

Malpasset arch dam: an example of major accident caused by the effect of water in a rock foundation

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Case Study

Keywords

Arch dam
Accident
Hydraulic uplift
Natural fractures
Rock foundation

Abstract

Malpasset dam accident is considered as the classical example of the detrimental effects of water in a rock foundation. Technically, it represents a unique case of arch dam collapse induced by the water and the subsequent uplift pressures acting on a very large rock wedge in a fractured rock foundation. At the time of the accident (1950's), such an effect was totally unknown by the dam designers and geologists as well. After a summary of the background and geological context, the investigations and the design (initial and final one) are presented. Then, the chronology of events is reconstructed in detail, including the structural deformation of the arch during the filling of the reservoir and the heavy rainfalls. After all, the collapse is analysed based upon the observation carried out immediately after the accident as well as the post failure investigations that took place during the following years. Despite the total failure of the arch dam, the origin of the accident cannot be attributed to the dam structure itself but solely to the rock foundation. The accident of Malpasset led to the creation of the International Society for Rock Mechanics (ISRM) in 1962 as well as main developments in engineering geology, hydrogeology, soil and rock mechanics.

1. Introduction

Dams and tunnels have served the human being since the earliest times of civilization. It is not registered exactly when irrigations systems and dams started to be used, but the estimative shows for at least for 5000 years, as evidenced by the ruins of Babylonia, Egypt, India, Persia and Far East. Studies of the ancient China, India, Iran and Egypt reveals that their use started thousands of years ago. Menes, the first Pharaoh of Egypt (3150 B.C.), ordered irrigations works for harnessing the waters of the Nile River. The holy scriptures of India mention very early operation of wells, dams and channels and that the peoples of ancient Persians also recognized the importance of irrigations to the sustenance of civilization (Jansen, 1983).

It seems that the first tunnelling was done by prehistoric people seeking to enlarge their caves. Registers show that all major ancient civilizations developed tunnelling methods mainly for irrigation. A pedestrian passage some 900 m long, for example, was built in sandstone (2180-2160 B.C.) under the Euphrates River to connect the royal palace with the temple. The Egyptians developed techniques for cutting soft rocks using abrasive materials. To save the need for lining,

most ancient tunnels were made in strong rock, spalled by a method involving heating the rock with fire and suddenly cooling. Both the Greeks and Romans made extensive use of tunnels to reclaim marshes by drainage and water aqueducts (Lane & Garfield, 1972).

In the course of the History the presence of the groundwater was always one unfavourable factor in dam engineering and tunnelling, due to the flows and pressures able to affect significantly the stability of the excavations in tunnels and the foundations and abutments of dams. Unfortunately, natural hazards and human mistakes associated to the deficiency in knowledge, caused sometimes great tragedies to occur over the centuries with this type of works. Malpasset dam failure is probably the most significant accident occurred that occurred in dam engineering in the 20th century.

2. Background

2.1 Malpasset project

The arch dam of Malpasset was located at the Var Department, Southeast of France, close to the French Riviera,

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near Fréjus, an old Roman city at the Mediterranean Sea which, 2000 years before was chosen to settle part of the fleet taken from Marcus Antonius in the battle of Actium. A number of buildings, ramparts and harbour installations were built there at that time. In the Middle Ages, Fréjus was just a small village. The Aqueduct of Fréjus was built in the middle of the 1st century. It was a 42-km long aqueduct, with water intake at the Neïssoun springs (EL. 481; elevation in metres above sea level) and distribution in the town of Fréjus (at sea level).

In 1874, a concession of 26000 m³/d (300 l/s) was made, using the Roman aqueduct for irrigation of fields in Caillan and Montauroux. Twenty years later the system was partly rebuilt and extended to supply Fréjus. After the 1st World War, new complementary channels were built to supply Fréjus, St Raphael and other towns. A sad chapter in the history of Fréjus and the aqueduct started with the construction of the Malpasset dam in the Reyran valley in 1950. In 1959, when excessive rains filled the reservoir for the first time, the dam collapsed. The village was destroyed and the Roman aqueduct partially destroyed. The fact that the Roman engineers buttressed and cross-braced the pillars of the Aqueduct among other reinforcements, could have been a warning for modern engineers about the general complexity of the geology, with variable and fractured rock types and faults, treacherous for a reservoir.

Malpasset represents a classical example of the disastrous effect of the action of water on foundations and structures of dams. A huge number of scientific questions were raised related to the unknown mechanical and hydraulic behaviour of the rock masses after the accident. The rock masses properties were very little known at that time, particularly when submitted to the stresses of a dam. The collapse led to the development of many researches and theoretical analyses in an attempt to understand the causes and mechanisms involved with the rupture and brought a significant change in the state-of-art for the geotechnical engineering. A large amount of literature was published with discussions on investigation, design, construction and operation of the dam after the accident. Circumstantially, due to the recognition that it was necessary to deepen the knowledge of underground rocky materials under engineering structures, three years after the collapse the International Society for Rock Mechanics and Rock Engineering (ISRM) was founded in Salzburg by Prof. Leopold Müller (<https://isrm.net>).

The factors that induced the rupture are presented here based on publications, lectures, conferences and in some personal communications with Pierre Duffaut who, at the time of the accident, was working for the French Electric Board

(Electricité de France or EDF). Duffaut used to emphasize the relevance of the geological investigations for the analysis and interpretation of the hydro-mechanical behaviour of dam foundations and abutments considering the interaction of the structure with the surrounding geological materials.

2.2 Investigation and design

The site selection and investigations for the dam construction took place from 1946 to 1951. An initial site has been chosen considering two main reasons: i) the topography of the valley, based on the concept of a narrow section where a gravity dam could be conveniently sited; ii) the geology, offering good rock conditions and a specific tectonic structure, a horst allowing an older and deeper gneiss to outcrop at the ground surface, surrounded by more recent sedimentary environment. For the reservoir, the Carboniferous sedimentary rocks, forming a syncline with wider valley sections, were considered offering the required tightness. For the dam, according to the description given in two reports written by Prof. G. Corroy, from the Marseille University, responsible for the geological studies, the gneiss was considered to provide a fair rock type for a dam foundation despite being heterogeneous with alternation of felsic and mafic layers as well as massive basement with schistose texture, crushed by numerous shear fractures and faults (Goguel, 2010).

The investigations had been performed for the construction of a gravity dam. According to Bellier (1967), Roubault (1970) and Londe (1965), the programme of investigations has been well performed. However, this initial site was shifted 200 m downstream to a new site selected by Eng. André Coyne for an arch dam. Yet, Prof. Corroy did not disagree with this major modification (Valentini & Bertini, 2003). The arch dam was then designed by André Coyne, a world-famous expert in arch dam engineering, at the head of ACJB from 1947 to 1960 (Bordes & Tardieu, 2015). Coyne supervised the construction works made by the main contractor Léon Ballot (Duffaut, 2013a). Due to the small distance between the two dam sites, the geology looking very similar (bedrock and alluvium) and the banks covered by a quasi-impenetrable vegetation, no additional investigation was conducted for the new site (Boudou, 2015) and the design was simply adapted to the arch dam selected.

The technical data for the arch structure and the reservoir is presented in Table 1, following the French Committee for Dams and Reservoirs (CFBR, 2009) and the website of the city of Fréjus (Ville de Fréjus, 2016). Figures 1 and 2 show

Table 1. Technical data used in the design of the Malpasset dam.

Reyran river, Var Department, France (between Cannes & St Tropez)		
Geography	Distance to sea	12.5 km
	Elevation (above sea level)	River bed 42 m
Geology		Dam crest 102.50 m
	Rock type	Gneiss

Source: Comité Français des Barrages et Reservoirs (CFBR, 2009) and city of Fréjus website (Ville de Fréjus, 2016).

Table 1. Continued...

Reyran river, Var Department, France (between Cannes & St Tropez)		
Contractual aspects	Owner	General Council of Var Department
	Designer	ACJB Bureau founded by André Coyne & Jean Bellier (1947)
Dam	Construction contractors	Ballot Bros.
	Type	Bachy
	Radius	Arch dam, thin double curvature type
	Height	Horizontal 105 m
	Thickness	66.50 m above foundation
	Length at crest	Crest 1.5 m; base 7 m
	Volume of concrete	222 m at 102.50 elevation
	Spillway	48.000 m ³
	Outlet valve	Central, on dam crest, 40 m long (elevation 100.40); discharge capacity 178m ³ /s
	Height	1.5 m conduit dimensioned for 40 m ³ /s
Reservoir		Operating elevation 98.50 m
	Capacity	Max elevation (spillway) 100.40 m
		51.4x106 m ³ at maximum water level

Source: Comité Français des Barrages et Reservoirs (CFBR, 2009) and city of Fréjus website (Ville de Fréjus, 2016).

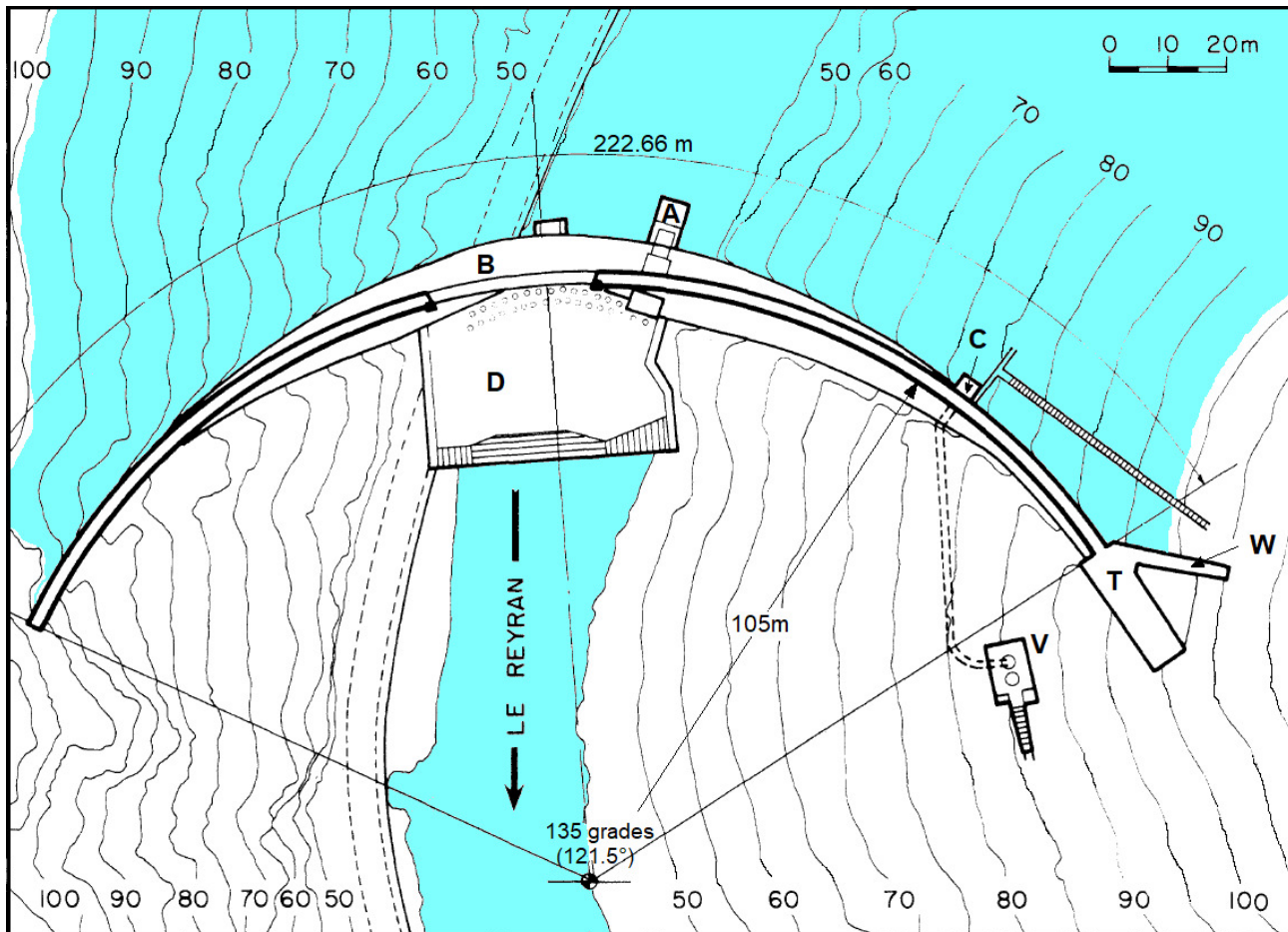


Figure 1. Location of the arch dam on the valley slope. Contour lines in m. (A) Bottom valve gate; (B) Surface overflow weir control; (C) Water intake; (D) stilling basing. V = control valve. In the left abutment: a gravity thrust block (T) counteracts the crest arch thrust and is protected from the water thrust by a wing wall (W) perpendicular to the contour lines. Modified from Bellier (1967) and Duffaut (2013a).

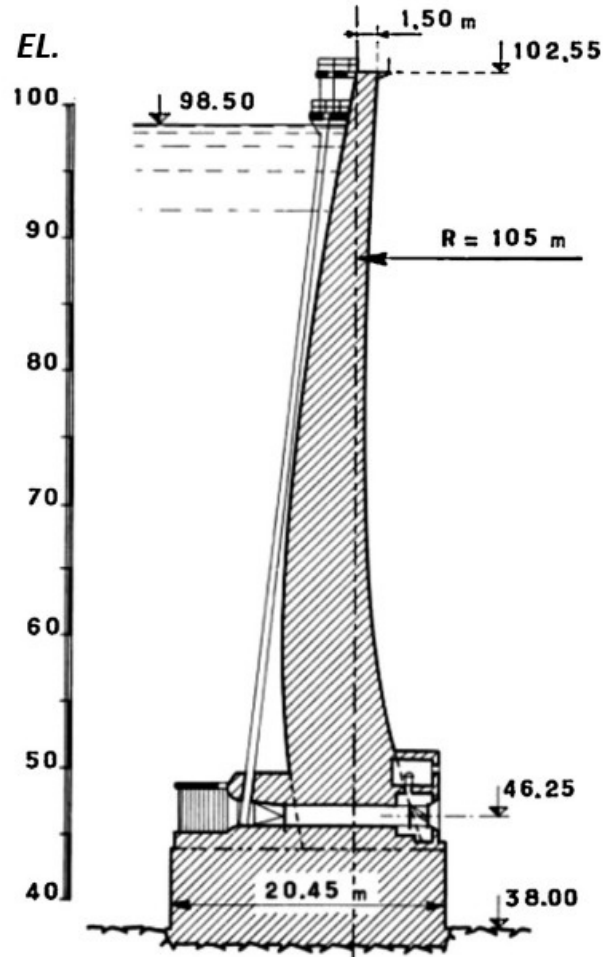


Figure 2. Highest dam cross section. The support of the hollow valve and its control gate explains the widened foundation at this place. Crest elevation: 102,55 m; crest length 223.0 m; spillway elevation: 104.40 m; normal operation level: 98.50 m; bedrock level: 38.00 m (Duffaut, 2013a).

the contour map of the site and the cross section of the dam (Duffaut, 2013a).

The dam was designed as a pure double curvature arch, following the practice in the fifties. The concrete arch structure had a 105.0 m axis radius and a central angle of 135 grades (121.5 degrees) between the two rock abutments on the Reyran river (Figure 1). The maximum thickness of the structure was 7.0 m on the base and 1.5 m on the crest (Figure 2). The right abutment consisted of a massive rock mass. On the left bank, a wingwall was needed due to the site topography of the site (the left bank being smoother than the right steep bank).

As reported by Londe (1965) the rock mass deformability was assumed to be ten times greater than that of the concrete ($E_{\text{concrete}}/E_{\text{rock}} = 10$). For the reservoir at EL. 102 m, the maximum compressive stress in the concrete was 6.9 MPa and the maximum tension -1.5 MPa. The average concrete compressive strength (90 days) measured during the construction

was 32.5 MPa. The average stress at the foundation surface was 2.5 MPa and the maximum local value was 5.0 MPa. For the same rock mass deformability, the thrust of the arch on the left bank abutment was 4800 t providing a high factor of safety against sliding. Figure 3 shows the principal stresses, computed from the trial-load method assuming a simplified symmetrical arch, on the upstream and downstream face of the full reservoir, as usual at that time.

The geological studies concentrated more on the area of the reservoir and on its tightness, than on the area of the concrete arch foundation. A summary of the investigations and the geology of the site were presented by Londe (1965), Goguel, (2010), Duffaut (2009, 2013a, b) and others. The overburden had low thickness, around 4m depth, according to the boreholes made in the central part of the valley. Lugeon tests have been performed in the area of the foundation. Along the sides of the valley only visual geological observations were carried out. The foundation was described as a synclinal

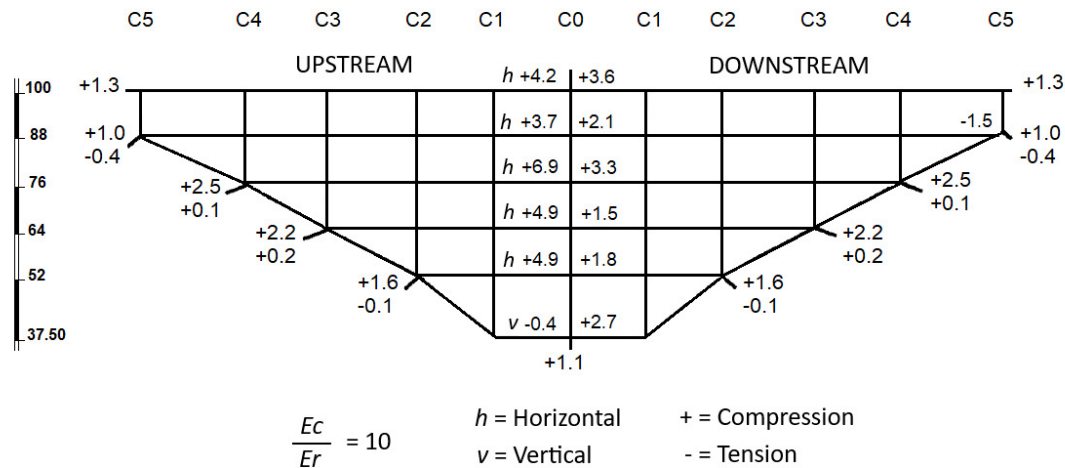


Figure 3. Stresses calculated according to the trial-load method. The calculations considered a rock mass ten times more deformable than the concrete and a maximum elevation of 100 m for the reservoir (data from Londe, 1965).

carboniferous zone enclosed by the metamorphic horizons of the Massif of Esterel basement.

The contact grouting (one 5-m long hole for 10 to 15 m²) took an average of 30 kg of cement per metre of hole. A deep grout curtain was not considered necessary, as the Lugeon tests showed a very low permeability of the rock below 5 m of depth, based on data from 8 boreholes performed with 20 to 35 m depth. Absorptions corresponded only to 2 Lugeon units and many of them to a fraction of Lugeon, except in one stage of 12 Lugeon units. No drainage had been provided in the dam or in the foundation as this was not the practice in the 1950's. Even in gravity dams, drainage was in general shallow, to treat only the concrete-rock contacts. No instrumentation was installed in the dam, although vibrating wires extensometers to be embedded in the concrete, were planned by the designer (Londe, 1965).

During the 1950's, mostly of the rock masses characteristics were not considered in the design of dam structures. In general, technical reports used to refer only to the "bedrock" (in this case the gneiss), excluding some significant characteristics of the rock masses such as joints, faults, shear zones, intrusion dykes and weathering. After the Malpasset collapse, the design started to integrate the concrete structure of the dams with the properties and behaviour of the rock masses, following the approach developed by Wittke & Leonards (1987) during the analysis of the collapse. A photo of the dam at the end of the construction is shown in Figure 4 (Duffaut, 2011).

At the end of the construction, the dam had the record of the thinnest arch dam with a maximum thickness of 6.78 meters at the bottom and 1.50 meters at the top. In the centre of the crest, it had one un-gated spillway and in the left abutment a thrust block was built below the concrete arch to raise it to the necessary height (see Figure 1).

3. Chronology of the accident

3.1 Filling of the reservoir

As reported by Jansen (1983) and Londe (1965) the filling of the reservoir started very slow on April 20, 1954, after the closure of the outlet valve at the base. The dam had a relatively uneventful life until the fall of 1959. After five years, the filling was not complete, when heavy rains occurred in the region. The graph presented in Figure 5 shows the readings of the level of the reservoir from 1955 to 1959. The arrows indicate when the readings were done. The insert in the figure shows the very fast 5-m rise of the reservoir level in just 3 days. The deformations of the concrete blocks during the filling of the reservoir were monitored only by geodetic measurements installed on 28 targets fixed on the downstream face. The first readings were taken in September 1955, when the level was at EL. 79.75 m and the last readings were performed in July 1959, when the level was at EL. 94.10 m.

In the middle of November, the reservoir was at EL. 95.20 m, 5.20 m below the maximum level. Some few weeks before cracks have been also observed in a small area in the concrete adjacent to the toe of the dam. At that time, the operator discovered a trickle of clear water at the right abutment about 20 m downstream, at EL. 80 m. On November 28th the water level reached EL. 95.75 m. Due to heavy rains, the level increased to EL. 97 m on November 30th. It was also noted that the seepage had increased. On December 2nd at noon, the level was at EL. 100 m. At 18h00 the outlet valve was opened and a very small lowering was registered an hour and a half later.



Figure 4. Photo of Malpasset dam at the end of the construction (photo from Coyne and Bellier, in Duffaut, 2011). The construction started in April 1952 and was completed two years later, in October 1954, say 5 years before the accident.

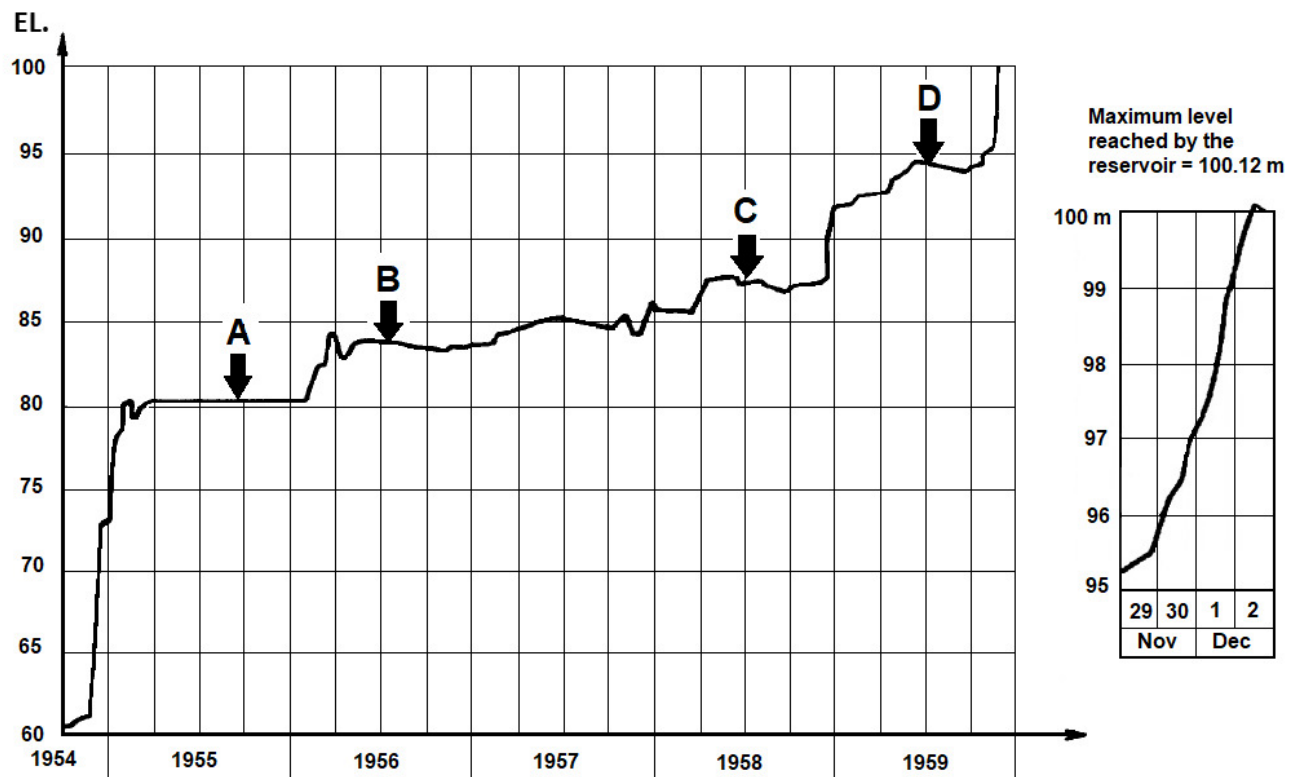


Figure 5. Graph of the reservoir filling, showing position and dates of the four geodesical measurements (A to D). Readings were registered along 1954-1959. The insert in the right shows the very fast 5 m rising of the reservoir level in just 3 days, before the dam failure. The opening of the bottom gate 3 hours before the catastrophe, allowed a lowering of only 2 cm on the water level (Mary, 1968; Duffaut, 2013b).

Figures 6 to 8 show in plan and vertical cross-sections the register of displacements along the blocks of the arch for the total rise of the reservoir around 14 m, from September 1955 to July 1959 (Mary, 1968, in Londe, 1965). The scale in the figures applies to the target displacements. The letters in bold refer to the measurement dates (see Figure 5 for reference). The other letters refer to the construction joints. Q is in the left abutment bank block. Segments CD presents the deformations between July 1958-1959 showing a general move towards the left bank. A careful analysis of these results would have evidenced a change in the behaviour of the structure before the latest 5-m rise showed in the insert of Figure 5.

Figure 8 shows the monitored and calculated deformations during the operation of the dam, computed for a foundation modulus 10 times lower than the concrete modulus. Results are in good agreement in general, except at the base of the dam where the rock yielded much more than the calculated results.

Figure 9 shows the longitudinal section of the dam with the joint displacements between the years 1955 and 1959. A general tilting of the arch structure for the left is observed - the right bank being lifted when compared to the left bank, particularly in the vicinity of the structural joint F. Figure 10 shows the downstream elevation after the

accident. The water rose some 20 metres following what could be expected from the sudden collapse, based on the height of the dam and drop of the reservoir.

Attention was called by Londe for the total radial displacement related to joint F. Instead of 17.2 mm, the correct displacement is 13 mm making the behaviour of joint F similar to the other joints (Londe, 1965).

3.2 The collapse

As reported after the accident (Habib, 1987; Mary, 1968; Londe, 1965; Duffaut, 2013a, b) in the afternoon of the 2nd of December, engineers visited the site to decide how to avoid detrimental effects of spilling due to the heavy rains of some days, particularly on a downstream highway bridge, under construction. They did not notice anything abnormal in the dam and decided to open the bottom gate to control the rising of the reservoir. This was done at 18h00 and no vibrations were noticed. The caretaker left the dam at 20h45 and went back to his home on the hillside, about 1.6 km downstream. At about 21h10 he heard a loud cracking noise and soon after, a violent blast blew open doors and windows. The failure occurred a few minutes later.

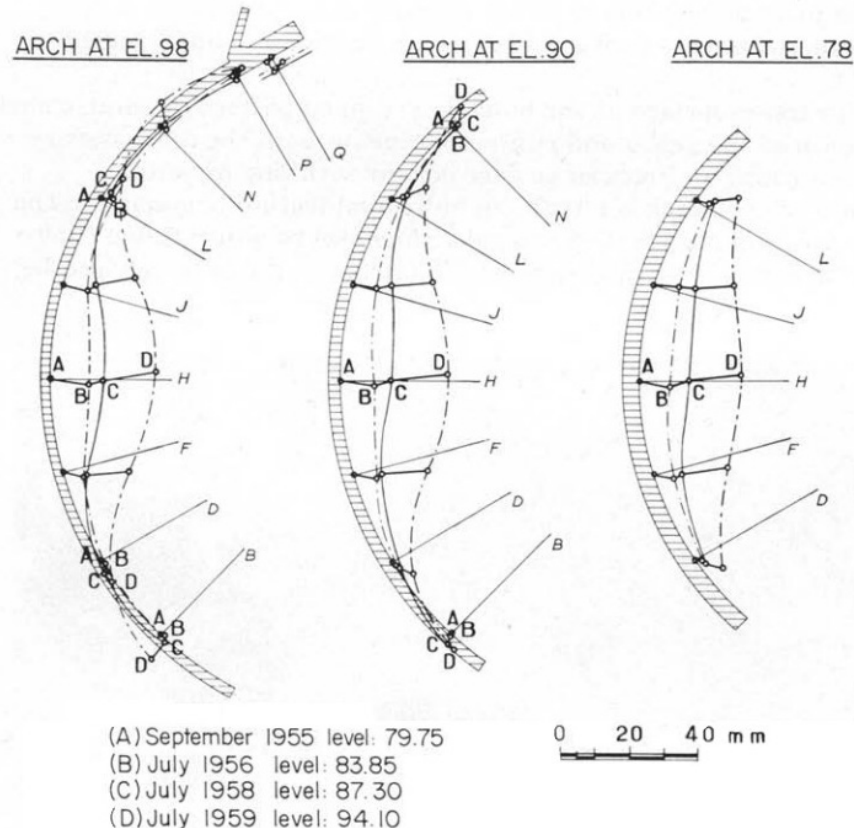


Figure 6. Displacements of the targets along the arches for El. 98, 90 and 78 before the rupture: plan view. Segments CD shows deformations occurred between July 1958 and 1959 (Londe, 1965).

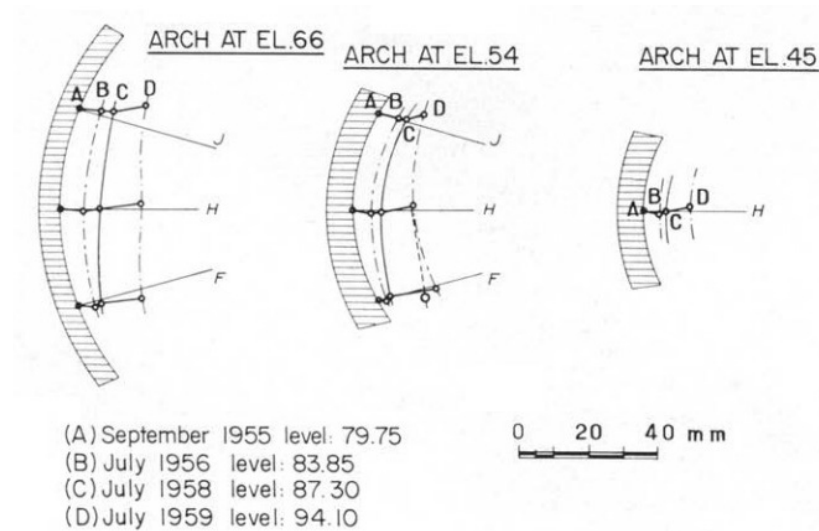


Figure 7. Displacements of the targets along the arches for El. 66, 54 and 45 before the rupture: plan view. Segments CD shows deformations between July 1958 and 1959 (Londe, 1965).

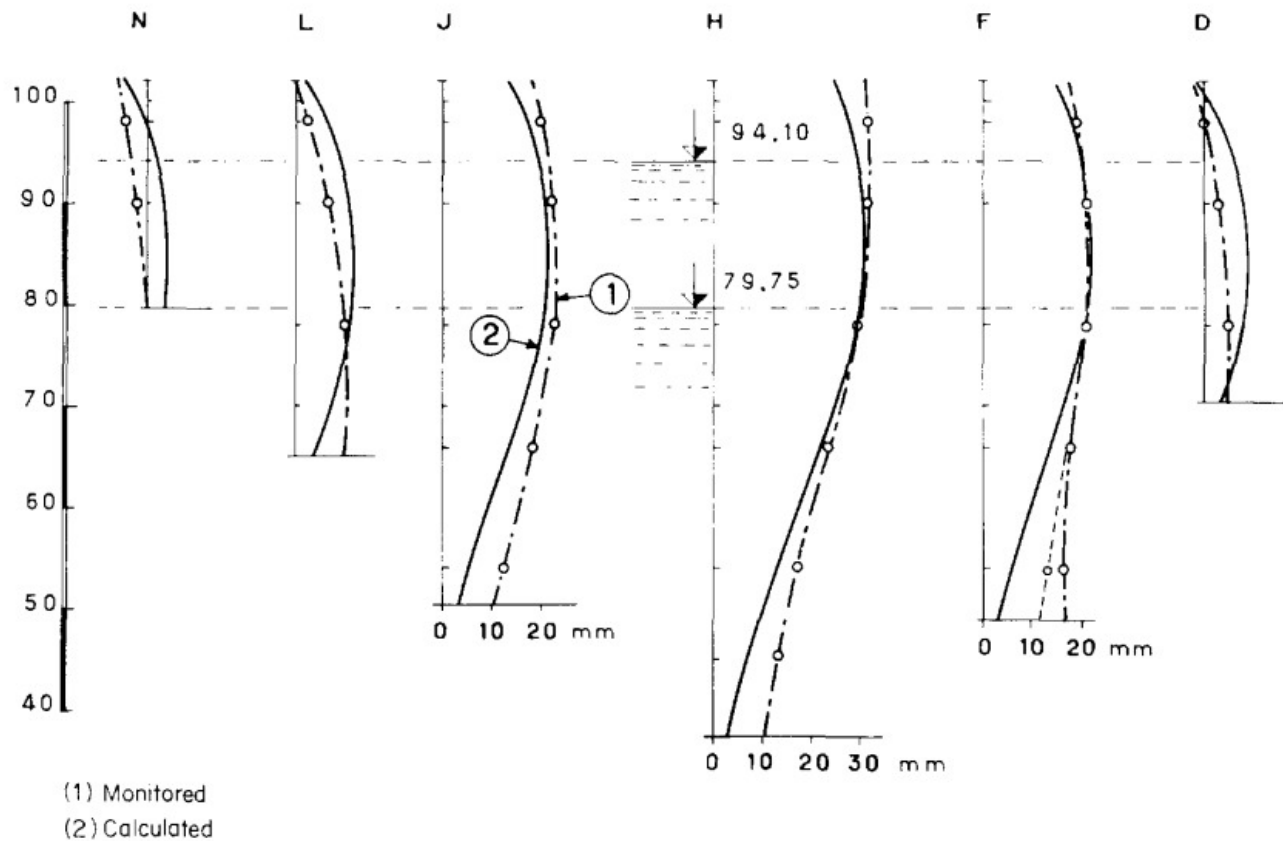


Figure 8. Comparison between monitored and calculated deformations during the operation of the dam (Londe, 1965).

The very few witnesses to the failure (located at the bridge site) reported that first occurred a violent trembling of the ground; then a brief rumble very loud followed by a strong

blast of air and finally the arrival of the water in two phases starting with a wave that overflowed the banks following the valley and taking the temporary support of the highway bridge

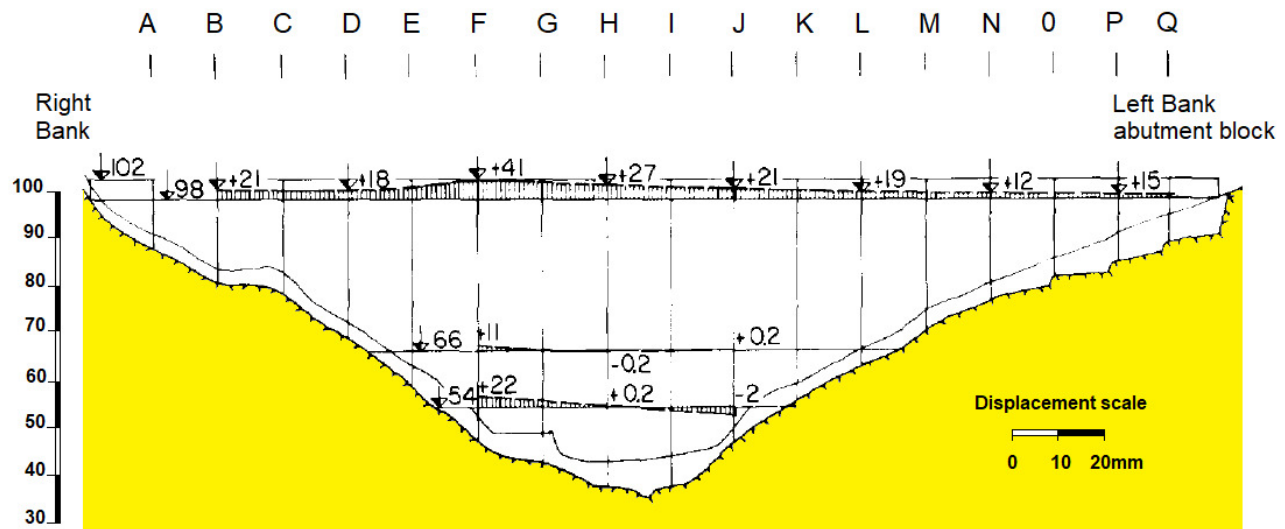


Figure 9. Longitudinal section of Malpasset dam before the collapse, showing the rock top on the excavations, the vertical displacements between the years 1955 and 1959 and a general tilting of the arch structure to the left. Modified from Londe (1965).

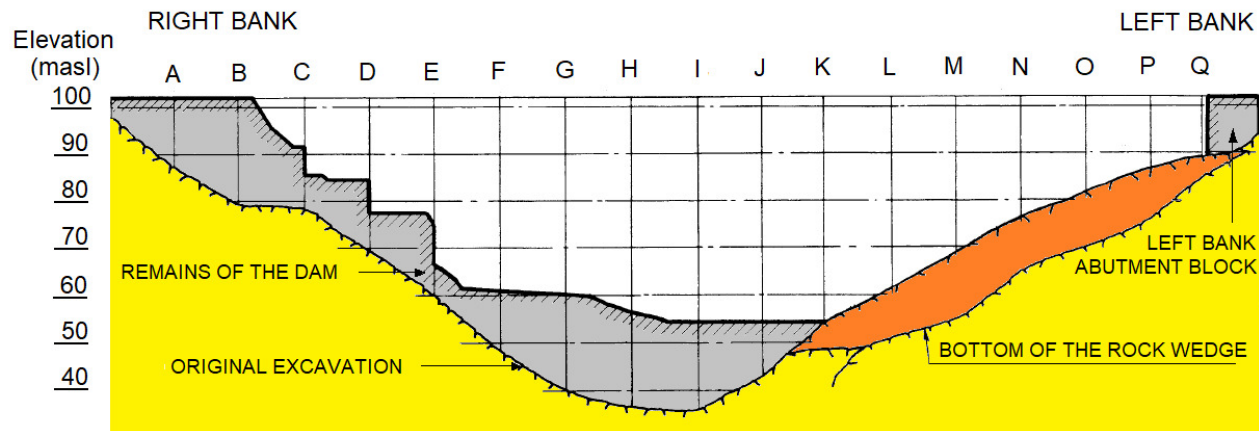


Figure 10. Longitudinal section of the Malpasset dam after the collapse. On the right bank, only the base of the concrete arch was still there. On the left bank a huge wedge of rock was ejected carrying away the concrete. Only remains the wing wall on the top, part of the concrete in the abutment and the thrust block, which had been separated from the wing wall and slid downward as showed in Figure 12. Modified from Bellier (1967).

which was still illuminated. Then a huge wall of water surged from the gorge and crashed over the top of the spur on the left bank. At this moment the electric power was cut. The power line of the dam was reported to have broken at 21h13. On the following morning only the base of the concrete arch in the right bank was still there, together with some remain of the thrust block in the left bank (Figures 11 and 12).

The elements of the concrete arch were cracked and the structural joints were displaced as shown in Figure 13. Some of the structural blocks were found after the accident as shown in the photo of Figure 14.

A large volume of the left rock abutment downstream disappeared and the exposed rock formation extensively

cracked was displayed, making it easier for retrospective geological studies. An impressive demonstration of the dam's movement during the failure was the large gap opened between the upstream face and the right abutment rock as seen in Figures 15 and 16. Investigators were able to enter the gap to a depth of about 8 metres (Londe, 1965).

4. Analysis of the accident

4.1 Post failure investigations

Following the collapse, studies and investigations (including adits on both sides and in-situ and laboratory



Figure 11. Photo of the dam after the accident on 2nd December, 1959 (Mary, 1968 in Duffaut, 2013a).

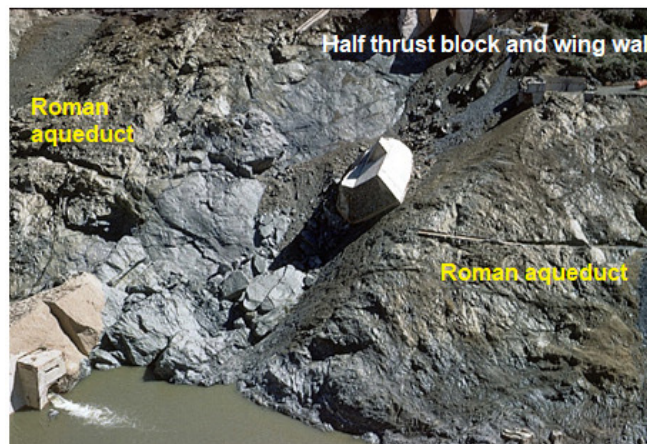


Figure 12. Left bank after the failure - Photograph showing the exposed rock formation extensively cracked and some remains of the concrete arch: half of the thrust block and wing wall (T and W in Figure 1) and the bottom gate in the lower left. The tracing of the Roman aqueduct still appears interrupted by the empty space left by the ejected rock wedge (photo taken by Duffaut in May 1960, showed in Duffaut, 2013a).

tests) were made, some of them on behalf of the designer André Coyne, before he passed away in 1960, others for EDF. An official Inquiry Commission appointed by the French Government, was formed to study in details the rupture and origins of the accident (Goguel, 2010). Many of these studies started to be made immediately after the accident. However, during the following decades and until recently, many academic works like doctoral theses and distinct types of publications have been produced on this tragedy, as for example: Habib (2001), Bordes (2010), Duffaut (2011, 2013a), Boudou (2015), Shaffner (2015), Carrère (2010) and many others.

Details of the geology of the site were reported by the known French geologist Jean Goguel in 1960 and reproduced in 2010. He was in charge by the Inquiry Commission of writing on the geological and mineralogical studies and investigations made before the construction, considering particularly the foundations and abutments of the dam. Investigations showed that the geological and hydrogeological conditions in the site were in the origin of the catastrophe. Also, that the heavy rains occurred in the region with the sudden elevation of the water in the reservoir, contributed to the increasing of pressures in the foundation and to the catastrophe. Prof. M. Roubault, Director of the Engineering Geological School of

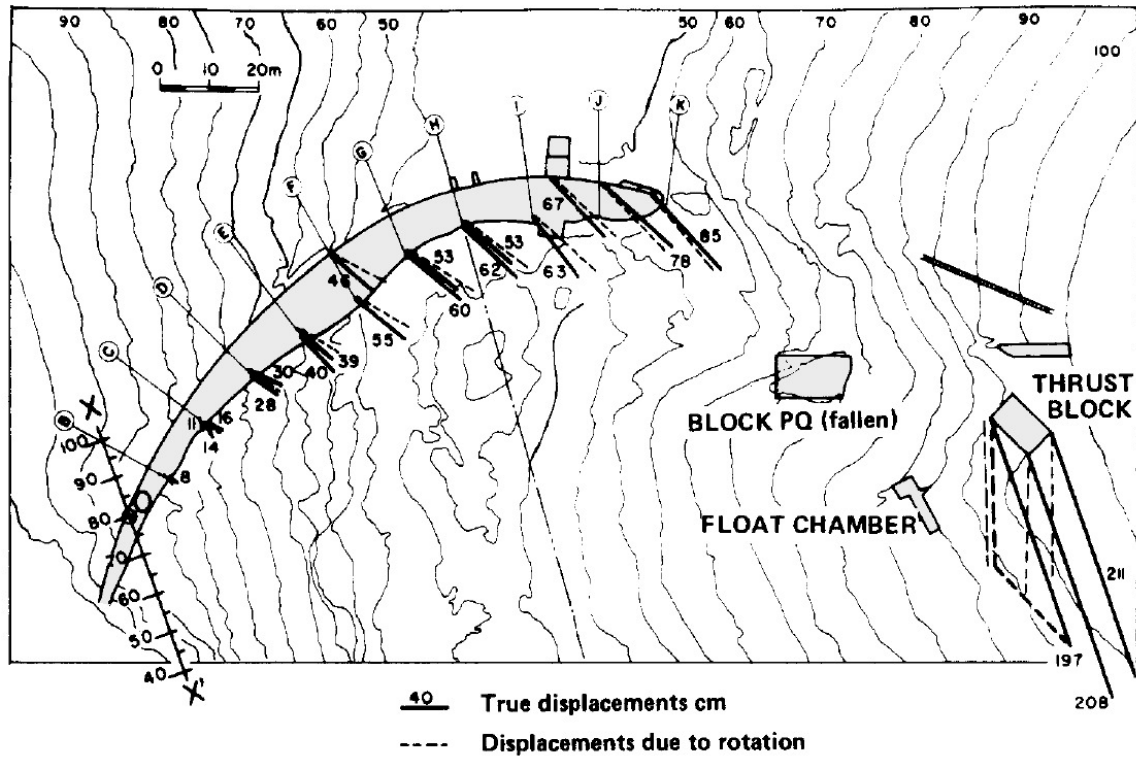


Figure 13. Displacement of the concrete arch joints. Modified from Londe (1965).



Figure 14. Damaged dam structure after the accident. Courtesy of Pierre Duffaut; Photo taken in 2010.



Figure 15. A large gap (crevice) displayed after the accident between the upstream face and the right abutment of the dam. Photo taken in May 1960. Courtesy of Pierre Duffaut.



Figure 16. The same crevice on a photo taken by Duffaut in 2012. Courtesy of Pierre Duffaut.

Nancy, member of the Inquiry Commission, remarked the lack of investigations of the dam foundations as: “with faults cutting the rock mass, water reaching a certain elevation in the reservoir and pressures acting inside the foundation, the die was cast for the inevitable catastrophe” (Roubault, 1970).

Several geological studies made after the collapse, unveiled that the dam was built on a foundation formed by gneiss, with localised lenses of pegmatite. Three structural fracture sets have been evidenced by Goguel (2010):

- the gneiss foliation with a low spacing and a SW dip direction; presence of clay minerals (sericite) in the mafic bands;
- planar joints with a very low spacing, probably because of the shallow depth;
- faults, with or without mylonite filling, undulating and not continuous, forming a braided network with a general N to NE dip direction.

The print of the wedge located on the left bank as seen in Figure 18 is delimited by two of these structural defects, the foliation showing shearing and a fault, both having very close azimuths but opposite dip directions (Table 2).

Thus, the foliation and the fault were roughly orthogonal, creating rectangular cuboidal rock wedges (Goguel, 2010; Duffaut, 2013a) as the large rock wedge on the left bank.

In retrospect, three sets of discontinuities were distinguished in Malpasset. The two major ones were associated to the foliation with low spacing, not continuous but well developed at the dam site, especially on the left bank. In addition, larger fractures (faults) crossing the foundations were also identified. The attitude of these discontinuities was rather similar regarding the azimuth (generally NW - SE), but opposite regarding the dipping angle: around 45°SW (downstream) for the foliation and around 45°NE (upstream) for faults. These characteristics could not have been considered in the design, in view that the normal practice at that time was not for a detailed and systematic inventory of the rock structure in depth. Also, the designers could not have been able to consider the mechanical effects of the water under pressure, flowing through the geological discontinuities and cracks appearing in the foundation of the dam just on the upstream side of the abutments. Indeed, the state-of-art at that time was mostly concerned with the behaviour of porous media than with the still not well known discontinuous fractured rock mass behaviour. The mechanisms adopted at that time in the design, were most linked to shear strength and sliding in the interface concrete-rock.

Many other factors contributed to the rupture since the beginning, as: the tracing of the new axis had not been specifically investigated, due to the assumption that the new site was geologically similar to the one chosen before;

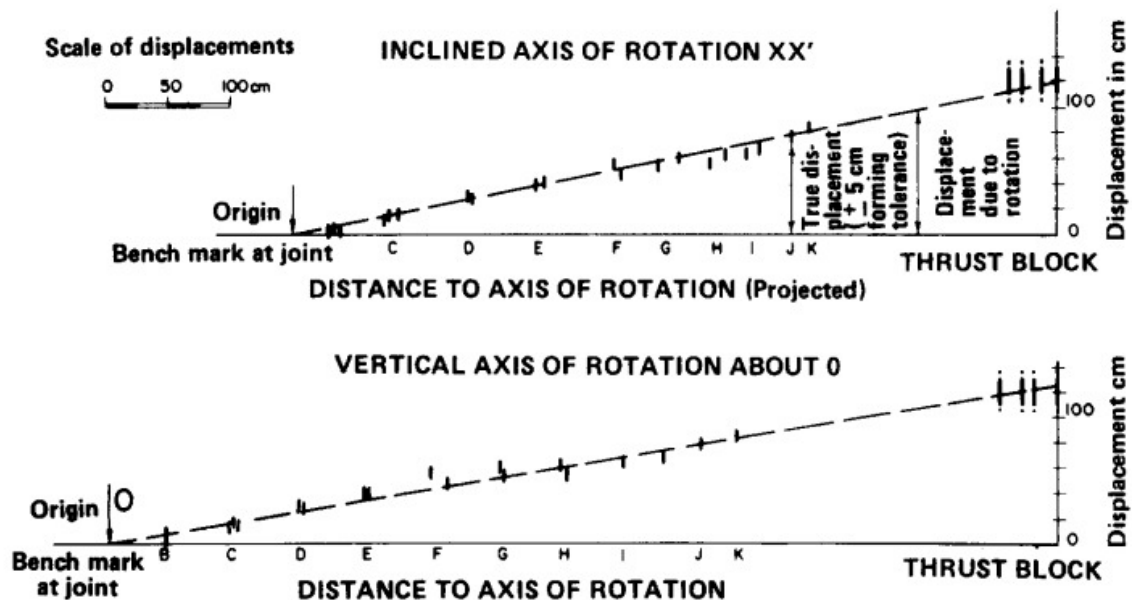


Figure 17. Rotation of the arch (Londe, 1965).

Table 2. Attitude of rock defects in the left bank foundation.

Type	Dip angle / Dip direction	Azimuth / Dip angle
Sheared foliation	45° / N255°E	N165°E / 45°SW
Fault	40° / N15°E	N102°E / 40°NNE

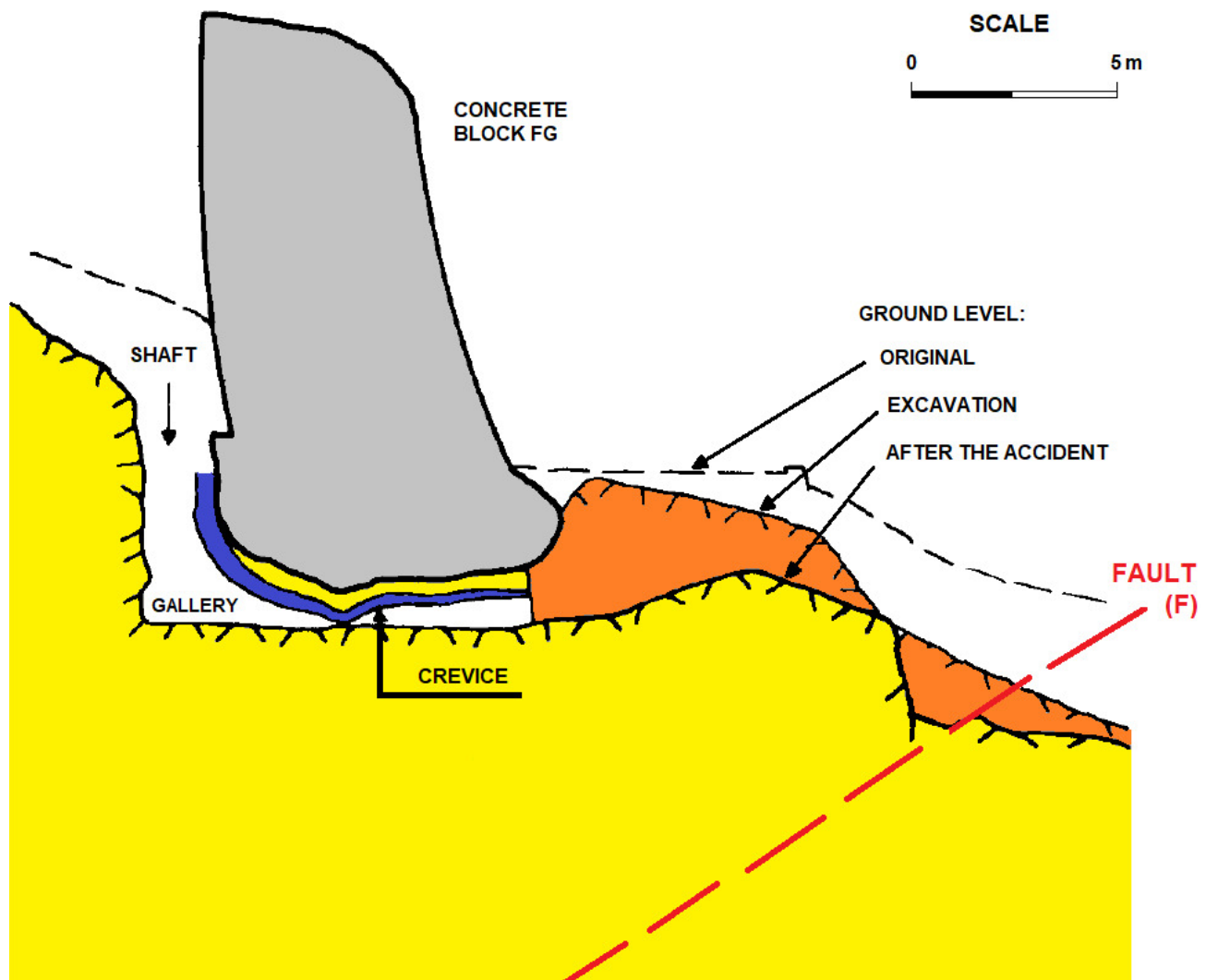


Figure 18. Vertical cross section under the concrete at the foot of the right bank along the shaft and the gallery opened in 1962. The investigations showed the evidence of a continuous crevice in the rock foundation, close to the concrete block FG. This is considered a sign of the movement of the dam on the foundation in the right bank, as described in the text. Modified from Bellier (1967).

topographical points had been installed, however the deformations of the arch during the filling of the reservoir were not monitored; the targets clearly evidenced movements of the arch and vertical displacements as shown in Figure 10, however they were not considered; the springs of water that appeared on the right bank and the deformations observed in the joints of the arch (Figures 6 to 10) would have given indication of some slow movement in the foundation in a monitoring plan.

A survey of the post failure positions and measurements of the remaining dam elements, performed under the requirement of the Inquiry Commission showed that there had been a rotation of the dam as a solid body. Figure 17 (Londe, 1965) shows that the axis of rotation was located at the very end of the arch on the right bank and the best fit is given by an inclined axis (XX' in Figure 17) perpendicular to the fault

plane forming the downstream face of the wedge of rock in the left bank which supported the dam and disappeared during the accident.

This kinematic chain suggests that sliding on this face occurred when the arch dam was still a solid body. The upward component of the movement resulting from this sliding is confirmed by the wide-open crevice under the block FG (right bank) in the investigation shaft and gallery (Figure 18) excavated after the failure (Londe, 1965).

On the left side, the concrete was displaced more than 2 meters in a direction approximately tangent to the arch. The uplift pressures in the rock abutment were also regarded as a possible causative factor, in view of the fine fissures in the gneiss. However, the investigations discharged this hypothesis in view that the problem was inside the foundation. Later on, a consensus came that the seepage seen on the left

bank after the collapse was a consequence of the foundation movement associated with the dam collapse (Jansen, 1983).

The conclusion of the Inquiry Commission pointed out that the construction works of the dam were very good, particularly as regards the quality of the concrete and the contact between the concrete and the rock foundation. The detailed analysis of the conditions prevailing during the construction did not reveal any element which could explain the catastrophe. The stresses computed by the designers (as seen in Figure 3) were within acceptable limits and the anchorage of the concrete in the rock abutment was good. Also, the rupture did not occur at the interface of contact between the concrete and the rock, but within the foundation.

Rock tests and many researches were conducted in the laboratory and site in an attempt to understand the phenomena involved with the catastrophe. The Inquiry Commission decided that an extreme overloading caused the abutment and the underlying rock to move relatively to a plane of shear. The arch could not deform as much as required to transmit the thrust to the deformable rock. It was also estimated that the slow redistribution of stresses in the arch in the first phase of the rupture, may have taken several days or even weeks. However, the second phase represented by the final collapse, took just some few seconds. Terzaghi (1962), stated: "The left abutment of the dam appears to have failed by sliding along a continuous seam of weak material covering a large area".

Witnesses at court testified that there were two faults on the left abutment forming an angle and creating a dihedral piece of rock which was displaced by water pressure. There was also, testimony that measurements of dam deflections were made annually in the period 1955 to 1959 and that a displacement at the dam base of 17 mm was reported, despite that the chief designer had judged the tolerable deflection at that point to be only 10 mm. The court learned that a firm of specialists had conducted the displacement survey, but the design office had not been informed of the excessive deflection until after the failure (Londe, 1965).

The fatal plane of weakness in the rock of the left abutment was about 30 m from the dam and had not been regarded as a threat to the safety of the arch. An unrecognized characteristic of the gneiss rock at the damsite, however, was that it had a permeability varying from 100 to 1000 times, depending on the variation of stresses (tensile or compressive) in the foundation. Researches performed by Bernaix (1967) showed that the influence of stresses had an important role on the behaviour of the rock of the dam foundation. The stresses in the left abutment in response to the reservoir loading on the dam caused a rapid infiltration and development of water pressures on fracture planes normal to the weak foundation plane. This forced the dislocation of a large wedge of rock which had been most probably already loosened and weathered to trigger the collapse of the dam. de A

Another important characteristic of the rock mass in Malpasset was the alteration and deformability of the rock in the left abutment, which had also influence in the rupture.

In fact, there were two types of rock in the left side. One that remained in place after the accident and the other one that was carried away during the process of rupture (see Figure 12), which contained the large wedge that disappeared and most probably had been loosened and weathered more than the rest of the rock in situ due to tectonic effects near the downstream fault. The elastic deformation modulus of the rock derived from jack tests performed in 1960 showed modulus of 850 MPa vertically and 1600 MPa horizontally in the NS and EW direction respectively (Bellier, 1967). According to Mary (1968, in Post & Bonazzi, 1987) the deformation modulus would be around 1000 MPa, perhaps even less. In both cases, however, the modulus is low affecting the stiffness of the foundation.

A model analysis considering the elastic modulus in the left bank (500 to 1000 MPa), the lack of symmetry between the two banks and the artificially stiffened left bank by the thrust block, lead to the conclusion that the elastic modulus of the rock in the left bank was very low (0 to 500 Pa), particularly between the joints K and N. Hence, the left bank thrust block would have been subjected to much higher loads than it had been designed for, say about 25000 ton instead of 5000 ton acting in an unfavourable direction. It is assumed that one of the very first event in Malpasset process would have been the failure of the thrust block, because it had to withstand a major part of the thrust from the dam towards to the left bank (Post & Bonazzi, 1987).

4.2 Sequence of events leading to the accident

After many studies and researches the sequence of events was admitted as follows:

- Existence of major structural features in the dam foundation: the sheared-foliation dipping 45 toward the South and a major fault (F) dipping 40 toward the North and filled with mylonite (geological report written by B. Goguel in 1960 and reprinted in 2010) (Goguel, 2010). On the left bank, the attitudes of these two features allowed the formation of a large wedge of rock which was much relevant in the process of rupture;
- In the dam foundation, the gneiss was intensively fissured, even at the scale of a core and highly sensitive to jacking under water pressure (determined later from radial permeability measurements made by Jean Bernaix, and described in Habib, 1987). Due to the low modulus of the foliated gneiss (discovered only later as written in Duffaut, 2011), the opening of a crevice along the upstream foot of the dam, developing mostly linearly with the reservoir elevation had influence on the water seepage and then water springs at the contact of the concrete-foundation;
- At the downstream foot, the pressure of the dam induced a decrease of the permeability (hydraulic conductivity) inside the foliated foundation which

was qualified as a very sensitive issue at Malpasset site, reaching two orders of magnitude (Bellier, 1967) and leading to an increase of the under-pressure in the foundation, if undrained (Carrère, 2010);

- Increase of water pressure (almost equal to the pressure induced by the height of the reservoir) and, at the same time, reduction of the hydraulic conductivity at the downstream foot of the dam, favouring the effect of pressure directly on the foliation face of the wedge (sf on Figure 19);
- Due to the filling of the reservoir as well as the opening of the upper crevice, the hydrostatic pressure reaches the lithostatic pressure of the wedge. When rounding the density of gneiss at 3 and water at 1 it can be supposed that when the height of water H (H = height of the reservoir + depth along the wedge) becomes 3 times the height of the wedge (h), the stability of the wedge is highly limited and only relies of the friction along the fault face. This face of the wedge is the fault filled with crushed rock and clayey gouge (see Figures 18 and 19) therefore, it could be assumed that the friction angle was very low and the cohesion negligible. Laboratory tests showed this angle to be around 30 to 35 degrees. When H becomes higher than 3 times h , which is the case when the reservoir is almost full, the wedge starts sliding along the fault face, inducing an opening of the upstream face with sheared foliation in which water surges, making the process irreversible (Figure 18);
- Ultimately, the hydraulic uplift pressure on a large rock wedge due to the force of the water percolation in the foundation, causes dislocations and induces ruptures

in the concrete of the arch on the left bank, which cannot support the deformations and the collapse of the left part of the dam is almost complete. After the catastrophe, the print of the wedge became visible with the two faces, the length reaching near 50 m (Figure 19).

The water pressure increase was linear and took time, being visible with water springs two weeks before the failure. The failure itself occurred very quickly, probably in a few minutes.

An unrecognized characteristic of the gneiss rock at the damsite, was that it has a permeability from 100 to 1000 times higher in tension than in compression. As a result, the stresses in the abutment in response to the reservoir loading on the dam caused a rapid infiltration and development of water pressures on planes normal to the weak foundation plane. This forced a large rock wedge to lift up which triggered the collapse of the dam. Unfortunately, the designers did not preview any dispositive of drainage in the foundation following the use at that time, for arch dams.

The court conclusion was for the understanding that human factors had certainly favoured the catastrophe, but could never trigger that. Also, that the state of the art at that time, would not have permitted a deep understanding of the effects of water and in particular of the understanding of percolation forces acting into fractured foundations.

The conjunction of the geological structures at Malpasset dam, together with the effects of the water pressure acting in the foundation of the dam, were in the origins of the collapse. Without the weathering and shearing of the foliation, it had been difficult the infiltration of water deeper in the foundation as well as the action of the uplift pressures in a

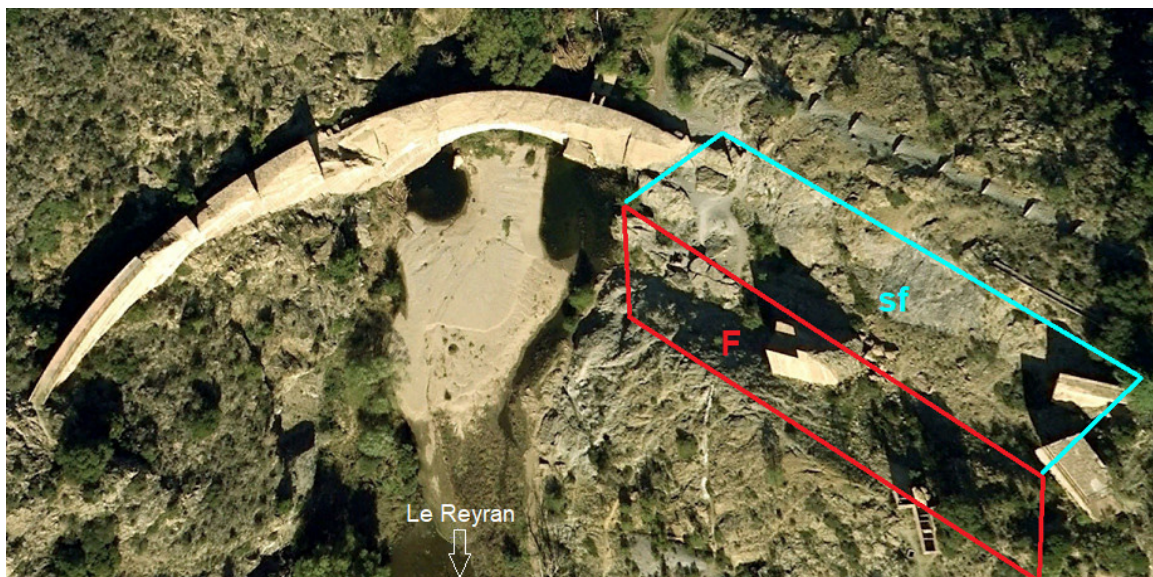


Figure 19. View of the print of the rock wedge and the damaged dam structure on the left bank. (Google Earth® 2012 archive view).
sf = sheared foliation; F = Fault

huge rock wedge. In the same way, without a fault forming the downstream face of the wedge (Figures 20 and 21), it would not be possible for the wedge to slide easily and allow the water to penetrate and destroy the whole foundation and the dam above.

Figure 20 shows that on the left bank, the concrete arch followed the foliation. The axis of the arch is almost parallel to the foliation and the dam loading induced a

compressed zone inside the foundation due to the foliation. This geometrical situation induced an increase of the compressive strength and a severe reduction of the hydraulic conductivity inside the compressed zone which, associated with the water accumulated behind the dam and inside the foundation upstream of the induced hydraulic barrier caused, in consequence, an uplift pressure that lifted up the rock wedge delimited by the compressed foliation and the

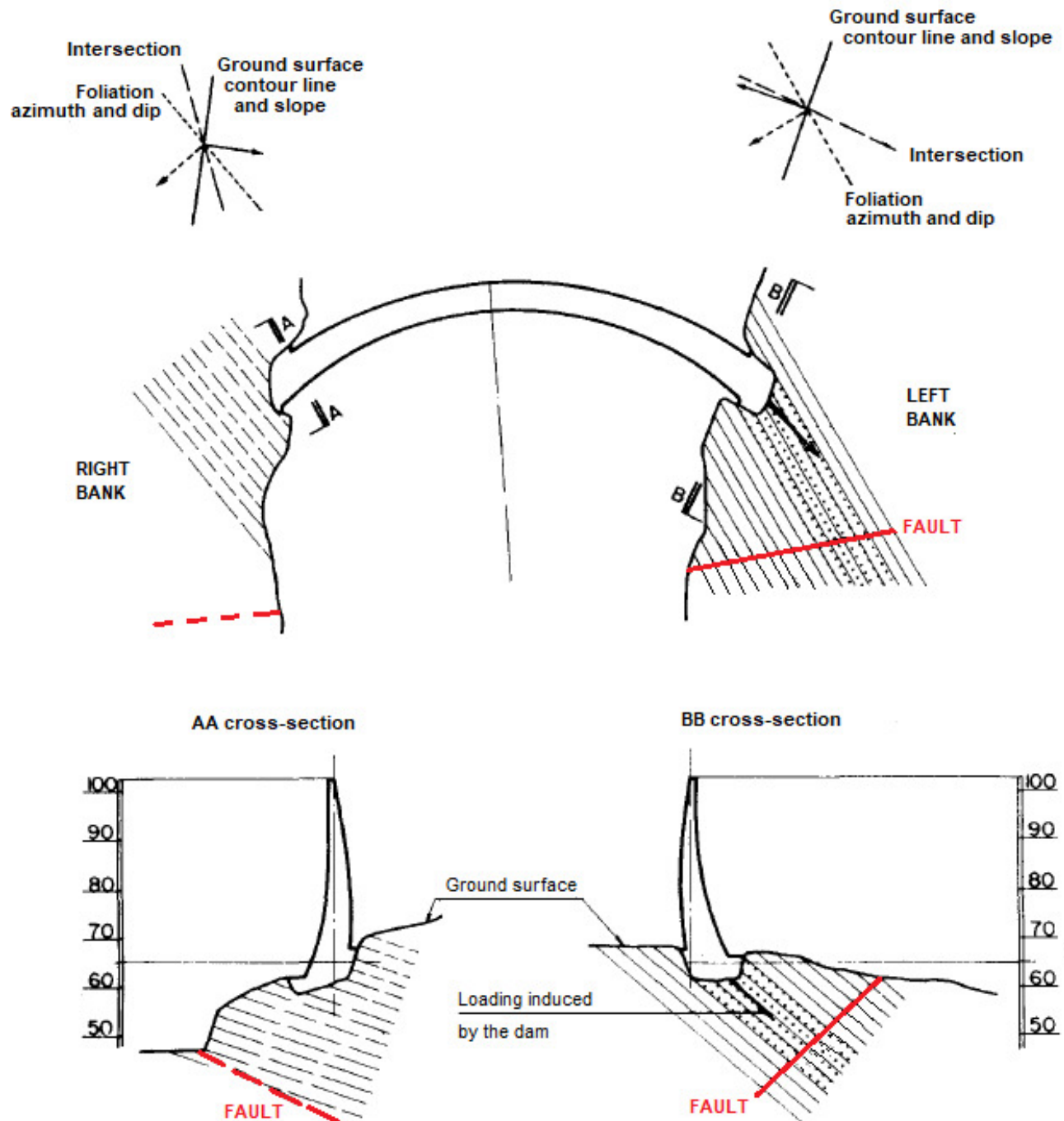


Figure 20. Plan and cross section of the concrete arch and the integration with the local geology. Modified from Bellier (1967).

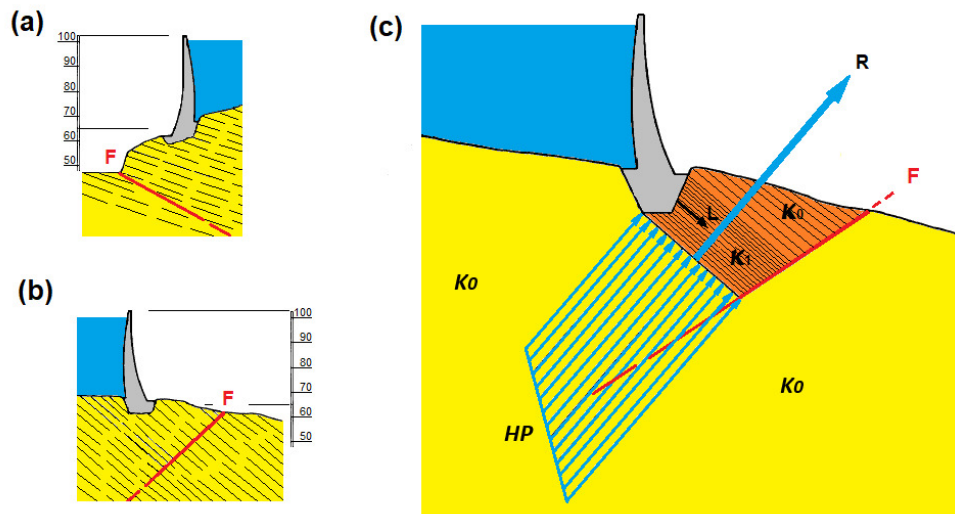


Figure 21. Geological cross-sections (modified from Bellier, 1967) with (downstream) fault and foliation (a) on the left bank; (b) on the right bank; (c) sketch showing the resultant R of the water pressure (uplift pressure) acting on the rock foundation with loading from the arch (L) and downstream fault plane (F). Common legend: Yellow = foundation; Orange = rock wedge; Dashed lines = foliation; F = downstream Fault (in red); HP = full Hydrostatic Pressure corresponding to the maximum water level in the reservoir; L = Dam Loading; R = Resulting Pressure (uplift); K_0 = permeability of the foundation; K_1 = reduced permeability in the loaded zone ($K_1 \leq K_0/100$).

downstream fault, leading progressively to the dam failure as showed in Figure 21.

In the right bank the situation is very distinct. In the two corresponding cross sections, it is easy to observe the differences between the two banks. In particular the cross-section BB shows the zone compressed by the thrust of the arch on the left bank. This zone could penetrate far in depth due to the foliation.

Figure 21 describes that mechanism of rupture. The compressed zone of the left bank foundation was made watertight by the closing of the fissures. The permeability has become 100 times or even 1000 times lower, transforming the rock into a real “underground dam” that supports the full hydrostatic pressure (HP) of the reservoir. In the figure, it is possible to see how large and in unfavourable direction this force is putting in risk the stability of the whole left bank.

In fact, the process occurred in Malpasset, resulting from the effect of water and uplift pressures acting in a very large wedge of rock in a fractured rock foundation, was totally unknown by the engineers at time. Thus, it is possible to admit that only a real lack of knowledge, could have made the designers and the geologists to be so confident when the geological features were pointing for a tragedy.

5. Conclusions on the effects of Malpasset accident on dam engineering and rock mechanics development

More than sixty years after the accident, Malpasset still represents a unique case of a collapse of a concrete arch dam

in the history. The lessons learnt from the failure of Malpasset dam have been many and they were not restricted only to dam engineering. Many branches of the geotechnical sciences as engineering geology, hydrogeology, soil and rock mechanics and so on, had a strong development after this catastrophe.

After the accident and the study and understanding of the origins of the collapse, many advanced studies and researches were conducted in different countries around the world particularly from the sixties, to understand the problems related to flow in fractured rocks. USA (Berkeley), UK (Imperial College), France (BRGM), Germany (Aachen) and many other Academies and Research Institutes in the world developed sophisticated laboratory researches on this subject. Modelling in fractured formations had also benefited much from these studies and researches, particularly with the possibility of better input data for the difficult problems.

Some subsequent dramatic improvements occurred during the following decades. Among others:

- The analysis of the rock mass behaviour as a whole, considering the effects of the geological structure and the water flow on the global stability of the dam (Carrère, 2010) and also in tunnelling;
- Development of a graphical method by Pierre Londe (1965) so-called Londe’s wedge method, for assessing the stability of arch dam foundations and also of the hydro-mechanical coupled concepts and models for the study of arch dam foundations (Carrère, 2010; Wittke & Leonards, 1987). These concepts and principles developed for flow in fractured formations

were applied also for the stability analysis and design of gravity dams and varied subterranean works in rocks;

- The systematic use of drainage curtains, introduced in England in 1890 and applied in France for the design of earth dams after 1920 (Bordes, 2010), had its importance highlighted after the accident of Malpasset. The use of grouting and drainage curtains for arch dams built on fractured foundations had expanded in dam engineering to avoid the increase of water pressure and to reduce the uplift pressure in the rock foundations and abutments;
- Monitoring of dams' behaviour starting since the beginning of the construction and accompanying the infilling of the reservoir became mandatory. Also, maintenance during operation (Bordes, 2010; Carrère, 2010);
- Rules and administrative controls before the dam construction and reservoir filling were also improved after the accident (Duffaut, 2009, 2013a, b).

Malpasset raised the opportunity for drastic changes in various different domains of dam engineering, starting with many improvements of investigations using also the advances in technology to introduce computer data acquisition in field tests, as modelling of rock foundations and abutments, design and construction. After Malpasset, the effect of the groundwater pressure has been carefully analysed on every dam site, worldwide (USBR, 1998). The design in mining and all subterranean works in rock masses, also had much improvement when considering the hydrogeological effect on the excavations taking in account the geological scales varying from the regional to the local scales of the projects.

These improvements started in France and were spread to the world since then. It can be said that most of the developments in design, construction and operation of dams in these last almost 100 years, is owed to some of the accidents that occurred in the last century involving arch and gravity dams, in particular as St Francis in USA (1928), Malpasset in France (1959) and Vajont in Italy (1963).

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Declaration of interest

The authors declare no conflict of interest between them and the companies cited in this publication. This technical case

has only been studied for the benefit of the rock engineering community.

Authors' contributions

Eda Freitas de Quadros: formal analysis, methodology, conceptualization, supervision, validation, writing - original draft. Philippe Vaskou: data curation, visualization, writing – review & editing.

Data availability

All the data used in the current study come from the published literature: the historical and design data can mainly be found in Comité Français des Barrages et Réservoirs (CFBR, 2009) and the quantified data are available from Bellier (1967), Mary (1968) and Londe (1965).

Declaration of use of generative artificial intelligence

This work was prepared without the assistance of any generative artificial intelligence (GenAI) tools or services. All aspects of the manuscript were developed solely by the authors, who take full responsibility for the content of this publication.

List of symbols and abbreviations

h	Horizontal
v	Vertical
sf	Shear foliation
ACJB	Name of the engineering consultancy company established in 1947 by André Coyne and Jean Bellier, and specialized into dam design. Also referred to as COB.
BC	Before Christ
BRGM	French Geological Survey (Bureau de Recherches Géologiques et Minières)
C	Profiles (C0 to C5) for the calculation of principal stresses in the concrete dam
Dec	December
<i>Ec</i>	Young modulus of concrete
EDF	Electricité de France (French Electric Board)
EL.	Elevation (in metres above mean sea level)
<i>Er</i>	Young modulus of foundation rock
F	Fault
<i>HP</i>	Hydraulic pressure
ISRM	International Society for Rock Mechanics and Rock Engineering
<i>K0</i>	Permeability of the foundation
<i>K1</i>	Reduced permeability in the loaded zone ($K1 \leq K0 / 100$)
L	Loading
Nov	November

R Resulting pressure (uplift)
Sf Shear foliation

References

- Bellier, J. (1967). Le barrage de Malpasset. *Revue Travaux*, (July), 3-23 (in French).
- Bernaix, J. (1967). *Étude géotechnique de la roche de Malpasset* [Doctoral thesis]. University of Paris, Dunod.
- Bordes, J.-L. (2010). Les barrages en France du XVIII à la fin du XX siècle. *Pour Mémoire*, 9, 70-120 (in French).
- Bordes, J.-L., & Tardieu, B. (2015). André Coyne, de la Dordogne au Zambèze, la passion de construire. *Bulletin de la Sabix*, 56, 8-53 (in French). Retrieved in July 16, 2021, from <http://journals.openedition.org/sabix/1452>
- Boudou, M. (2015). *Approche multidisciplinaire pour la caractérisation d'inondations remarquables* [Doctoral thesis]. University of Montpellier III, France (in French). Retrieved in April 2, 2026, from <https://tel.archives-ouvertes.fr/tel-01305937>
- Carrère, A. (2010). Les leçons de Malpasset. Leur application aux barrages d'aujourd'hui. *Revue Française de Géotechnique*, (131-132), 37-51. <https://doi.org/10.1051/geotech/2010131037>.
- Comité Français des Barrages et Réservoirs – CFBR. (2009). *Barrage de Malpasset (Var). L'accident du 2 décembre 1959* (5 p.). Paris (in French).
- Duffaut, P. (2009). Cinquantenaire de la rupture des fondations et du barrage de Malpasset (Var). In *Travaux du Comité français d'Histoire de la Géologie (COFRHIGEO)* (pp. 201-224), Paris (in French).
- Duffaut, P. (2011). What modern rock mechanics owe to the Malpasset arch dam failure. In *Proceedings of the 12th ISRM Congress*, 18-21 October 2011, Beijing. <https://doi.org/10.1201/b11646-359>.
- Duffaut, P. (2013a). The traps behind the failure of Malpasset arch dam, France in 1959. *Journal of Rock Mechanics and Geotechnical Engineering*, 5(5), 335-341. <https://doi.org/10.1016/j.jrmge.2013.07.004>.
- Duffaut, P. (2013b). *Rock mechanics lessons from dams*. International Society for Rock Mechanics. Retrieved in July 16, 2021, from <https://www.isrm.net/gca/?id=1230>
- Goguel, J. (2010). Rapport Géologique Malpasset. *Revue Française de Géotechnique*, 131-132(131-132), 25-36. <https://doi.org/10.1051/geotech/2010131025>.
- Habib, P. (1987). The Malpasset dam failure. *Engineering Geology*, 24(1-4), 331-338. [https://doi.org/10.1016/0013-7952\(87\)90070-6](https://doi.org/10.1016/0013-7952(87)90070-6).
- Habib, P. (2001). Recherche appliquée, responsabilité et principe de précaution. *Revue Française de Géotechnique*, 94(94), 25-30. <https://doi.org/10.1051/geotech/2001094025>.
- Jansen, R.B. (1983). *Dams and public safety*. Washington, D.C.: U.S. Department of Interior, Bureau of Reclamation.
- Lane, K. & Garfield, L. A. (Eds.). (1972). *Proceedings of the North American Rapid Excavation and Tunnelling Conference*. San Ramon, CA: American Society of Civil Engineers, American Institute of Mining, Metallurgical, Petroleum Engineers.
- Londe, P. (1965). Une méthode d'analyse à trois dimensions de la stabilité d'une rive rocheuse. *Annales des Ponts et Chaussées*, 135(1), 37-60 (in French).
- Mary, M. (1968). *Barrages-Voûtes: historique, accidents et incidents* (Vol. 1, 159 p.). Paris: Dunod (in French).
- Post, G., & Bonazzi, D. (1987). Latest thinking on the Malpasset accident. *Engineering Geology*, 24(1-4), 339-353. [https://doi.org/10.1016/0013-7952\(87\)90071-8](https://doi.org/10.1016/0013-7952(87)90071-8).
- Roubault, M. (1970). *Peut-on prévoir les catastrophes naturelles*. Paris: Presses Universitaires de France (in French).
- Shaffner, P. (2015). Turning geologic data into knowledge: a summary of dam failure case histories. In *Proceedings of the AEG's 58th Annual Meeting*, Pittsburgh, PA. Retrieved in July 16, 2021, from https://www.aegweb.org/assets/docs/ds11_summary_of_dam_failure.pdf
- Terzaghi, K. (1962). *Comment on Malpasset failure in February*. Washington, D.C.: Dam Safety Office, Department of Interior.
- United States Department of Interior – USBR. Bureau of Reclamation. (1998). *Concrete dams: case histories of failures and nonfailures with back calculations: structural analysis* (DSO-98-05). Washington, D.C.: Dam Safety Office, Department of Interior.
- Valentini, V., & Bertini, A. (2003). *Barrage de Malpasset: de sa conception à sa rupture* (Hors Série, No. 3, 223 p.). Saint-Raphaël: Société d'histoire de Fréjus et de sa région (SHFR) (in French).
- Ville de Fréjus. (2016). *Barrage de Malpasset: l'historique*. Retrieved in July 16, 2021, from <https://frejus.fr/decouverte/le-barrage-de-malpasset/>
- Wittke, W., & Leonards, G. (1987). Modified hypothesis for failure of Malpasset dam. *Engineering Geology*, 24(1-4), 367-394. [https://doi.org/10.1016/0013-7952\(87\)90073-1](https://doi.org/10.1016/0013-7952(87)90073-1).