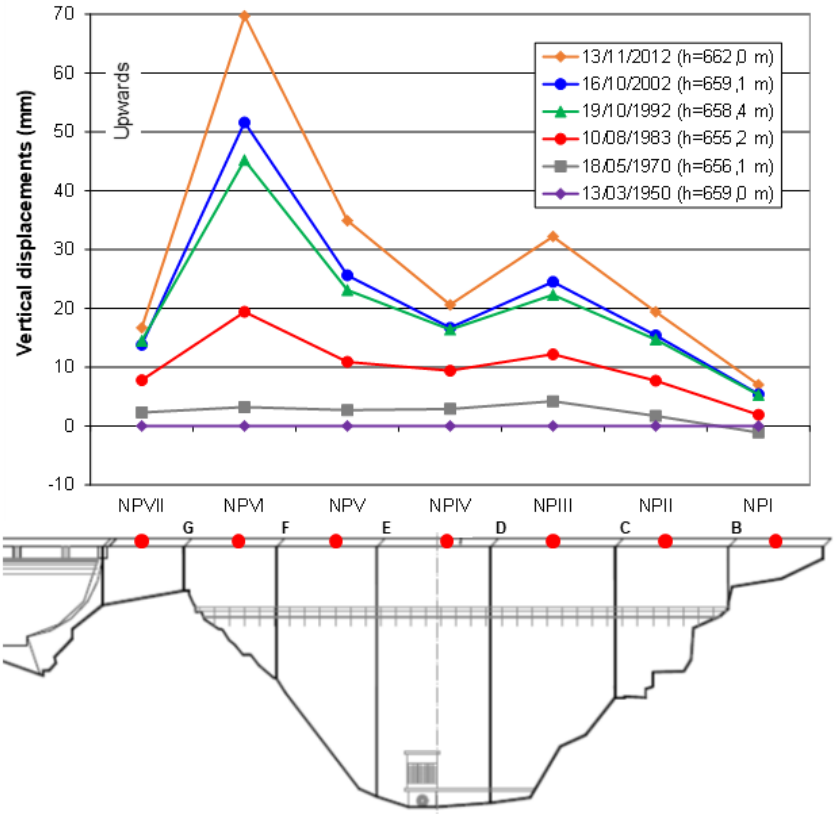


Figure 2.6: Alto Ceira dam. Vertical displacements monitored by precision geometric levelling, between 1950 and 2012

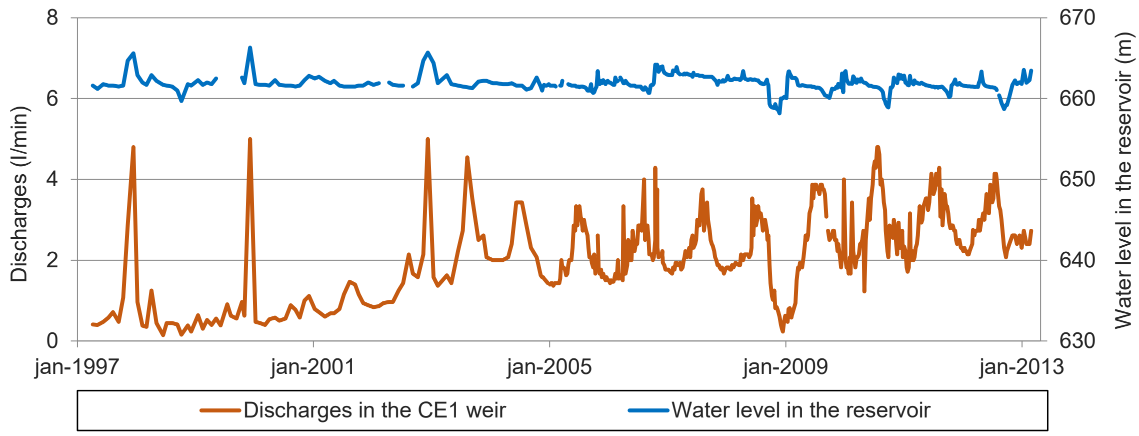


Figure 2.7: Alto Ceira dam. Discharges through cracks at the bottom of BC block, collected in the CE1 weir, between 1997 and 2013

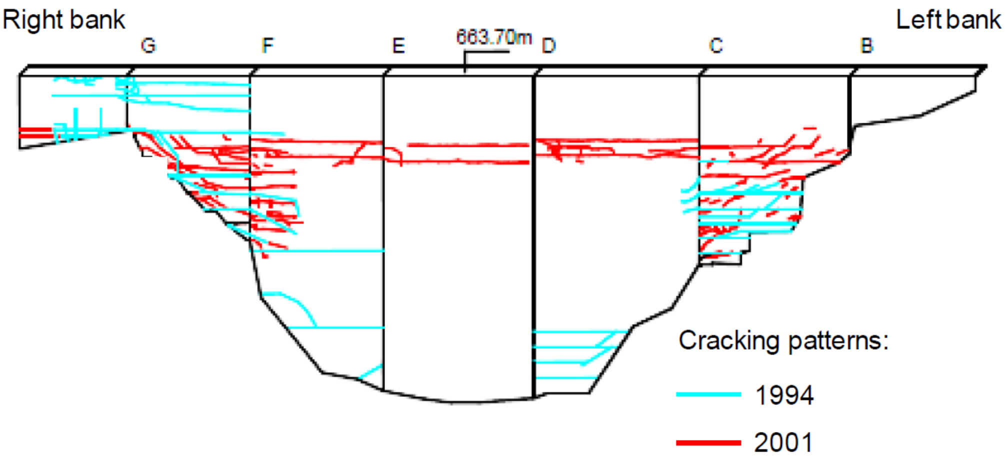


Figure 2.8: Alto Ceira dam. Main cracks on the downstream face, identified during the surveys carried out in 1994 and 2001, using traditional techniques [9]

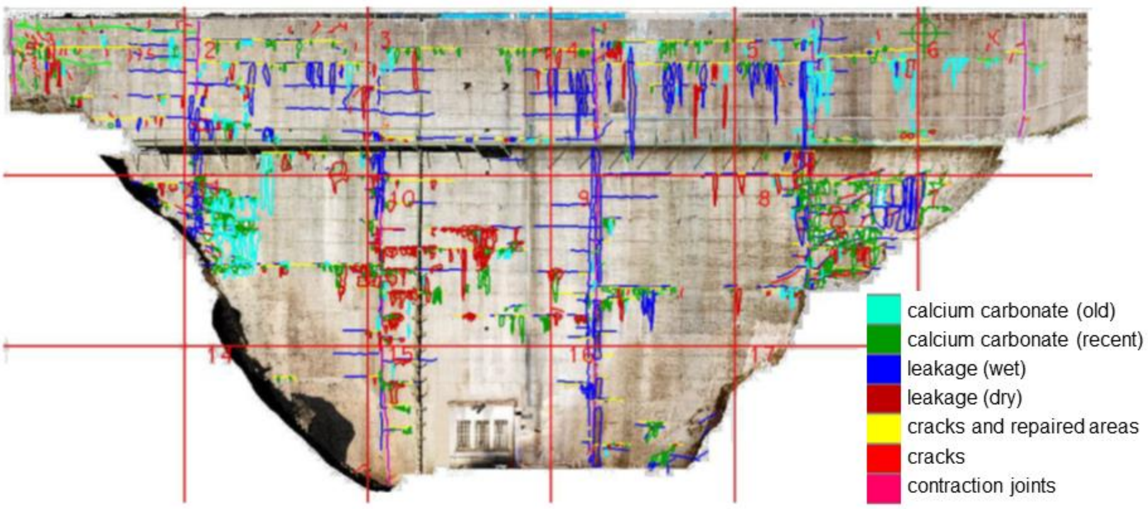


Figure 2.9: Alto Ceira dam. Results of occurrence survey of the downstream face, carried out in 2005, using georeferenced digital photography [7,8]

The behavior modelling and structural safety assessment studies of the dam, performed in 1994 [5], based on the finite element method, were supported by a mesh of three-dimensional finite elements of the dam and its mass rock foundation. In 2004, these studies were updated [9] confirming the magnitude of the evolution of the structural degradation predicted about a decade earlier. Given the depth of the main cracks and the progressive evolution of the water leakages in the bearing zones at mid-height, it was recommended to the dam's owner to carry out studies aiming the dam's rehabilitation or even its replacement. Subsequent studies by the owner concluded that the location and the characteristics of the dam, associated with the great magnitude of displacements and opening of cracks, made its rehabilitation impracticable [10]. The owner then proceeded with the design of the new dam, also of the arch type, which was built about 300 m downstream (Figure 2.10), between 2011 and 2013 [11]. The old dam was deactivated through the demolition of the central section of the structure. This is the only case of replacement of a concrete dam, in Portugal, due to structural deterioration.

Figure 2.10: Alto Ceira dam. View of the new dam (called Alto Ceira II dam) and of the old dam (partially demolished), after the first filling of the reservoir, in July 2014

2.2 Santa Luzia dam

Santa Luzia is a 76 m high cylindrical arch dam, located in a natural gorge excavated by the Unhais stream in a quartzite outcrop. It consists of a main arch and an arch-gravity structure that closes the upper section of the left bank (Figure 2.11). The dam was completed in 1943. It is the first arch dam built in Portugal. The dam was designed by André Coyne. It was also the first dam studied at LNEC using physical structural models, years later.

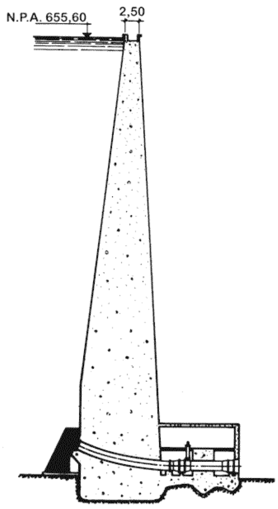


Figure 2.11: Santa Luzia dam. General view and central cantilever cross-section

The information about the composition of the concrete used in the dam's construction is scarce. Nonetheless, coarse quartzite aggregates, crushed on site, and normal Portland cement with an average dosage of 240 kg/m^3 of concrete, are known to have been used [12].

Blistering at the dam crest was detected for the first time in 1966 [13]. Radial towards upstream and upward vertical displacements had increased at constant rates between 1960 and 2000. However, in recent decades, these rates been decreasing, indicating that the reaction is exhausting. Figure 2.12 shows the irreversible vertical displacements computed for the levelling marks at the crest in the periods 1945-1970, 1970-1996 and 1996-2019, obtained by means of a quantitative analysis. It is noted that the vertical strains, corresponding to the accumulated displacements, are greater in the arch-gravity than in the arch, associated with the greater structural deterioration of that part.

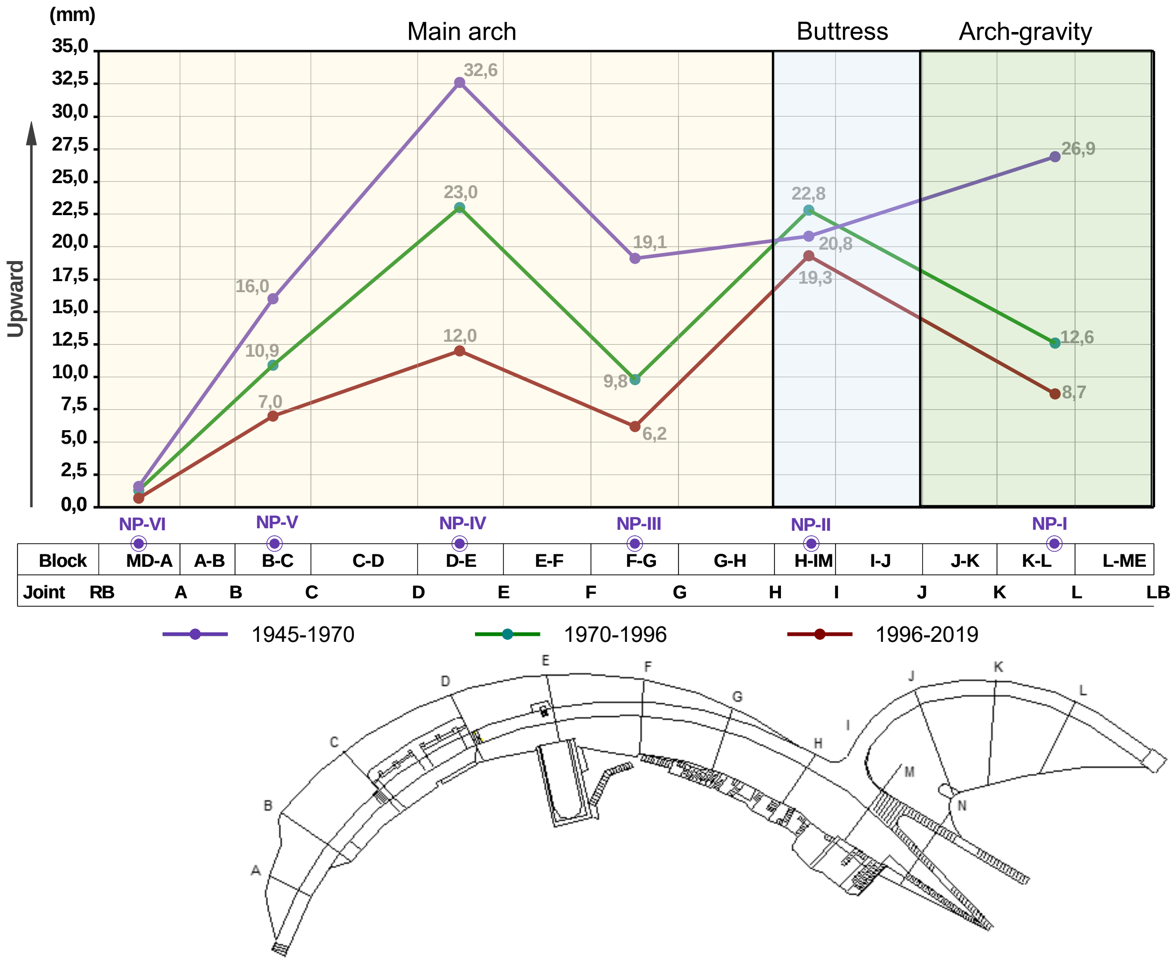


Figure 2.12: Santa Luzia dam. Irreversible vertical displacements computed for the levelling marks at the crest in the periods 1945-1970, 1970-1996 and 1996-2019 [21]

LNEC carried out surveys on the cracking state of the dam in 1979/1980 [14] and in 1998 [15]. Since then, these surveys have been carried out by the dam's owner. Cracking evolved up to the present, being much more intense in the arch-gravity and along the connection with the arch section. The cracking is predominantly diffuse, but there are cracks of linear development with openings of several centimetres at the connection zone and at the entire crest, and linear cracks with smaller opening on the faces of the arch and of the arch-gravity. Figures 2.13 and 2.14 show photographs, taken in 2019, of some areas of the dam where there is greater cracking in the concrete.

Figure 2.13: Santa Luzia dam. Cracks in the main arch, specifically along concreting joints on the downstream face, with deposits of calcium carbonate (on left), and linear cracks at the crest, on the right bank side (on right), in 2019

Figure 2.14: Santa Luzia dam. Large cracks at the crest, along the connection zone of the main arch and the arch-gravity (on left) and diffuse cracking on the upper section of the arch-gravity upstream face (on right), in 2019

The initial monitoring system of the dam allowed the determination of: i) horizontal displacements at 9 points of the downstream face, by geodesic methods; ii) vertical displacements at 6 points of the crest, by precision geometric levelling; iii) temperatures in the dam body, with electrical resistance thermometers; and iv) strains at several points along the downstream face, with vibrating wire strain meters. In 1945, 4 more geodetic targets were installed on the downstream face. In 1948, it was found that the equipment for measuring temperatures and strains was broken. In 1959, deformeter bases were installed in contraction joints. After identifying the anomalous behavior of the dam, the monitoring system was improved in two stages. In 1987, rod extensometers were installed on the foundation, downstream, and on the dam body, between the foundation and the crest, to measure vertical displacements, and the subsystem for measuring relative displacements between blocks was updated with 32 more deformeter bases. In 1994, using the walkway installed downstream, at mid-height of the arch, the following devices were installed: i) two plumb lines, outside the central block DE, and an optical plummet, also outside the BC block, on the right bank side; ii) 14 thermometers in the dam's body, 12 in the main arch and 2 in the arch-gravity; and iii) new deformeter bases, replacing others that had exceeded the measurement field, to measure the opening of some cracks [16].

The main studies to characterize the properties and expansions of the concrete were carried out in the 1990s [17, 18, 19]. The core samples extracted in 1995 allowed the identification of the ASR products, but also, to a lesser extent, expansive products such as ettringite and thaumasite, which result from the ISR. The expansion tests showed that the remaining expansion potential was low, on the order of $100x10^{-6}$. The compressive strength of concrete had an average value of 30.6 MPa, with a maximum of 40.0 MPa and a minimum of 19.2 MPa. Regarding the direct tensile strength, the following results were obtained: i) for the 40 specimens not submitted to expansion tests, the average value was 1.35 MPa; and ii) for the 55 specimens previously submitted to expansion tests, the average value was 1.11 MPa. Regarding the modulus of elasticity, an average value of 20.9 GPa was obtained. Creep tests, carried out on 6 specimens, showed considerable deformability of the concrete. From this set of results, it was concluded that the concrete presented significant internal damage, given the low values obtained for the tensile strength and for the modulus of elasticity, and creep rates not compatible with a healthy concrete of about 50 years old.

From 2003 to 2007, studies were carried out to analyse and interpret the dam structural behavior observed until 2003 [20] and until 2006 [12].

The monitoring results in recent years show that expansion rates are decreasing [21], so the dam's owner intends to update the diagnostic and prognostic studies of the concrete and of the dam itself, in order to define the most appropriate interventions to be performed.

2.3 Fagilde dam

Fagilde dam, located on the river Dão, was built from August 1982 to the end of 1983. It is a concrete structure consisting of a central part with buttresses, flanked by two cylindrical arches that close the banks. The dam rests on a granitic rock mass foundation. The central structure has a maximum height of 26.6 m and comprises three buttresses, interleaved with two spillways (one near the crest and the other at mid-height), each with two spans. The two cylindrical arches have a variable thickness, vertical upstream face and a maximum height of 18.0 m, being supported by lateral buttresses and artificial abutments (Figure 2.15). The first filling of the reservoir took place between June 1985 and January 1987.

Figure 2.15: Fagilde dam. Downstream view

Ready-mixed concrete (class B225) was used in the construction of the dam, with Portland cement dosage of 360 kg/m^3, coarse aggregate mainly consisting of crushed limestone rock, containing rare silica, and fine aggregates formed by siliceous gravel and sand (Table 2.1).

Table 2.1: Fagilde dam. Concrete composition

Components (size in mm)		Content (%)	Content (kg.m^{-3})
Coarse aggregate	38.1 – 25.4	35	661
	25.4 – 9.5	15	283
	9.5 – 4.8	12	227
Sand	Coarse	27	510
	Fine	11	208
Cement (Portland)		-	360
Water		-	160
Water-cement ratio		0.44	-

The characteristic values of the compressive strength, obtained from tests carried out during the construction to control the quality of the concrete, using cubic specimens 20 cm side, were 26.7 MPa, 31.8 MPa and 33.8 MPa at 7, 14 and 28 days of age, respectively [22]. It can therefore be considered that the maturation was rapid and that at 28 days of age a high value was obtained in relation to the prescribed resistance grade (B225). Considering the cement dosage used and the thickness of the concreted elements, it is likely that the concrete temperatures have reached high values, providing conditions for the development of the ISR [23].

In LNEC's annual inspection of January 2001, for the first time a significant number of cracks on the dam's faces was recorded. The occurrence of progressive horizontal displacements upstream and upward vertical displacements of the crest also indicated the existence of a deterioration process of the concrete [22].

Due to the progressive deterioration of the dam, its monitoring became even more careful and the following studies were carried out successively: i) characterization of the concrete swelling process, according to a previously defined plan [24], comprising a survey on the cracking state of the faces [25], the study of the creep of concrete [26] and the petrographic, physical and chemical characterization of concrete [23, 27]; ii) complementary instrumentation of the dam [28]; iii) detailed analysis of the results of the continuous monitoring [29, 30]; and iv) analysis and interpretation of the observed behavior using mathematical modelling [31, 32].

In 2004, a crack mapping was carried out, to establishing a reference state. The main existing cracks on the upstream and downstream faces, buttress faces and crest were characterized, with the register of their numbering, location, opening, development and average orientation. Figure 2.16 shows the cracking pattern, complemented with new cracks detected in the update survey carried out in 2009. The cracking on the faces was considerable, with most cracks opening of less than 1 mm. However, a reduced number of cracks (the most expressive ones) had a larger opening, although always less than 10 mm. Figures 2.17 and 2.18 shows photographs, obtained in 2015 and 2016, respectively, of the concrete faces cracking of the arches and of the central buttress.

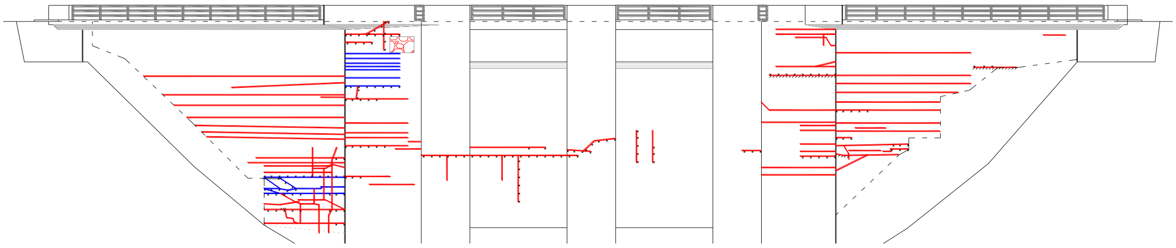


Figure 2.16: Fagilde dam. Cracking pattern of the downstream face in 2009 [29]

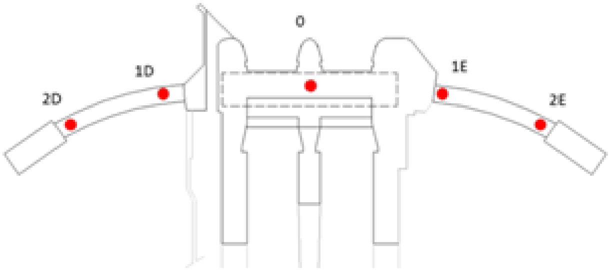
Figure 2.17: Fagilde dam. Downstream views of the bottom zone of the right arch (on left) and of the upper zone of the left arch (on right), in 2015

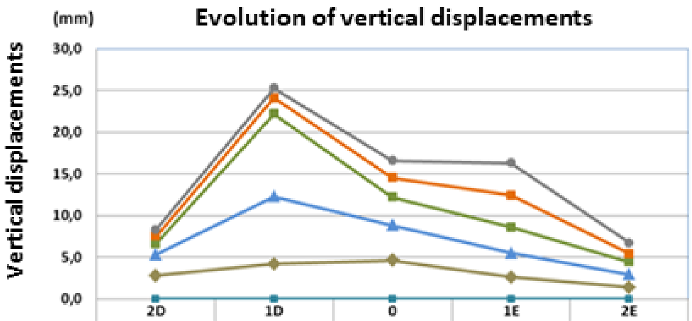
Figure 2.18: Fagilde dam. General view (on left) and detailed view (on right) of the central buttress, in 2016

In 2008, an evaluation of the mechanical properties of the structural concrete was carried out on specimens prepared from 4 short 150 mm diameter core samples, extracted from the arches and lateral buttresses. The compressive and tensile strengths were determined, as well as the modulus of elasticity and the creep function. The test results showed a high compressive strength, with an average value of 33.7 MPa and a minimum of 21.1 MPa. Comparing these results with those obtained from the tests carried out during the construction, a small depreciation of the compressive strength due to the deterioration process was noticed. Regarding the tensile strength by diametral compression, an average value of 3.0 MPa with a minimum of 2.25 MPa were obtained, which presupposes a small loss of tensile strength. For the modulus of elasticity, values between 19.1 GPa and 24.5 GPa were obtained, which can be considered low for this type of concrete, probably due to the micro-cracking caused by the swelling reactions.

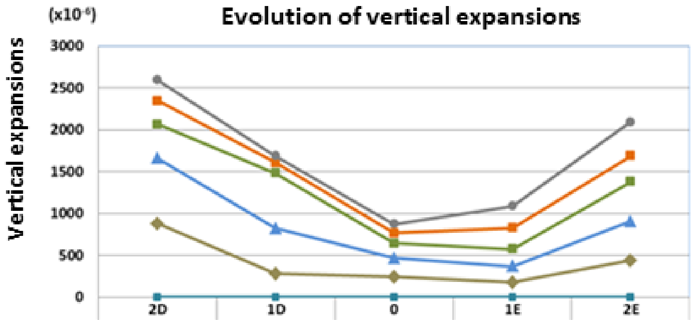
The dam monitoring system allows the evaluation of the main actions and the structural and hydraulic responses. The absolute displacements (horizontal and vertical) are monitored by geodesic methods and the joint movements through deformeter bases. In 2008, the updating of the monitoring system was proposed, with the goal of improving the characterization of the expansive reaction and its effects [28]. Some of the work has already been done, namely the holes for installing vertical rod extensometers, in 2008 (due to various difficulties, they have not yet been installed), and the improvement of the geodetic targets, in July 2016.

Figure 2.19 summarizes the results of the quantitative analysis of the vertical displacements monitored between 1985 and 2015, as well as the corresponding vertical strains [30]. Between 1985 and 2015 there were very high accumulated strains, on the order of 2140×10^{-6} in the right arch, 1590×10^{-6} in the left arch and 870×10^{-6} in the central buttress.





	2D	1D	0	1E	2E
July 1985	0,0	0,0	0,0	0,0	0,0
April 1995	2,8	4,2	4,6	2,6	1,4
September 2000	5,3	12,3	8,8	5,5	2,9
November 2005	6,6	22,2	12,2	8,6	4,4
June 2010	7,5	24,1	14,5	12,4	5,4
October 2015	8,3	25,3	16,6	16,3	6,7



	2D	1D	0	1E	2E
July 1985	0	0	0	0	0
April 1995	875	280	242	173	438
September 2000	1656	820	463	367	906
November 2005	2063	1480	642	573	1375
June 2010	2344	1607	763	827	1688
October 2015	2594	1687	874	1087	2094

Figure 2.19: Fagilde dam. Results of the quantitative analysis of the vertical displacements observed at the crest by precision geometric levelling, until 2015, and estimation of the corresponding vertical expansions [30]

Given the concrete degradation, three alternative interventions have been pointed out, namely: i) waterproofing the upstream face and cement grouting of cracks in the arches; ii) the replacement of the two arches; and iii) the replacement of the dam itself. The first two alternatives would require detailed studies to be carried out on the dam’s behavior in its new structural condition. However, due to the dam's limited capacity to supply water in the dry season, it is considered advantageous to replace the dam with a new one, located in a suitable section of the river a few hundred meters downstream.

2.4 Pracana dam

Pracana dam is a structure composed by 12 buttresses (P1 to P12), limited by contraction joints spaced 13.00 m apart, flanked on both sides by gravity structures, with 51.1 m on the right bank (blocks B0 to B2) and 38.4 m on the left bank (blocks B12 to B15) (Figure 2.20). The total length of the crest is 245.5 m. It is 60 m high and is founded on a rock mass composed of phyllites and greywackes. It was completed in 1951 and rehabilitated between 1988 and 1992. Quartzite and granite aggregates were used in the concrete mix. The original dam was equipped with a flood spillway in a well, on the right bank side.

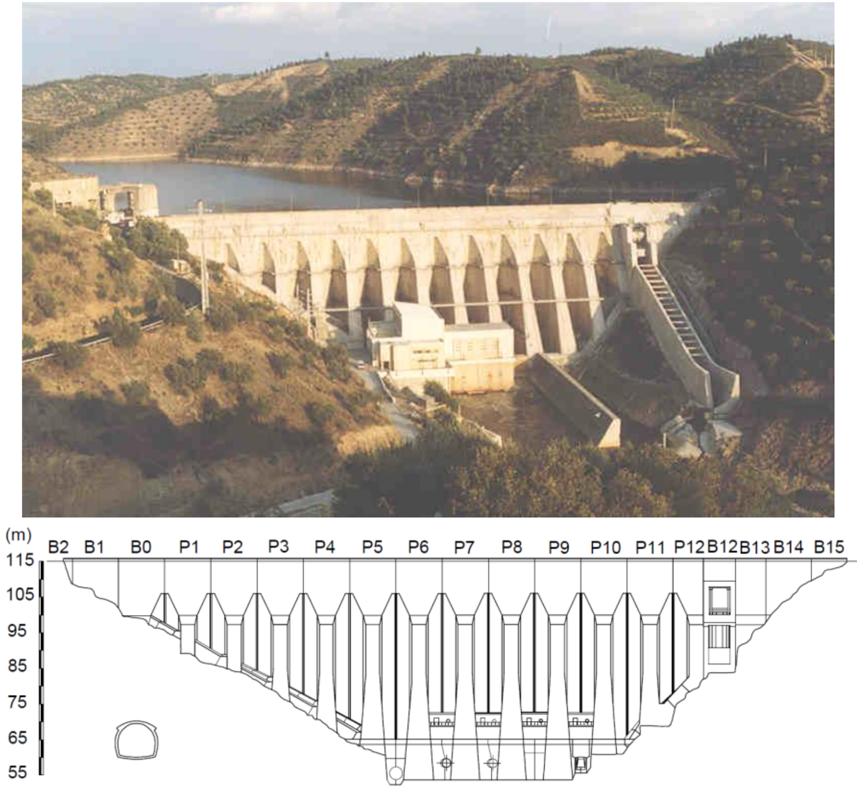
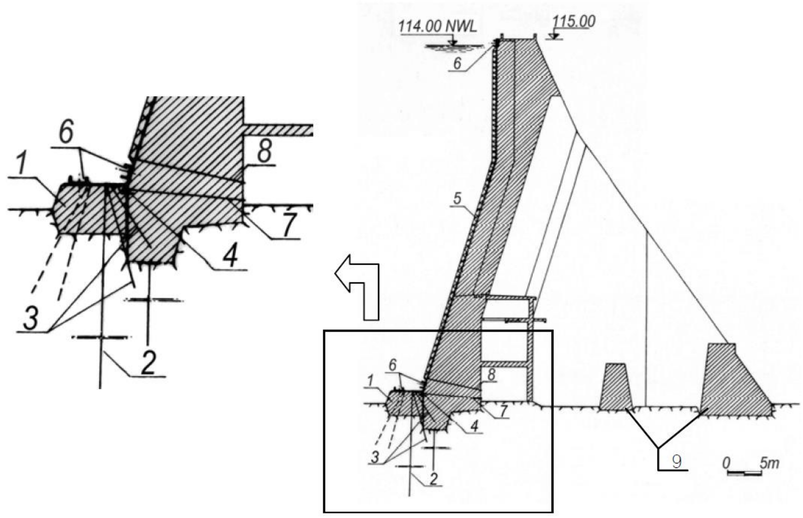


Figure 2.20: Pracana dam. Downstream view and elevation in 1992, after the rehabilitation works

The dam showed an abnormal behavior since it began operating, characterized by progressive displacements, in vertical and upstream-downstream directions, and rapid evolution of generalized cracking. There were significant water seepage through the dam body, essentially through horizontal cracks in concreting joints, with thick calcium carbonate deposits (Figure 2.21).

Figure 2.21: Pracana dam. Map cracking of the upstream face (on the left) and of the buttress webs (on the right) (photographs taken before the rehabilitation works)

Physical and chemical tests, carried out in samples extracted from the dam's body, confirmed the existence of the ASR and the ISR, with expansions of high magnitude. Between 1980 and 1992 the reservoir was empty. The rehabilitation design was elaborated by EDP in 1985. Figure 2.22 shows some of the rehabilitation works carried out between 1988 and 1992 [33, 34, 35].



1 – Upstream plinth

2 – Impervious grout curtain

3 – Chemical grouting

4 – Water stop

5 – Geomembrane

6 – Watertight fastenings

7 – Foundation drainage

8 – Membrane drainage

9 – Concrete struts

Figure 2.22: Pracana dam. Global waterproofing system and downstream bracing struts [34]

The interventions in the dam included: i) the general treatment of the concrete, for its regeneration, through the injection of cracks with epoxy resins and cement grout; ii) the installation of an impermeable membrane on the upstream face, including the construction of a foundation plinth at the upstream toe; iii) the construction of two concrete struts downstream, to lock the webs of the buttresses, one close to the downstream toe and the other in an intermediate position; iv) the consolidation of the foundation and the execution of new waterproofing and drainage curtains, the waterproofing curtain being done from the top of the new upstream plinth; and v) the recovery and reinforcement of the monitoring system. A frontal auxiliary spillway was also built on the left bank, to provide the dam with sufficient discharge capacity. The power plant was updated, which included the construction of the water intake for a new generation unit.

Concrete regeneration was carried out through: i) the treatment of cracks with an opening greater than 0.5 mm (which corresponded to about 20% of the mapped cracks), by cement grouting; and ii) the mass treatment of the concrete, by injection of epoxy resin in the cracks with smaller opening [34] (Figure 2.23).

The waterproofing of the upstream face was performed with a non-adherent geomembrane (Figure 2.24). A waterproofing system patented by Carpi was used, formed by a 2.5 mm thick waterproof flexible PVC geomembrane, thermally coupled to a 500 g/m^2 non-woven geotextile.

The monitoring system was rehabilitated and reinforced, to better monitor the structural behavior and the evolution of the expansion process (Figure 2.25). The following new instruments were installed: i) multiple rod extensometers, along the head of five buttresses (P1, P4, P6, P9 and P12); ii) two rod extensometers at the foundation of each buttress, one near the head and one at the downstream toe; iii) inverted plumb-lines in five buttresses (P1, P4, P6, P9 and P12), with suspension close to the crest and deep fixation in the rock mass foundation; iv) thermometers in the body of two buttresses (P5 and P7); and v) new jointmeters to control the opening/closing movements between buttress heads. The geodetic monitoring system was also revised and improved.

The rehabilitation project was supported by studies aiming the modelling of the dam's structural behavior. Later, to interpret the behavior observed before and after the rehabilitation works, new and more advanced models were developed [12, 35, 36].

The reservoir was refilled at the beginning of 1993, but between June and September 1994 it was emptied again for inspection of the waterproofing membrane, repair its connection to the plinth and reinforcement of the drainage system on its back side.

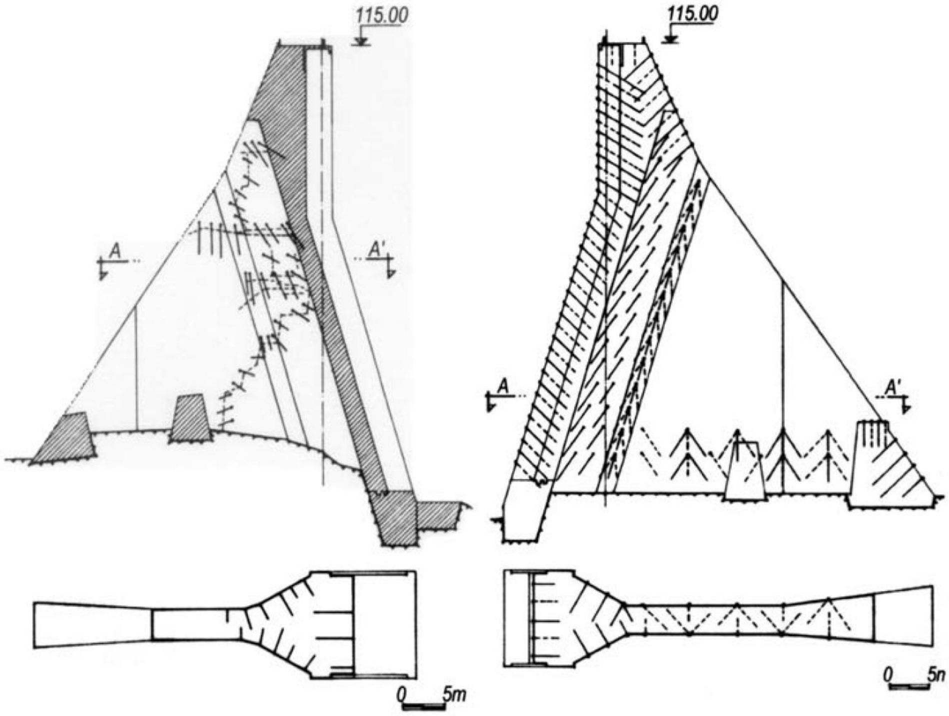
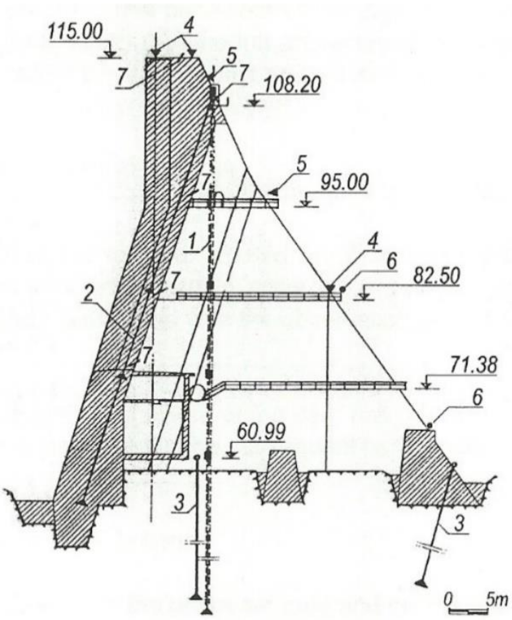


Figure 2.23: Pracana dam. Individual crack treatment (on left) and drilling pattern for mass treatment of the concrete (on right) [34]

Figure 2.24: Pracana dam. Installation of the upstream impervious membrane

The rehabilitation works of Pracana dam, carried out between 1988 and 1992, allowed the annual expansion rates in the buttresses to be reduced to around 1/8, currently presenting average values of about $7x10^{-6}$/year (Table 2.2). The drastic reduction in expansion rates after 1992 is also due to the internal drying of the concrete during the 12 years the reservoir was empty. It can be generally considered that the works in 1988-1992 produced the intended effects [37, 38], but it is important to maintain the surveyance of the dam behavior, in particular the lateral blocks of the abutments, as they have shown, in recent years, expansion rates much larger than the buttresses.



1 – Inverted plumb-line

2 – Multiple rod extensometer (dam's body)

3 – Rod extensometer (foundation)

4 – Levelling marks

5 – Geodetic targets

6 – Angles and distances measurements targets

7 – Joint meters

Figure 2.25: Pracana dam. General scheme of the new monitoring instruments installed in 1992 to measure displacements of the dam and its foundation [35]

Table 2.2: Average swelling accumulated and average annual rates, during the periods of 1952-1980 and 1992-2019, obtained through the analysis of the crest vertical displacements (levelling) [38]

Buttress	Height (m)	First period: October 1952 to May 1980			Second period: December 1992 to January 2019		
		Accumulated vertical displacements (mm)	Accumulated vertical strain (x10^{-6})	Annual swelling rate (x10^{-6})	Accumulated vertical displacements (mm)	Accumulated vertical strain (x10^{-6})	Annual swelling rate (x10^{-6})
B2	3.7	-	-	-	2.7	716	27.4
B1	10.8	5.6	521	18.4	7.4	684	26.2
B0	17.2	-	-	-	6.0	349	13.4
P1	25.1	-	-	-	4.9	195	7.5
P2	32.1	33.1	1050	37.1	4.4	137	5.3
P3	38.6	-	-	-	3.0	76	2.9
P4	47.3	31.8	550	19.4	3.2	67	2.6
P5	57.8	-	-	-	4.2	73	2.8
P6	63.0	32.1	535	18.9	5.1	80	3.1
P7	62.3	-	-	-	5.5	88	3.4
P8	62.3	32.2	535	18.9	6.4	102	3.9
P9	62.2	-	-	-	5.5	88	3.4
P10	60.9	26.8	466	16.5	5.2	85	3.2
P11	53.4	33.1	685	24.2	6.3	117	4.5
P12	38.4	30.6	836	29.5	5.5	142	5.4
B13	24.2	-	-	-	3.3	134	5.1
B14	12.6	25.5	2024	71.5	9.3	734	28.1
B15	4.2	-	-	-	11.2	2667	102.2

3. SYNTHESIS OF THE RESULTS FOR ALL THE DAMS

Table 3.1 shows the types of studies carried out to characterize the expansive phenomena in the 20 Portuguese dams affected by swelling reactions and Table 3.2 presents a summary regarding the type of reaction and the appraisal of the vertical expansions of each dam.

The most common expansive reaction in Portuguese dams is the ASR, only in the Fagilde dam the ISR is dominant, probably due to concrete casting.

Table 3.1: Portuguese dams affected by concrete swelling. Phenomena identification and characterization procedures [1]

Dam	Structural type	Year of completion	Height (m)	Phenomena identification and characterization		
				Visual inspections	Results from monitoring systems	Results from physical and chemical tests
Santa Luzia	Arch	1942	76	V1 + V2	M2 + M3	T1 + T2 + T3
Alto Ceira	Arch	1949	36	V1 + V2 + V3	M2 + M3	T1 + T2 + T3
Penide	Gravity	1949	18	V1 + V2	M2	T2
Pracana	Buttress	1951	60	V1 + V2 + V3	M1 + M2 + M3	T1 + T2 + T3
Covão do Meio	Arch	1953	28	V1 + V2	M2	T1 + T2 + T3
Cabril	Arch	1954	132	V1 + V2 + V3	M1 + M2	T1 + T3
Bouçã	Arch	1955	65	V1 + V2	M1 + M2	-
Caniçada	Arch	1955	76	V1	M1 + M2 + M3	-
Picote	Arch	1958	99	V1 + V3	M1 + M2 + M3	T1
Miranda	Buttress	1961	80	V1 + V2	M1 + M2 + M3	T1 + T2 + T3
Bemposta	Arch-gravity	1964	87	V1 + V2 + V3	M1 + M2 + M3	T1
Alto Rabagão	Arch	1964	94	V1 + V3	M1 + M2	T1
Caia	Buttress	1967	52	V1 + V2	M2	T1
Fratel	Gravity	1973	43	V1 + V2	M2 + M3	T1 + T2 + T3
Penha Garcia	Gravity	1980	25	V1 + V2	M2 + M3	-
Aguieira	Multiple arch	1981	89	V1 + V2	M1 + M2	-
Raiva	Gravity	1981	36	V1 + V2	M2	-
Coimbra	Gravity	1981	40	V1 + V2	-	-
Monte Novo	Gravity	1982	30	V1	M2	T1
Fagilde	Gravity	1984	27	V1 + V2	M2 + M3	T1 + T2 + T3

V1 – Diffuse (map) cracking
V2 – Linear cracking
V3 – Gel exudation

M1 – Progressive strains
M2 – Progressive displacements
M3 – Differential movements of joints

T1 – Product identification
T2 – Strength and deformability
T3 – Expansibility

Table 3.2: Portuguese dams affected by concrete swelling. Reaction and aggregate types and vertical strains due to swelling [1]

Dam	Year of completion	Height (m)	Reaction type	Aggregates		Average accumulated vertical strain in 2020 ($x10^{-6}$)	Average annual rate in the last 10 years ($x10^{-6}$)
				Coarse	Sand		
Santa Luzia	1942	76	ASR	Quartzite	Quartzite	600 to 2400	5 to 20
Alto Ceira	1949	36	ASR	Quartzite	Quartzite	600 to 4000	5 to 50
Penide	1949	18	ASR (?)	Granite	?	?	10
Pracana	1951	60	ASR/ISR	Quartzite	Quartzite	800 a 1400	0 to 50
Covão do Meio	1953	28	ASR	Granite	?	?	30
Cabril	1954	132	ASR/ISR	Granite	?	80	< 5
Bouçã	1955	65	ASR (?)	Granite	?	80	< 5
Caniçada	1955	76	ASR	Granite	?	160	5 to 10
Picote	1958	99	ASR	Granite	Granite	100	< 5
Miranda	1961	80	ASR	Granite	Granite	20	5
Alto Rabagão	1964	94	ASR	Granite	Granite	100	< 5
Bemposta	1964	87	ASR	Granite	?	250	5 to 10
Caia	1967	52	ASR/ISR	Granite	Quartzite	?	?
Fratel	1973	43	ASR/ISR	Granite	Siliceous	300 to 400	5 to 15
Penha Garcia	1980	25	ASR (?)	Quartzite	Siliceous	70	< 5
Coimbra	1981	40	ASR (?)	?	Siliceous	?	?
Aguieira	1981	89	ASR	Granite	Siliceous	120	< 5
Raiva	1981	36	ASR (?)	Granite	Siliceous	250	< 10
Monte Novo	1982	30	ASR	Granite	?	100	5
Fagilde	1984	27	ISR	Limestone	Siliceous	900 to 2400	20 to 50

There are 14 dams affected by swelling reactions, usually of the ASR type, in which granitic aggregates, some highly deformed and containing subgrained quartz, were used in the concrete mix: Alto Rabagão, Caniçada and Penide, in the northwest of the country; Miranda, Picote and Bemposta (strained granite), in the international section of Douro river; Aguieira, Raiva, Cabril, Bouçã, Covão do Meio and Fratel, in the central part of the country; and Monte Novo and Caia, south of Tagus river. Those dams were built between 1942 and 1982, exhibiting a behavior characterized by very low expansion rates during the first 20 to 30 years, but then showing increasing rates over time. Table 3.3 shows the average values, in 2020, of the free strains observed in the stress-free strain-meters and the vertical strains estimated from the results of the crest levelling, as well as the respective evolution rates in the last 10 years. Figure 3.1 shows the evolution, up to 2015, of the aforementioned strains, for some of these dams [39]. This type of behavior is related to the slow reactivity of aggregates with undeformed crystalline silica [40], so those that were built with concrete produced with (strained) quartzite aggregates developed expansive reactions very early on, while in dams where granitic aggregates were used the initial dormant period of this phenomenon reached more than 20 to 30 years.

Table 3.3: Portuguese dams built with granite aggregates and affected by the ASR. Average values, in 2020, of the monitored free strains by stress-free Carlson strain-meters and strains obtained from geodetic levelling [1]

Barragem	Age in 2020 (years)	Free strain measured by the stress-free strain-meters		Vertical strain estimated from the vertical displacements measured by geodetic levelling of the crest	
		Average accumulated strain ($x10^{-6}$)	Average annual rate in the last 10 years ($\times10^{-6}$)	Average accumulated vertical strain ($x10^{-6}$)	Average annual rate in the last 10 years ($\times10^{-6}$)
Penide	66	-	-	200 (since 1997)	10
Covão do Meio	62	-	-	600 (since 1985)	30
Cabril	61	250	10	80	< 5
Bouçã	55	150	< 10	80	< 5
Caniçada	60	150 (since 1990)	< 10	170	5 to 10
Picote	57	200	< 10	100	< 5
Miranda	54	300	10	200	5
Alto Rabagão	51	150	10	100	< 5
Bemposta	51	200	< 10	250	5 to 10
Caia	48	-	-	50 (12 years, 1996-2008)	< 5 (2003-2008)
Fratel	42	-	-	200 (since 2000)	5 to 15
Aguieira	34	150	< 10	120	< 5
Raiva	34	-	-	250	< 10
Monte Novo	33	-	-	100	5

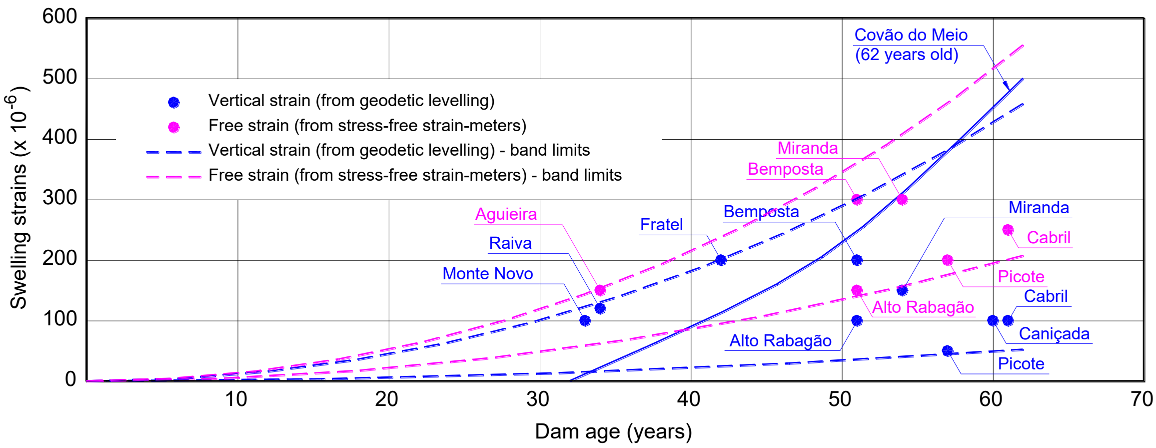


Figure 3.1: Portuguese dams built with granite aggregates and affected by the ASR. Evolution until 2015 of the average values of the monitored free strains by stress-free Carlson strain-meters and strains obtained from geodetic levelling, in terms of the dam age [39]

Table 3.4 presents a summary of the types of tests carried out “in situ” and of structural modelling, using the finite element method, for the interpretation of the observed behavior of Portuguese dams affected by concrete swelling. It can be noted that the adopted models consider, for the concrete properties, the aging viscoelasticity and the damage to simulate the delayed behavior and the cracking formation and propagation, respectively, as well as the generation of structural expansions as a function of temperature, internal humidity and stress fields.

Table 3.4: Portuguese dams affected by concrete swelling. In situ tests and structural modelling for the interpretation of the observed behavior [1]

Dam	Structural type	Year of completion	Height (m)	"In situ" tests and modelling of the structural behavior by the finite element method	
				"In situ" tests	Structural modelling
Santa Luzia	Arch	1942	76	S1 + S3 + S4	E4 + E5 + E6 + E7
Alto Ceira	Arch	1949	36	S1 + S2 + S3 + S4 +	E1 + E2
Penide	Gravity	1949	18	-	-
Pracana	Buttress	1951	60	S1 + S2 + S5	E3 + E4 + E5 + E6 + E7
Covão do Meio	Arch	1953	28	S1 + S4	E3 + E4 + E5 + E6 + E7
Cabril	Arch	1954	132	S1 + S2 + S3 + S4	E1
Bouçã	Arch	1955	65	S1 + S4	-
Caniçada	Arch	1955	76	-	-
Picote	Arch	1958	99	-	-
Miranda	Buttress	1961	80	S3	-
Bemposta	Arch-gravity	1964	87	-	E4 + E5 + E6 + E7
Alto Rabagão	Arch	1964	94	-	-
Caia	Buttress	1967	52	S2	-
Fratel	Gravity	1973	43	-	E4 + E5 + E6 + E7 (*)
Penha Garcia	Gravity	1980	25	-	-
Aguieira	Multiple arch	1981	89	S3 + S4	E1 + E5
Raiva	Gravity	1981	36	-	-
Coimbra	Gravity	1981	40	-	-
Monte Novo	Gravity	1982	30	-	-
Fagilde	Gravity	1984	27	S1	E4 + E5 + E6 + E7

"In situ" tests:
S1 – Mapping of cracks and of other occurrences
S2 – Ultrasonic
S3 – Forced vibration
S4 – Natural vibration
S5 – Stress measurement
(*) ongoing

Type of structural modelling:
E1 – Elastic
E2 – Elastoplastic
E3 – Damage
E4 – Aging viscoelastic
E5 – Swelling generation
E6 – Dependence from temperature and humidity
E7 – Dependence from the stress state

The dams that showed expansions greater than 1000×10^{-6} are, as mentioned, Santa Luzia, Alto Ceira, Pracana and Fagilde dams. Only Alto Ceira and Fagilde dams showed annual expansion rates above 100×10^{-6}. In other affected dams, the deterioration processes are of low to moderate magnitude and have been developing at reduced rates, except for Covão do Meio dam, where, in recent years, there have been increased rates of vertical displacement corresponding to strains of about 30×10^{-6}/year.

Alto Ceira dam, which was hopelessly deteriorated, was replaced in 2013.

Santa Luzia dam seems to have practically exhausted its reaction and its structural damage is only significant in the zone connecting the main arch to the arch-gravity in the left bank. Its structural performance remains satisfactory. The dam's owner has been carrying out studies to schedule a rehabilitation intervention in the next years.

The rehabilitation of Pracana dam, carried out between 1988 and 1992, with the concrete integrity recovered and the installation of a geomembrane on the upstream face, is worldwide recognized as a case of enormous success (as already mentioned, the emptying of the reservoir for 12 years dried the concrete, which contributed decisively to this reality). Current expansion rates are reduced, except in the abutment blocks.

Fagilde dam has been monitored and remains functional and safe. If swelling rates maintain the current standards, the dam may need a major intervention in the medium term. However, this may not happen as the construction of a new dam downstream is being considered, with the goal of increasing the water volume stored in the reservoir.

ACKNOWLEDGMENT

Thanks are due to the Portuguese Environment Agency (APA), EDP, Engie, Águas de Portugal (AdP), Aguia Enlica and Associação de Beneficiários do Caia for the permission granted to publish elements of the observation of the dams mentioned in this paper.

REFERENCES

[1] Batista A (2021) Structural management and rehabilitation of dams affected by concrete swelling reactions *(in Portuguese)*. Research program, LNEC, Lisbon.

[2] Batista AL, Piteira Gomes J (2012). Practical assessment of the structural effects of swelling processes and update inventory of the affected Portuguese concrete dams. First International Dam World Conferece, Maceió, Brazil.

[3] Tavares de Castro A, Ramos JM, Oliveira SB (2001). Evaluation of the behavior of an arch dam affected by a swelling process in the concrete. ICOLD European Symposium, Geiranger, Norway.

[4] LNEC (Reis, MOB) (1991). Study of the concrete of Alto Ceira dam. Collaboration of the Chemistry Unit *(in Portuguese)*. Report 170/1991, DMC/NQ, Lisbon.

[5] LNEC (Oliveira S, Pina C, Ramos JM, Tavares de Castro A) (1995). Study of the behavior of Alto Ceira dam. Structural safety assessment *(in Portuguese)*. Report 41/1995, DB/NDE/NEE/NO, Lisbon.

[6] LNEC (Rolo FM) (1991). Observation of Alto Ceira dam. Study of the concrete cracking (survey carried out in November 1986) *(in Portuguese)*. Report 41/1991, DB/NO, Lisbon.

[7] Berberan A, Portela EA, Boavida J (2006). Assisted visual inspection of dams as a tool for structural safety control. A case study. Hydro2006, Porto Carras, Greece.

[8] LNEC (Henriques MJ) (2013). Thermal survey of concrete dam faces to support the control of the evolution of pathologies *(in Portuguese)*. Report 425/2013, DBB/NGA, Lisbon.

[9] LNEC (Leitão N, Tavares de Castro A) (2004). Structural safety assessment of Alto Ceira dam *(in Portuguese)*. Report 412/2004, DBB/NO/NFOS, Lisbon.

[10] Cunha J, Silva Matos D (2008). The new Alto Ceira dam. An example of the substitution of an existing dam. Hydro2008, Ljubljana, Slovenia.

[11] Silva Matos D, Cunha J (2015). The substitution of an existing dam affected by AAR. The case of Alto Ceira dam. 25th International Congress on Large Dams (ICOLD), Q99-R35, Stavanger, Norway.

[12] Piteira Gomes J (2007). Structural behavior modelling of concrete dams subject to swelling reactions *(in Portuguese)*. PhD thesis, Faculdade de Ciências e Tecnologia, Universidade Nova de Lisboa, Lisbon.

[13] LNEC (Rodrigues OV) (1966). Observation of Santa Luzia dam in February 1966 *(in Portuguese)*. Report, SB/DMG, Lisbon.

[14] LNEC (Matos MEC) (1981). Visual inspection of Santa Luzia dam. Cracks and infiltrations *(in Portuguese)*. Report, DB/NO, Lisbon.

[15] LNEC (Andrade M) (1998). Santa Luzia dam. Survey of the existing cracks between 88.85 m and 112.70 m elevations on the upstream and downstream faces. Observation made during the summer of 1998 *(in Portuguese)*. Technical note 24/1998, DB/NO, Lisbon.

[16] Hidrorumo (1994). Santa Luzia dam. Monitoring plan *(in Portuguese)*. EDP, Porto.

[17] LNEC (Ramos JM, Tavares de Castro A) (1991). Observation of Santa Luzia dam. Opinion on the behavior of the dam *(in Portuguese)*. Report 35/1991, DB/NO, Lisbon.

[18] LNEC (Silva HS) (1996). Santa Luzia dam. Observation and analysis of concrete samples *(in Portuguese)*. Report 212/1996, DB/NO, Lisbon.

[19] LNEC (Ramos JM, Vieira LM) (1997). Characterization tests of the concrete rheology of the Santa Luzia dam *(in Portuguese)*. Report 313/1997, DB/NDE/NEE/NO, Lisbon.

[20] Piteira Gomes J, Batista AL, Oliveira SB (2004). Analysis of concrete dams under swelling processes. 12th International Conference on Alkali-Aggregate Reaction in Concrete, Beijing, China.

[21] Piteira Gomes J, Cunha J, Batista AL, Almeida F (2021). Monitoring and assessment of the structural effects due to the ASR in Santa Luzia dam (Portugal). 16th International Conference on Alkali-Aggregate Reaction (ICAAR 2020-2022), Lisbon.

[22] LNEC (Tavares de Castro A) (2001). Study of the behavior of Fagilde dam between 1985 and 2001 *(in Portuguese)*. Report 289/2001, DBB/NO, Lisbon.

[23] LNEC (Santos Silva A, Bettencourt Ribeiro A) (2010). Characterization of the expansive action of Fagilde dam concrete *(in Portuguese)*. Report 119/2010, DM/NB/NMM, Lisbon.

[24] LNEC (Piteira Gomes J, Tavares de Castro A, Ramos JM, Batista AL) (2003). Fagilde dam. Methodology for characterizing the concrete expansive process *(in Portuguese)*. Report 12/2003, DBB/NO, Lisbon.

[25] LNEC (Andrade M) (2006). Fagilde dam. Survey of the state of cracking carried out in 2004 *(in Portuguese)*. Technical note 58/2006, DBB/NO, Lisbon.

[26] LNEC (Ribeiro S, Bettencourt Ribeiro A) (2007) – Fagilde dam. Study of the concrete creep *(in Portuguese)*. Report 15/2007, DM/NB, Lisbon.

[27] FCUP (Fernandes I, Ribeiro MA, Noronha F) (2006). Fagilde Dam. Petrographic analysis of concrete *(in Portuguese)*. Report, Departamento de Geologia, Faculdade de Ciências da Universidade do Porto, Porto.

[28] LNEC (Piteira Gomes J, Tavares de Castro A) (2008). Fagilde Dam. Complementary instrumentation plan to the monitoring system *(in Portuguese)*. Report 111/2008, DBB/NO, Lisbon.

[29] Piteira Gomes J, Batista AL, Silva JC (2016). Evaluation of the effects of the deterioration process of the concrete of Fagilde dam. Part 1 – Characterization of the structural condition *(in Portuguese)*. II Encontro Luso-Brasileiro de Degradação de Estruturas de Betão, LNEC, Lisbon.

[30] Piteira Gomes J, Batista AL, Silva JC (2016). Evaluation of the effects of the deterioration process of the concrete of Fagilde dam. Part 2 – Analysis of observed behavior and mitigation recommendations *(in Portuguese)*. II Encontro Luso-Brasileiro de Degradação de Estruturas de Betão, LNEC, Lisbon.

[31] Sousa, SPA (2016). Analysis of the structural behavior of dams affected by concrete swelling. Application to Fagilde dam *(in Portuguese)*. Master thesis, Faculdade de Ciências e Tecnologia da Universidade Nova de Lisboa, Lisbon.

[32] Piteira Gomes J, Batista AL, Sousa SPM (2017). Swelling effects in Fagilde dam (Portugal). First approach to structural analysis and interpretation. Swelling Concrete in Dams and Hydraulic Structures (DSC 2017), Civil Engineering and geomechanics series, ISTE-Wiley, p. 218-234.

[33] Silva Matos D, Silva HS, Pinho JS, Camelo A (1995). Deterioration of Pracana dam due to the ASR. Main features and repair works. Second International Conference on Alkali-Aggregate Reactions in Hydroelectric Plants and Dams, Chattanooga, EUA.

[34] Liberal AO, Silva Matos D, Camelo A, Pinho JS, Tavares de Castro A, Machado Vale J (Work Group of the Portuguese National Commission on Large Dams) (2003). Ageing process and rehabilitation of Pracana dam. 21th International Congress on Large Dams (ICOLD), Q82-R8, Montreal, Canadá.

[35] Pinho JS, Tavares de Castro A, Piteira Gomes J Silva Matos D, Ferreira JI, Camelo A (Work Group of the Portuguese National Commission on Large Dams) (2003). Deterioration assessment and observed behavior of Pracana dam. 21th International Congress on Large Dams (ICOLD), Q82-R12, Montreal, Canadá.

[36] Piteira Gomes J, Batista AL, Oliveira SB (2009). Damage-chemo-viscoelastic model on the analysis of concrete dams under swelling processes. 2nd International Conference on Long Term Behaviour of Dams, Graz, Austria.

[37] Silva Matos D, Ferreira JI, Camelo A, Piteira Gomes J, Scuero A, Vaschetti G, Machado Vale J (2013). Pracana dam 20 years after rehabilitation. Observed behaviour of the dam and of its exposed geomembrane. ICOLD 2013 International Symposium, Seattle, EUA.

[38] Piteira Gomes J, Silva Matos D, Batista AL, Ferreira JI (2021). Structural behavior of Pracana dam 30 years after rehabilitation due to severe ISR-ASR damage. 16th International Conference on Alkali-Aggregate Reaction (ICAAR 2020-2022), Lisbon.

[39] Batista AL, Piteira Gomes J (2016). Characteristic behavior of the Portuguese large concrete dams built with granite aggregates and affected by the ASR. 15th International Conference on Alkali-Aggregate Reaction (ICAAR 2016), São Paulo, Brazil.

[40] Santos Silva A, Fernandes I, Ferraz AR, Soares D (2017). Can certain alkali minerals explain the slow reactivity of granitic aggregates in dams? Swelling Concrete in Dams and Hydraulic Structures (DSC 2017), Civil Engineering and Geomechanics Series, ISTE-Wiley, p. 93-105.