The contribution of numerical modelling to assess dam safety: the case of buttress, hollow and multiple arch/slab dams

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ABSTRACT: Buttress (both solid or hollow) or multiple arch/slab dams are a type practically abandoned internationally, even if there are some relatively recent examples of the 70s-80s (e.g., Haen dams, 1963 and Storfoss, 1982, in Norway). In the Italian context there are numerous examples of this type of works built between the two World Wars and immediately after the II World War. A total of 37 dams are in operation (40% buttress, 30% hollow gravity, 30% multiple arch/slab). The problems of these structures are known: cracks (generally caused by phenomena of thermal origin, differences of the buttresses height, expansive chemical reactions, etc.), degradation associated with environmental conditions (e.g., due to corrosion of the reinforcement bars when present), phenomena of aging and degradation. To these problems some critical issues are added related to compliance with current legislation which requires the verification of more stringent conditions not foreseen in the design phase (e.g., higher seismic loads or more up-to-date criteria on the evaluation of uplift pressures). The verification of these structures has highlighted the need to systematize the knowledge of the phenomena that gave rise to the problems mentioned above, to deepen the evolutionary dynamics of decay and cracking states, to share the experiences on rehabilitation interventions. Numerical modelling has certainly contributed significantly to the understanding of the phenomena and to the evaluation of the structural behaviour depending on the applied actions. Some of the above-mentioned problems were topics of the Benchmarks proposed by the ICOLD Technical Committee "*Computational Aspects of Analysis and Design of Dams*". This article presents an overview of the problems concerning this type of dams and highlights the support that numerical models can offer for the evaluation of their safety and to identify the most effective interventions for long-term safety conditions.

1 INTRODUCTION

The construction of large dams is closely related to the development of the hydroelectric sector which was the backbone of the industrialization process in many European countries since the last part of the nineteenth century and the beginning of the twentieth.

The need to exploit the water resource for energy purposes leads to a great development of the dams; just think that Italy went from about a dozen dams at the end of the nineteenth century to almost 400 only in the first half of the twentieth century.

As in all industrial processes, the necessity to increase the hydroelectric production and development necessarily led to the need to maximize benefits above all by reducing the costs and construction times of dams. The goal was to achieve the maximum manufacturing economy while still ensuring an adequate degree of safety.

These are the principles, combined with the socio-economic context that characterized those years, which led to the evolution of the construction techniques of the dams.

At the base of this process there are then some fundamental elements such as:

- the evolution of material and labour costs;
- progress in geological and geotechnical investigations and in the treatment of foundation soils;
- advances in the static assessment of structures;
- the knowledge acquired on the basis of the monitoring of the although still few works in operation;
- advances in the material technology, in the construction machinery and equipment and in the entire production process.

Already at the beginning of the twentieth century there was a trend towards the optimization of the shape of dams marked by an economy of volume.

The first constructions based on this evolutionary trend, focused on a decisive saving of volume and therefore of materials, occurred in the first twenty-five years of the 1900s with concrete buttress dams connected by slabs or vaults. However, the collapse of the Gleno Dam, belonging to this type of construction, on 1st December 1923 effectively sanctioned the end of this type of dams [Barbisan, 2007]. Figure 1 shows two images of the Gleno Dam before and after the collapse.

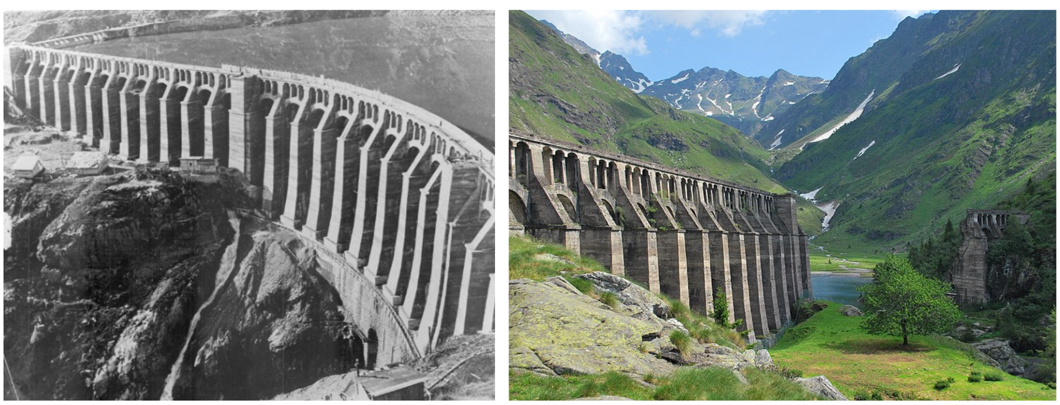

Figure 1 – Gleno Dam before (left) and after (right) the collapse.

From the investigations conducted to identify the responsibilities, it appears that the cause of the collapse was attributable to the poor execution of the massive gravity foundation pad that blocked a gorge in the central part of the work.

This event generated a sense of mistrust towards "lightened" dams in general, which in the years immediately following led to the prevalence of more massive structures that found development in Italy until the early 1960s.

Subsequently, the increase in the cost of labour, having a much greater impact for these works than for gravity dams, no longer found compensation in the reduction of volumes. The evolution of construction machinery has also contributed to the abandonment of “lightened” gravity dams, favouring the evolution of more massive gravity structures and, more recently, with a reduced cement content (this is the case of Rolled Compacted Concrete dams).

Figure 2, Figure 3 and Figure 4 show the three different types of dams that are the subject of this report: buttress, hollow gravity, multiple arch/slab dams, respectively.

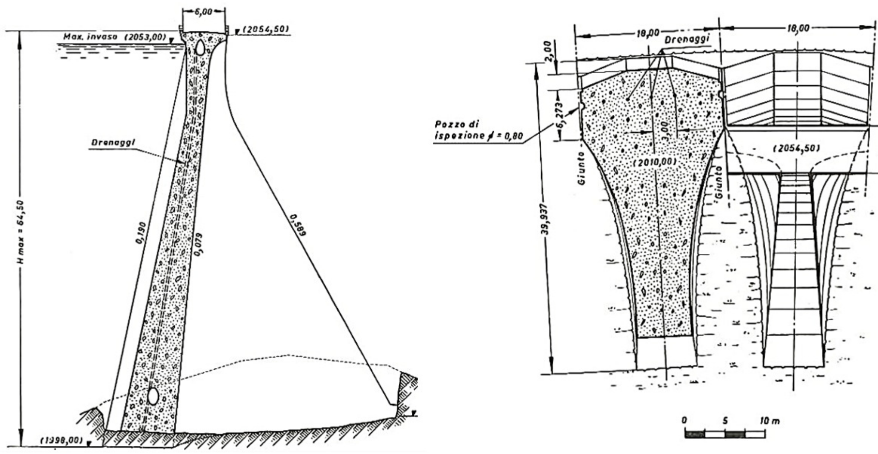

Figure 2 – Buttress dam.

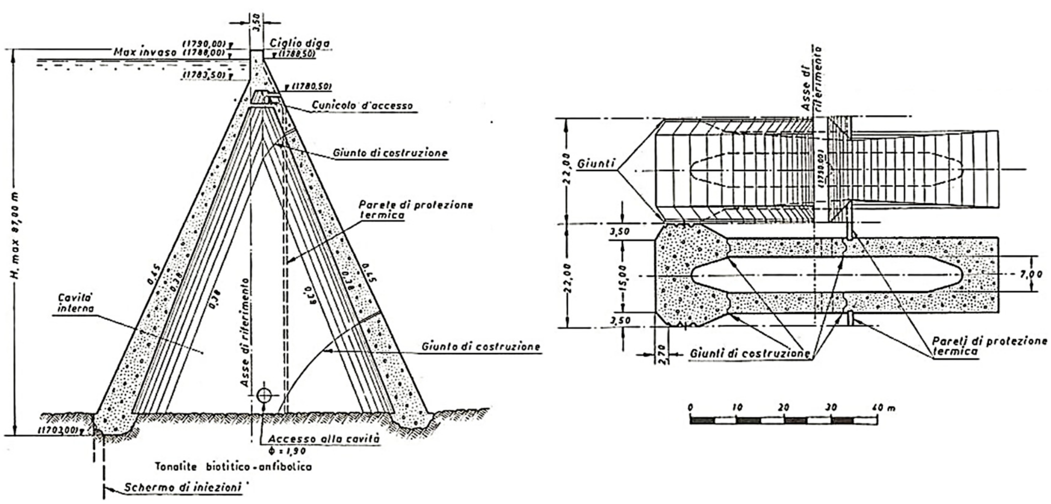

Figure 3 – Hollow gravity dam.

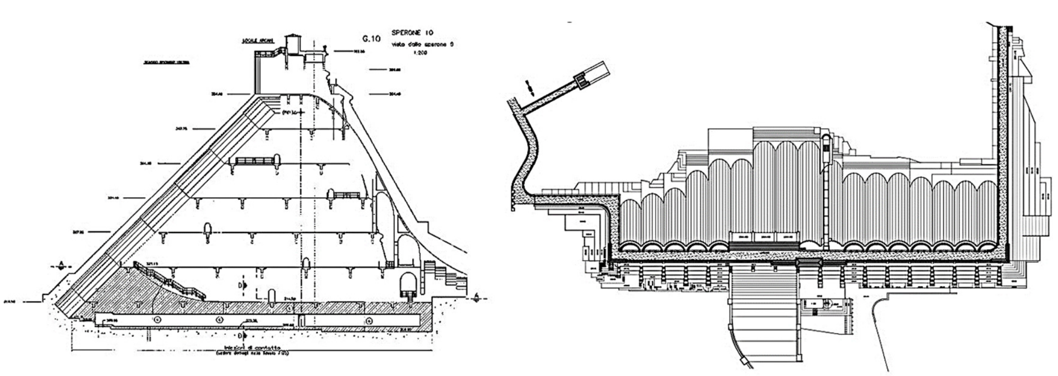

Figure 4 – Multiple arches dam.

2 MAIN PROBLEMS AND RESTORATION INTERVENTIONS

To systematize the knowledge relating to the behaviour of these structural types and the most appropriate criteria for assessing their safety, ITCOLD has set up the Working Group "*Behaviour, problems, rehabilitation of hollow gravity, buttress and multiple arch/slab dams*" and assigned the following Terms of Reference:

- Reconnaissance of the construction aspects.
- Reconnaissance on problems associated with this dam types.
- Reconnaissance of the remediation works.
- Methodologies of investigation.
- Monitoring methodologies.
- Criteria and methodologies for safety re-evaluation.

Figure 5 shows the trend of the construction of dams in Italy and the progression of rehabilitation interventions since the 1970s.

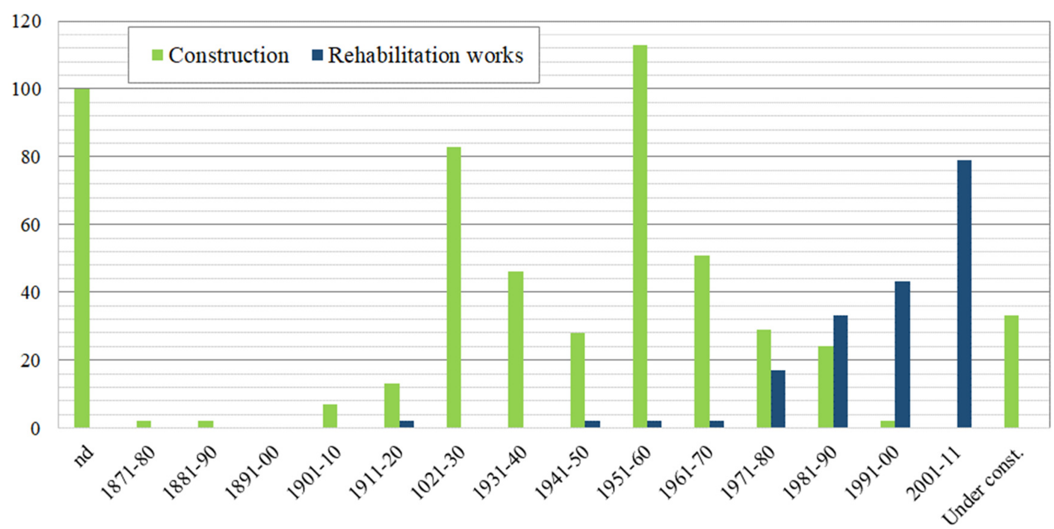

Figure 5 – Progress of the construction of dams in Italy and the progression of rehabilitation interventions since the 1970s.

Before addressing the specific problems of this type of dams, it is appropriate to recall what is reported in the ITCOLD Bulletin [ITCOLD, 2018] regarding the various criticalities encountered in the dams of any type in operation and the related remediation interventions that mainly concerned structural deficiencies and inadequacy of the outlet works.

Over 60% of the reports refer to:

- Structural problems.
- Insufficient impermeability of the dam body and of the grout cut off.
- Inadequate response to external or internal actions.

Over 30% of the reports belong to the category of inadequacy of the outlet works:

- Insufficient dimensioning of the outlet works.
- Inadequacy of the interception organs.
- Inadequate response to external actions (floating or sedimented material).

Table 1 shows the list of 37 Italian dams of this type indicating their main characteristics and whether rehabilitation works have been carried out.

Table 1. Italian multiple arch/slab, buttress and hollow gravity dams.

Dam name	Year construction	Type	Height (m)	Volume of reservoir (Mm^3)	Rehabilitation works
Alto Temo	1984	Buttress	54.1	91.1	
Ancipa	1952	Hollow gravity	104.4	30.4	**YES**
Bau Muggeris	1949	Hollow gravity	58.7	61.4	**YES**
Bau Pressiu	1972	Buttress	52.9	8.5	
Brugneto	1964	Buttress	22.5	0.95	
Casoli	1958	Buttress	47.0	21.0	
Combamala	1916	Multiple slabs	35.0	0.4	Decommissioned
Corbara	1963	Buttress	52.0	192.0	**YES**
Fedaia	1954	Buttress	63.9	16.7	
Fontanaluccia	1928	Multiple arches	40.0	2.7	
Gioveretto	1956	Buttress	81.4	19.98	**YES**
Lago di Trona	1942	Hollow gravity	53.0	5.35	**YES**
Lago Eugio	1959	Buttress	48.5	4.95	**YES**
Lago Inferno	1944	Hollow gravity	37.0	4.17	**YES**
Lago Venina	1926	Multiple arches	44.5	11.19	**YES**
Liscia	1962	Hollow gravity	65.0	105.13	
Lomellina	1910	Buttress	19.9	0.25	**YES**
Malga Bissina	1957	Hollow gravity	81.0	61.0	
Malga Boazzo	1956	Hollow gravity	53.5	12.26	
Molato	1928	Multiple arches	52.6	8.24	**YES**
Montagna Spaccata2	1958	Buttress	14.4	9.05	
Ozola	1029	Multiple arches	27.5	0.09	
Pantano D'Avio	1952	Hollow gravity	59.0	12.67	**YES**
Pavana	1925	Multiple arches	52.0	0.9	
Pian Sapeio	1926	Multiple arches	17.5	0.22	**YES**
Poglia	1950	Hollow gravity	49.4	0.5	**YES**
Ponte Vittorio	1956	Hollow gravity	36.0	0.53	
Rio Lunato	1920	Multiple arches	24.0	0.11	**YES**
Sa Cantoniera	1996	Buttress	93.2	748.2	
Sabbione	1953	Hollow gravity	61.0	44.12	
San Domenico	1927	Multiple arches	28.9	1.16	
San Giacomo	1950	Hollow gravity	83.5	64.0	**YES**
Scais	1939	Hollow gravity	60.0	9.06	**YES**
Sos Canales	1959	Buttress	47.0	4.34	
Valgrosina	1959	Hollow gravity	51.5	1.34	**YES**
Veneracolo	1958	Hollow gravity	26.9	2.55	**YES**
Vinchiana	1952	Buttress	22.2	0.12	

3 THE CONTRIBUTION OF NUMERICAL MODELING TO THE ASSESSMENT OF STRUCTURAL SAFETY

The buttress, hollow gravity and multiple arch/slab dams were a clear example of an approach to structural optimization: the "lightened" shape was obtained, in fact, from a simple but somewhat articulated evaluation between the resistance of the material (concrete or reinforced concrete) and the real work rate of the same in operating conditions.

Also, for this typology of dams, the conceptual calculation model used in the past to carry out the regulatory assessment was traced back to a triangular static scheme that recalled the much better-known scheme adopted for massive gravity dams.

The possibilities offered by numerical modelling - and of the Finite Element Method (FEM) in particular - have allowed, since the early 1990s, to deal with the study of the behaviour of these structures in a particularly effective way, both in the design phase and in the verification of behaviour in different operating conditions or to analyse exceptional or extreme conditions such as seismic actions.

Already in the first Benchmark Workshops organized by the ICOLD Technical Committee "*Computational Aspects of Analysis and Design of Dams*" topics concerning this type of dams were proposed. The salient aspects of the three cases examined between 1994 and 2005 are summarized below.

3.1 Benchmark Workshop # 3 - Theme A2: Evaluation of critical uniform temperature decrease for a cracked buttress dam (2D analysis) - Gennevilliers, France, 29th -30th September 1994.

Theme A2 proposed in the BW3 [ICOLD TCA, 1994] concerned the numerical evaluation of a uniform temperature decrease capable to give rise to the propagation of a pre-existing cracks in an idealized 2D buttress dam.

With regard to numerical modelling of cracking phenomena, different methods can be adopted according to the kind of problem to be solved, namely: initiation, stability or growth of cracks. In Theme A2, a crack stability problem was proposed.

The participants were asked to evaluate the critical uniform temperature decrease which gives rise to a critical stress state at the tip of the crack that leads to the propagation of the crack itself. This temperature initial state has been conventionally assumed to correspond to a uniform temperature of 0º C. A uniform thermal distribution in the dam (both slab and webs) is to be considered. The foundation is assumed to remain at constant average temperature (0º C).

The temperature value capable to gives rise to a critical stress state at the tip of the crack was required to be defined for five different crack lengths: 0.5 m; 2.0 m; 10.0 m; 20.0 m; 40. m.

The analyses were executed for two different foundation scenarios: rigid and deformable foundation. For both cases, a plane (2D) analysis (plane stress for the buttress and plane strain for the foundation) was required.

The main geometrical data are reported in Figure 6.

The Finite Element (FE) meshes relevant to both rigid and deformable foundation were given by the formulators to the participants. However, considering that for this type of problems the adopted FE meshes are tightly connected with the algorithm adopted in the analyses, the proposed FE meshes were considered just as a suggestion.

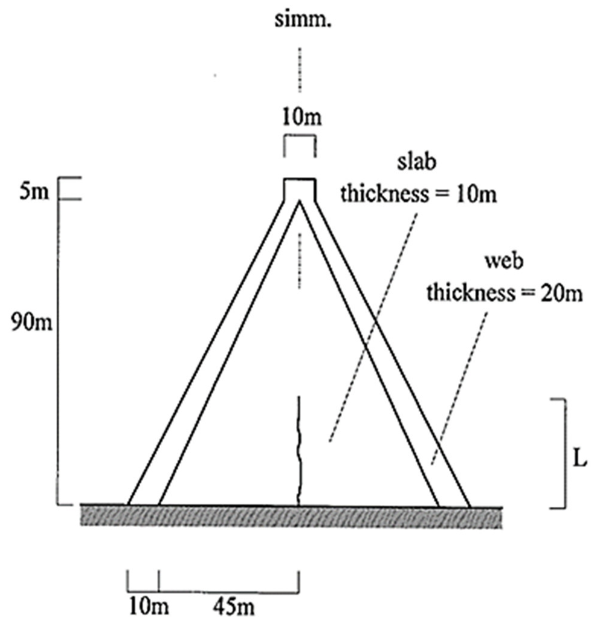

Figure 6 - Geometrical data of the ideal case proposed in the BW3.

The contributors provided a selected set of results for both foundation schemes (rigid and deformable) and for the five defined crack lengths. In addition, contour maps of horizontal and vertical displacements were requested.

The results presented by the participants showed generally good agreement. The observed discrepancies were attributed to the use of different meshes and interpolations methods.

Some results provided by the participants are presented in Figure 7 and Figure 8.

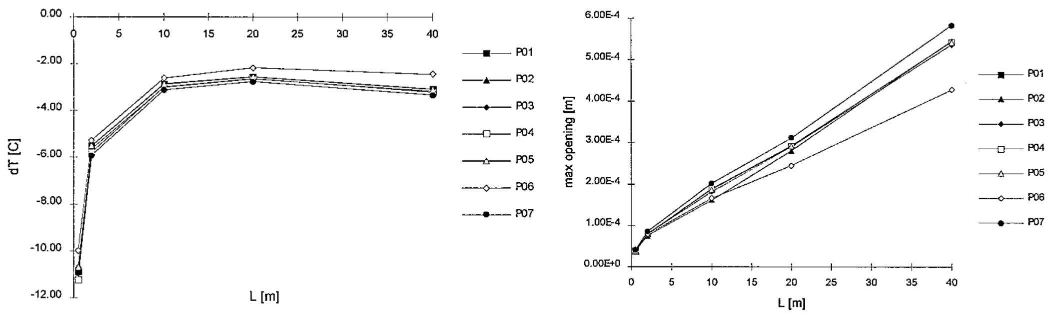

Figure 7 - Rigid foundation. Relationship between crack length and the associated critical temperature (left). Maximum openings relative to different crack length (right).

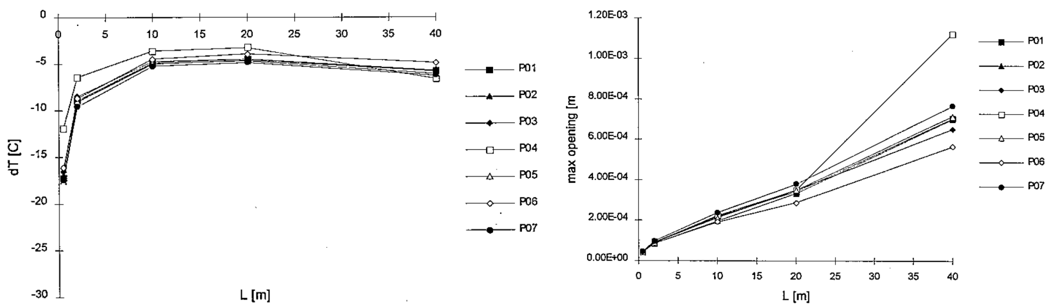

Figure 8 - Flexible foundation. Relationship between crack length and the associated critical temperature (left). Maximum openings relative to different crack length (right).

3.2 *Benchmark Workshop # 4 - Theme A2: Evaluation of stress intensity factor KI along the tip of the crack in a buttress dam under thermal gradient effects (3D analysis). - Madrid, Spain, 25th -27th September 1996.*

The Theme proposed in the BW4 [ICOLD TCA, 1996] concerns the application of a thermal gradient across the thickness of a buttress dam, using a three-dimensional analysis. The geometrical data of the idealized dam are reported in Figure 9.

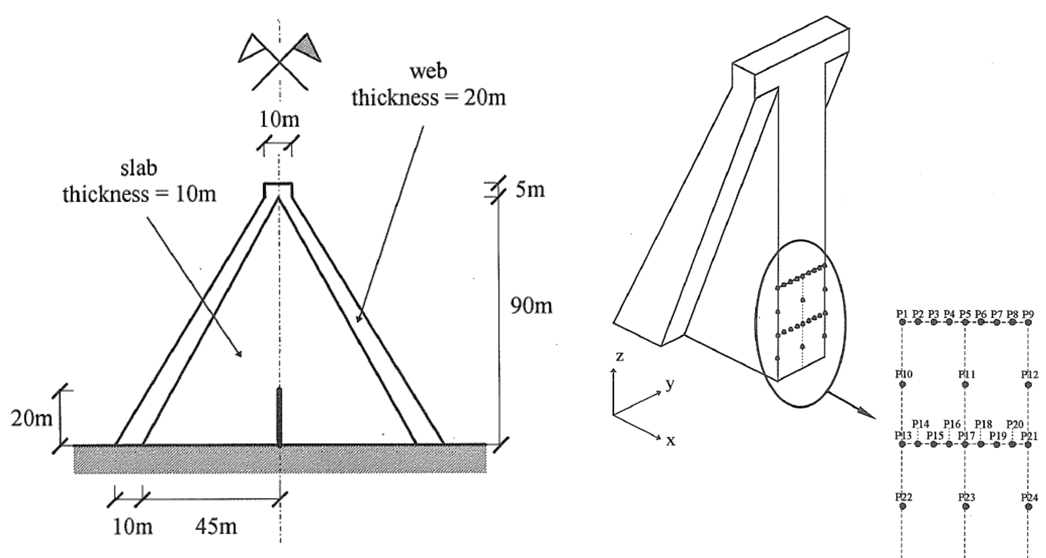

Figure 9 - Buttress dam: a) cross section with vertical crack close to foundation; b) 3D illustration of computational references points along the crack surfaces and tip.

No FE mesh was provided since a suitable mesh for Stress Intensity Factor (SIF) evaluation is tightly connected with the algorithm adopted by each participant. Nevertheless, in order to allow the comparison of the results, SIF results were requested at nine points along the crack tip.

The crack tip develops into a line across the thickness of the buttress. For sake of simplicity, the crack tip follows a straight path across the thickness of the buttress, normal to the external surface.

The analysis followed two steps: a first step with time-constant temperature loading, varying per surface, for complete opening of the crack surfaces and a second step with a periodic time history for temperature, which generates opening-closing cycles of the crack surfaces. The main challenges were:

- the temperature field evaluation as a function of time for the assigned boundary conditions;
- the stress field evaluation (3D static non-linear structural analysis, with non-linearities arising from the unilateral behaviour of the crack surfaces interaction, i.e., joints);
- the Stress Intensity Factor KI evaluation across the thickness of the buttress as a function of the position along the crack tip for each time considered.

The contributors provided detailed results of temperature and SIF along the crack tip, as well as displacements at selected points, for both the time-constant scenarios and two temperature time histories (winter and summer conditions).

SIF distributions along the crack front for one of the exercises proposed (# A2.2) provided insight on the effect of contact modelling (Figure 10).

The results are considered satisfactory in terms of validation needs, taking into account that the solutions provided are characterized by the use of different codes, different methods of calculating the stress intensity factors, and use of different type of contact elements. Hence, it was concluded that different modelling approaches were able to provide a consistent and narrow band of solutions for the engineering problem.

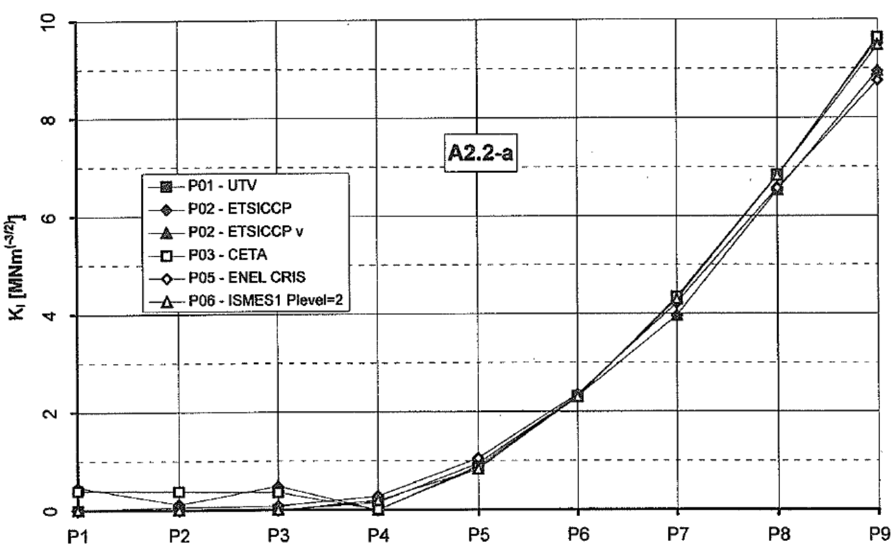

Figure 10 - K_I variation along the crack tip.

3.3 *Benchmark Workshop # 8 - Theme A: Evaluation of alkali-aggregate reaction effects on the behaviour of an Italian hollow gravity dam. - Wuhan, China, 23rd -30th October 2005.*

The Poglia Dam is a large concrete hollow gravity structure, located in the northern part of Italy, for the purpose of hydroelectric power generation. The dam is 50 m high and the crest is 137 m long. The construction works took place in 1949-1950. The dam (Figure 11, left) consists of four hollow diamond-head buttresses and two solid lateral gravity shoulders.

Since the seventies, hence roughly twenty years after its construction, the dam started to exhibit a drift in the displacements (detected by plumblines, collimation and levelling systems). In particular, in the main block the drift was estimated to be 1 mm per year in the vertical direction and 0.2 mm per year in the upstream-downstream direction. After a thorough investigation (i.e., laboratory tests and in situ investigations), the Alcali Aggregate Reaction (AAR) expansion phenomenon was recognized to be the cause of this drift.

Due to the non-straightness of the crown of the dam, the problem was particularly complex to establish BW parameters. Hence, for the sake of simplicity, in the benchmark the effects of AAR were only assessed for the main hollow gravity block for the evaluation of the stability against sliding. The provided geometry consisted of the block and a portion of the rock. The dam-foundation interface was included as well.

The aim of the proposed Theme [ICOLD TCA, 2005] was the evaluation of AAR effects on the operational and ultimate stability of the main block of the dam. Thus, the results of two loading paths had to be compared, with and without the AAR expansion:

- Dead weight + hydrostatic and uplift pressure.
- Dead weight + AAR expansion + hydrostatic and uplift pressure.

In both cases, two water levels were considered: the operational (630 m a.s.l.) and the ultimate reservoir elevation, which had to be found by participants (Figure 11, right). The presence of the drainage system was not considered.

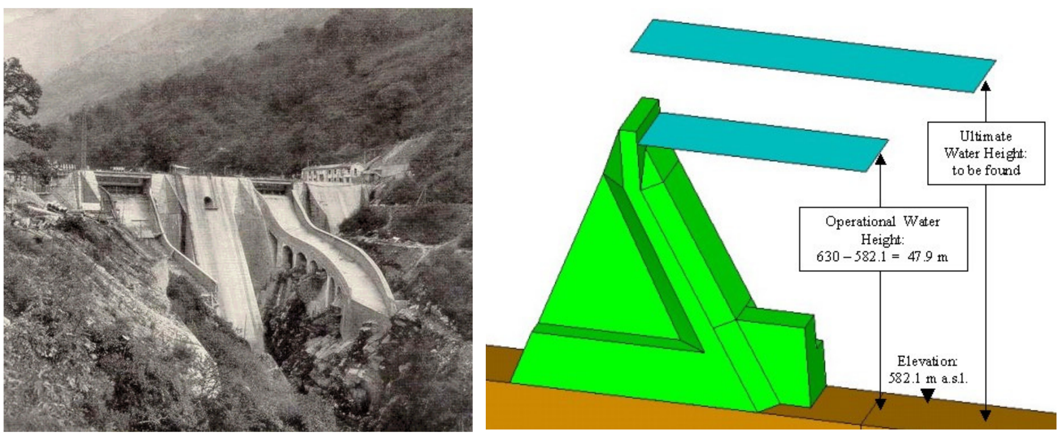

Figure 11 – Downstream view of Poglia Dam (left) and geometry of the main block and water heights to be considered (right).

The total vertical drift displacement at the top of the main block was provided to allow the calibration of the AAR expansion phenomena.

In order to evaluate the AAR effects on the global behavior of the dam, the results related to the analyses with and without the AAR expansion had to be compared in terms of curves evaluated considering the water height vs the horizontal displacement at the top of the block.

The limited number of participants did not allow to carry out an extensive comparison of results obtained through different methodologies or models. Anyway, some interesting comparisons relative to the application of two computer codes (the general purpose ABAQUS and the in-house CANT-SD) have been possible. Figure 12 shows the comparison of results obtained with the two codes considering or neglecting the effects of AAR.

The different computed behavior is due to the different characteristics of the joint model adopted by CANT-SD and ABAQUS. In fact, the ABAQUS joint model did not consider cohesion, while CANT-SD did.

Some general considerations can be drawn:

- self-balanced actions such as AAR does not seem to influence sliding limit equilibrium condition provided that the stresses in the dam body do not give rise to the formation of (local or global) mechanisms caused by damages in the dam body;

– with reference to the statement described just above, different limit states relevant to the concrete strength capacity (tensile strength, in general) require the use of suitable models capable to keeping into account smeared or discrete crack formation and propagation or, at least, of damage models;
– the complexity level of the models used to carry out numerical analyses has to be adequate to the data completeness and quality. This last statement, in spite of its apparent obviousness, is sometime underestimated by numerical analysts.

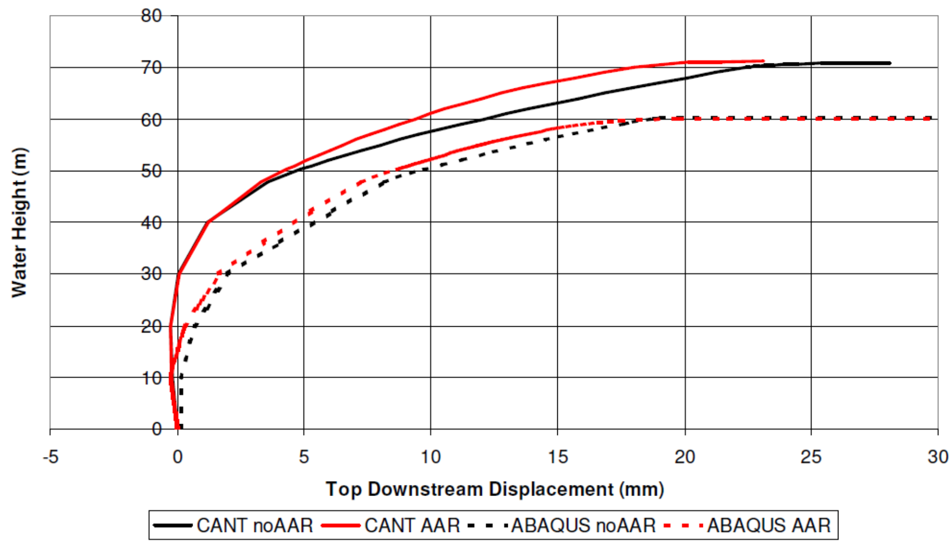

Figure 12 - Comparison of displacements at the dam crest computed with CANT-SD and ABAQUS.

3.4 Innovative methods to simulate crack propagation: the eXtended Finite Element Method

The eXtended Finite Element Method (XFEM) is a numerical technique especially designed for modelling crack growth without remeshing [Moës et al., 1999]. A standard displacement-based approximation is enriched by local functions in conjunction with additional degrees of freedom to model cracks. This approach allows to simulate the crack propagations without modifying the mesh: a crack can develop inside a finite element (Figure 13, left).

To facilitate the evaluation of the enrichment functions and their derivatives, in most XFEM codes the Level Set Method (LSM) is employed. According to this method, surfaces are not represented explicitly but level set functions are used instead. In general, two levels set functions are considered, Φ and Ψ (Figure 13, right).

The nodal value of the function Φ represents the distance of the node from the crack face: the value is assumed positive on one side of the crack face and negative on the opposite face. The set of points for which the function Φ is zero describes the crack surface.

The nodal value of the function Ψ represents the distance of the node from an almost-orthogonal surface passing through the crack tip: the values of function are assumed negative on the side towards the crack. The intersection of the two levels sets gives the crack front.

Making reference to the example in Figure 13, the value of the function Φ in nodes 1 and 2 is respectively equal to 0.25 and -0.25; whereas the value of function Ψ in nodes 1 and 2 is respectively -1.5 and -1.0.

To model the crack propagation, different techniques can be used such as the cohesive segment approach or the Linear Elastic Fracture Mechanics (LEFM)-based approach.

The XFEM could be usefully applied to study the crack initiation and propagation in concrete dams. In **Errore. L'origine riferimento non è stata trovata.** a first XFEM application on a concrete buttress dam is provided [Frigerio, 2020]. Buttress dams exhibit a particular crack pattern generally due to the thermo-mechanical phenomena occurring during construction while further propagation of a crack is mainly related to the ambient temperature variation.

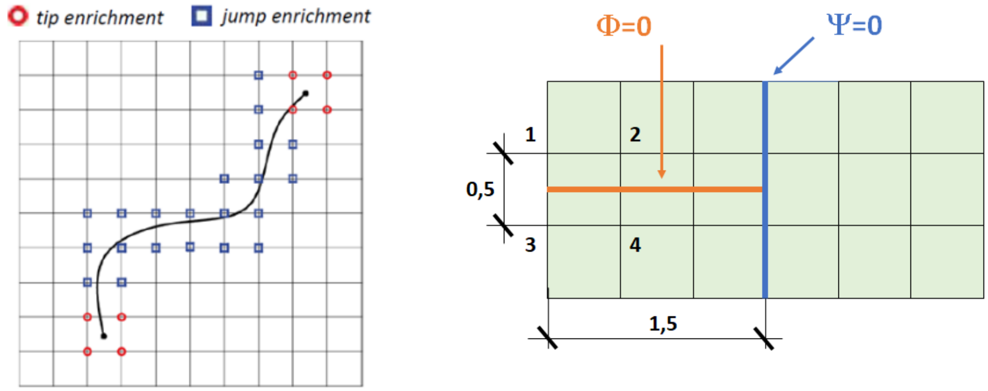

Figure 13 - An arbitrary crack on a uniform mesh (left) and the two-levels set functions generally used in XFEM to describe a crack and tracking its motion (right).

The thermo-mechanical analysis has been carried out by means of ABAQUS which provides the XFEM, simulating a three-years period of seasonal temperature variation. In **Errore. L'origine riferimento non è stata trovata.** (left picture) the geometric model of the buttress is shown, whereas the contour (right picture) is related to the STATUSXFEM parameter which describes the status of an enriched element. This parameter is equal to 1.0 if the element is completely cracked and 0.0 if the element contains no crack. If the element is partially cracked, the value of STATUSXFEM lies between 1.0 and 0.0. A crack initiation surface was inserted a priori into the geometric model along the foundation interface (**Errore. L'origine riferimento non è stata trovata.**, left), because this type of fractures takes place during construction but, in this test case, the thermal phenomena occurring during the casting sequence have not been modelled. The numerical results show how the crack propagates towards the upper part of the buttress due to temperature variation: the rate of propagation is greater in the first year and it slows down in the following ones.

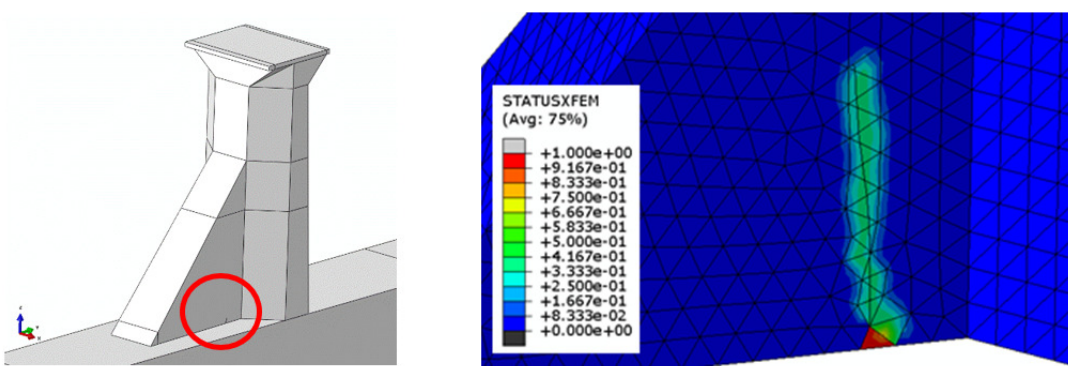

Figure 14 - XFEM applied to model the crack propagation in a concrete buttress dam: geometric model with the crack initiation surface (left) and contour of the STATUSXFEM parameter (right).

3.5 The seismic behaviour of buttress, hollow gravity, multiple arch/slab dams

Strong earthquakes can cause major damage to all types of structures. Modern FEM numerical calculation methods allow today to face seismic analysis of dams with much greater reliability than in the past, even if further research and development activities are necessary due to the (fortunately!) limited number of dams that have suffered earthquakes of strong intensity.

The dams, in fact, have shown over time to be rather resilient structures with regard to seismic actions. On the other hand, to date, there is no information on dams that have suffered an earthquake of an intensity comparable to MCE (Maximum Credible Earthquake).

In 2011 an interesting article was published [Nuss et al., 2011] in which the performance of 19 concrete dams that have suffered earthquakes of medium-high seismicity with a PGA greater than 0.3g was reported. The survey included arch, gravity and buttress dams.

Of the 19 examined dams, only 5 had sustained significant damage. In 4 of these cases the damage was repaired, and the dam returned to normal service. The fifth case refers to the great river crossing of Shih Kang (Taiwan) built on a fault that suffered so severe damage that it was put out of service.

From the investigation conducted by Nuss et al., two buttress dams were damaged by earthquakes: the Hsinfengkiang Dam (China) and Sefid Rud Dam (Iran).

In both cases the cracks that formed in the structures were horizontal and were highlighted in the upper part of the dams where there is the variation of the profile of the downstream face, as can be clearly seen in Figure 15 and Figure 16, where there is a sudden change in stiffness in the geometry of the dams.

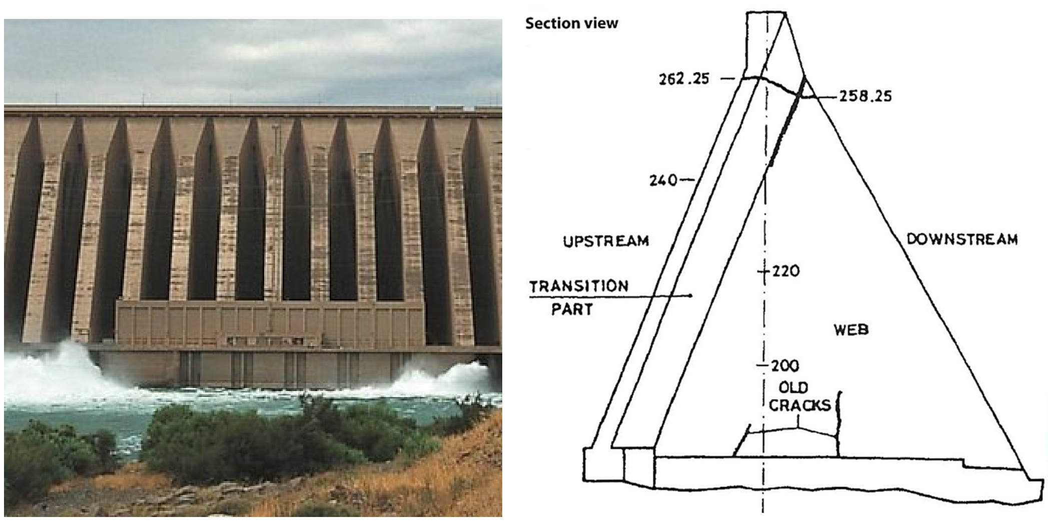

Figure 15 – The downstream view of the Sefid Rud Dam (left) and cracks after the earthquake (right).

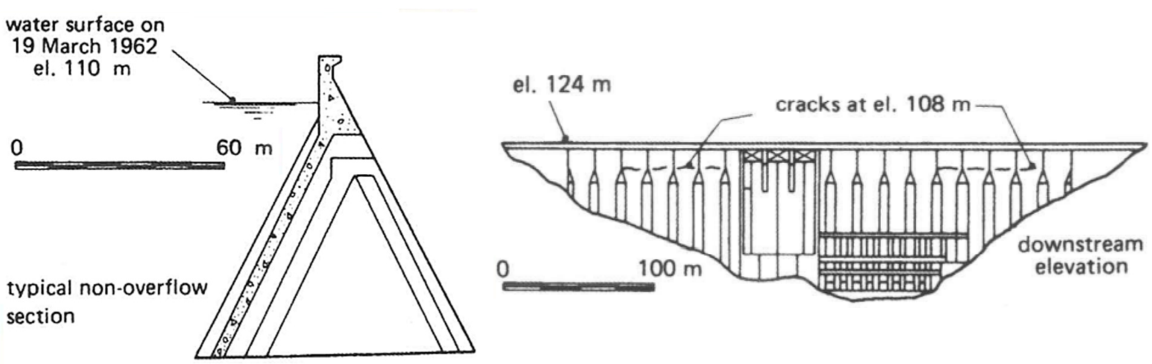

Figure 16– The Hsinfengkiang Dam after the earthquake.

In the case of the Sefid Rud Dam (and probably also in the case of the Chinese Hsinfengkiang Dam), the cracks in both faces created a substantial sub-horizontal rupture plane without associated significant sliding, probably due to the high value of the shear strength in concrete due to the roughness of the surface and the effect of the overlying weight.

A situation somewhat similar to the two described above is that relating to the well-known case of the Koyna Dam, in India (Figure 17).

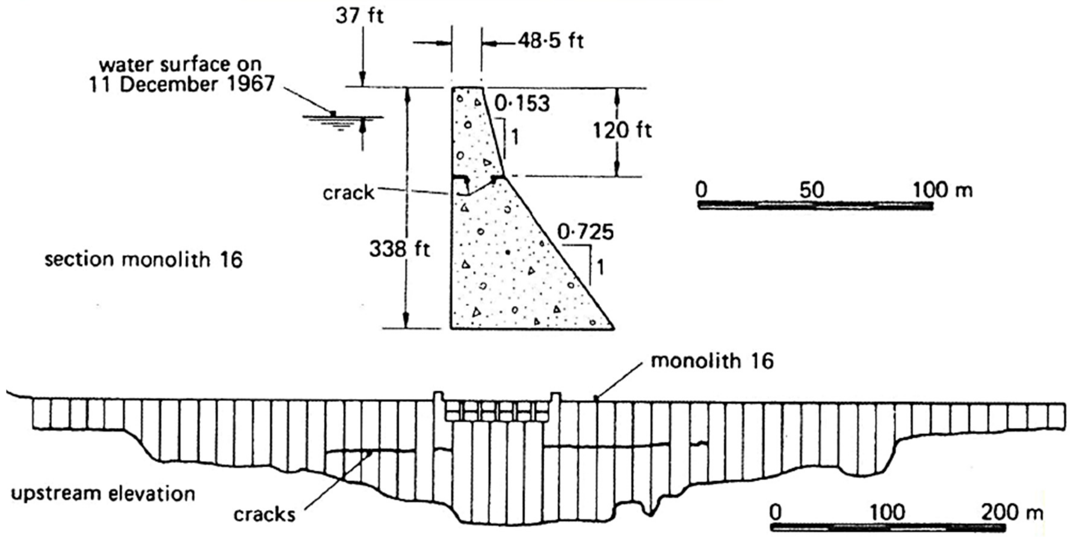

Figure 17 – The Koyna dam after the earthquake.

In this case, the cracks that appeared on the two upstream and downstream sides in the upper part of the dam fortunately did not manage to create a rupture surface. In any case, the position of the cracks caused by the earthquake is substantially similar to what has been observed in the two cases of the aforementioned buttress dams.

The article by Nuss et al. cited above briefly describes some interesting cases of buttress dams that have been the subject of reinforcement interventions to cope with the expected stresses on the basis of the seismic re-evaluation of the sites where the works are located. Here below two cases are briefly described.

The Big Bear Dam (a multiple arches structure built in 1912, 28 m high, with a 110 m crest length) was rehabilitated in 1989 for a project earthquake with a PGA of 0.71 g. The adaptation project was necessary both for fear of seismic actions (the work is located just 16 km from the San Andreas fault in the S. Bernardino mountains, 80 km east of Los Angeles) and for the need to adaptation of outlets.

The structure was reinforced with the partial filling of the compartments by means of the massive concrete casting which substantially reduced the extension of the buttresses. The intervention involved the execution of completion castings to ensure the monolithic behaviour between the old and new structure (Figure 18). It is interesting to note that in 1992, at one day from each other, there were two events estimated with magnitudes equal to 7.3 and 6.6 located respectively at 28 and 9 miles from the dam site.

The estimated PGA at the dam site was 0.35 g. The inspection carried out immediately after the earthquakes did not show any damage to the structure.

The second case refers to the Littlerock Dam, California (a multiple arch structure built in 1924, 53 m high, with a crown of 219 m), rehabilitated in 1994 for a project earthquake with a PGA of 0.70 g. The dam is located just 1.5 miles from the San Andreas Fault.

The studies conducted for the evaluation of the safety conditions of the work had highlighted its potential vulnerability with regard to transversal actions. Therefore, it was decided to reinforce the structure with the partial filling of the compartments according to the scheme illustrated in Figure 19.

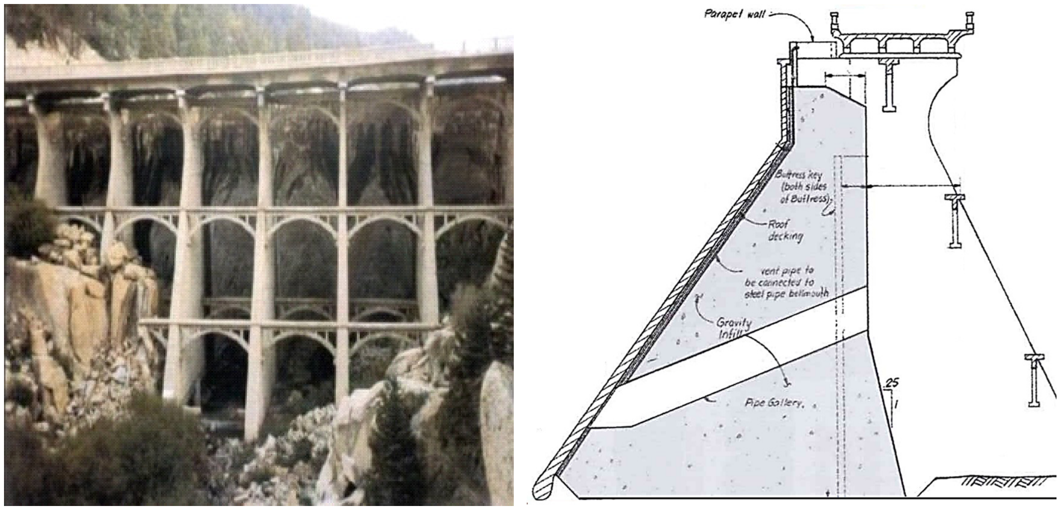

Figure 18 - The Big Bear Dam: vertical section of the structure being filled with massive concrete.

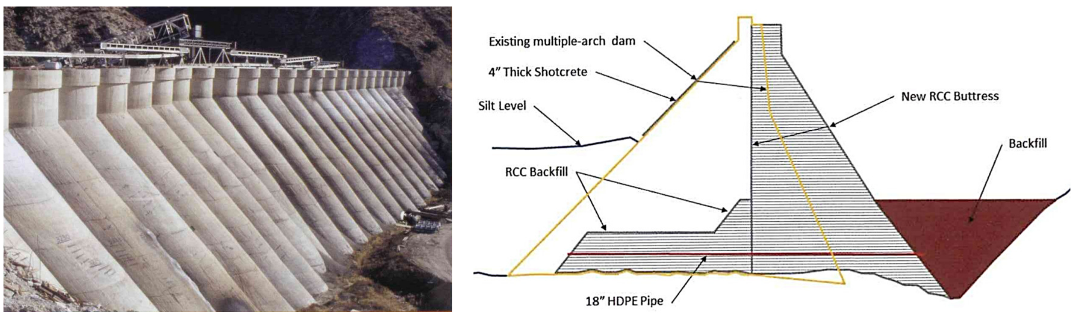

Figure 19 – The Littlerock Dam: a) the structure seen from the mountain; b) vertical section of the intervention with partial filling of the rooms with concrete (RCC).

4 CONCLUSIONS

Water is an indispensable resource for human life and its use must primarily be aimed at satisfying human needs (water supply for drinking and irrigation purposes). A company must pay attention to the rational use of this strategic resource reconciling all these needs with the objectives of industrial development, energy production and environmental protection that each Country has to foster within the framework of a common European policy.

However, in Italy the construction of new dams is experiencing a very unfavourable phase both at the political level and in the public opinion due to numerous factors:

- Financial constraints (due to the persistent critical phase of public finance and the low propensity of the private one to long-term investments).
- Intense anthropization of the Italian territory.
- Reduction of favourable sites from a geo-morphological point of view.
- Administrative and legislative constraints (e.g., regional constraints on the Minimum Vital Outflow).
- Competition between the "multiple uses" of the water resource.
- Widespread hostility from pressure groups.

The managers of hydro and hydroelectric plants therefore have to face complex problems related to the safety of infrastructures that in some cases have also exceeded what is considered their design service life.

Dams are the product of the application of successive design and construction criteria. A further aspect to take into account when considering dam safety concerns the fact that the safety levels of the structures are inevitably not homogeneous due to the evolution of the construction technologies, the investigation techniques, the calculation methods that took place over almost a century.

The dams covered by this article are fully part of this context and, taking into account their average life, in several cases close to a century, they require continuous surveillance and, in some cases, maintenance interventions (ordinary and, sometimes, extraordinary).

To this end, the modern calculation methods available today are able to offer operators responsible for dam management the necessary elements for a reliable assessment of structural safety.

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