Geotechnical Hazards | Geological Considerations in Dams Failure

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Abstract

Dams and reservoirs pose safety concerns to society worldwide. In case of a disaster, the water impounded in the reservoir escapes and destroys everything in its path. Reasons for failure range from geology, hydrology and seismicity, to design problems, lack of maintenance and poor field investigation. Prior cases show that various dams gave away mainly due to geological causes, so there is a particular interest to see how the local terrain features could influence the longevity of the structure.

Three historical case studies are discussed in order to emphasize the impact of geology regarding dam failure. The Saint Francis Dam is a prime example of poor site investigation, where the lack of knowledge on the foundation rock led to the rupture of the gravity dam. The Malpasset Dam gave away predominantly due to underestimated effects of the uplift, nevertheless, the geologists were unaware of an active fault system and the mechanical properties of the rock mass. The Baldwin Hills Reservoir comes with a more thorough site investigation, yet still, due to earth movements, the water from the reservoir infiltrated through the embankment.

Therefore, geological features at the site need to be included in the design options of the dam in order to ensure a safe, feasible and economical project. With respect to the way we build nowadays, engineers have learnt important lessons from past experiences, however, issues such as ageing of the structures and the unpredictability of geology and weather, could still influence the safety of modern dams.

Preface

Dams have been considered symbols of civilization and cultural development since ancient times. The first ever built dam is considered to be the 4000-year-old Sadd-el-Kafar, built by the Old Egyptian Empire, which was purposed to impound water on a larger scale (Jackson, 1997).The Roman Empire followed with great advancements in geotechnical structures and remarkable inventions such as the arch dam (please see Figure 1 for an example). Nowadays, maybe more than ever, the dam industry needs to grow rapidly due to an expanding demand in hydropower and water supply, so the search for new site locations and best designs remains an enormous challenge. Over the past two centuries, large reservoirs and dams were constructed, most of which still remain fully functioning today. However, several geotechnical disasters occurred over time, and whilst there is not an exact record of all failures, we can acknowledge the impact of the consequent flooding on human lives and the serious financial situation due to damage of properties and infrastructural facilities.

Figure 1: Glanum Dam was built in 1st century BC by the Roman Empire. Located in the southern France, the first ever built arch dam stored water for the Glanum Town. A new modern structure was built on top of the ruins and forms the Peiròu Lake ("Glanum Dam - Alchetron, The Free Social Encyclopedia," n.d.)

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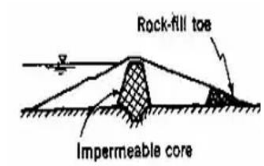
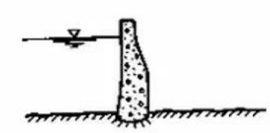
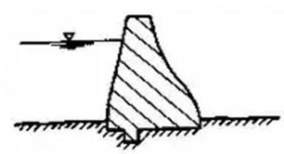
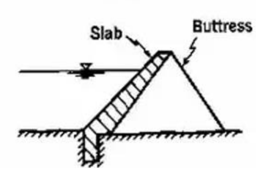
1. Introduction

Dams are barriers which restrict the flow from a river and create a reservoir behind with the aim to impound water so much needed by society. They are massive structures which vary in shape, complexity and function. What makes them so outstanding is their uniqueness, because there are no two dams in the world that are alike. However, most of their design are based on one of the types found in Table 1. Landscape diversity and rock anisotropy at the site require the engineering team's ability to adapt to each situation and create a structure never built before.

Table 1: Basic dam classification in terms of design and material

TYPES OF DAMS	CROSS-SECTION
1. EMBANKMENT DAM ❖ *MATERIAL:* ROCKFILL OR EARTHEN ❖ *CONTROLLING FACTOR:* MATERIAL LOCALLY AVAILABLE ❖ *DESIGN:* IMPERMEABLE CLAY CORE ❖ *FOUNDATION:* LOW BEARING STRENGTH ❖ *LOCATION:* WIDE VALLEYS	
2. CONCRETE ARCH OR DOME DAM ❖ *DESIGN:* THIN OR THICK ARCHES WITH STEEL RODS AND CABLES ❖ *FOUNDATION:* HIGH BEARING STRENGTH ❖ *LOCATION:* GORGES OR NARROW VALLEYS	
3. CONCRETE GRAVITY DAM ❖ *CONTROLLING FACTOR:* LARGE AMOUNT OF AGGREGATE STONE NEEDS TO BE LOCALLY AVAILABLE, SOMETIMES MASONRY ❖ *CONTROLLING FACTOR:* HARD ROCK NEAR SURFACE ❖ *FOUNDATION:* MEDIUM BEARING-STRENGTH	
4. CONCRETE BUTTRESS DAM ❖ *MATERIAL:* LOWER AMOUNT OF AGGREGATE STONE ❖ *FOUNDATION:* MODERATE TO HIGH BEARING STRENGTH REQUIRED ❖ *SPECIAL FEATURES:* ELIMINATION OF UPLIFT PRESSURE	

The occasional misjudgment of the engineering team is one of the topics that will be argued in this paper. Misguided decisions, due to lack of knowledge combined with incomplete site investigations are typically the reasons behind major dam failures. These massive geotechnical structures have been improved over the years, as teams of engineers across the world learn from mistakes after every disaster.

In this paper, sources of failure behind several major disasters with respect to dams and reservoirs will be addressed. A condensed overview of the primary cause of collapse can be reviewed from Table 2. This classification also gives examples of structures which gave away, however, in most of the circumstances, the dams failed due to more than one source. In consequence, one might need to consider a causal mechanism which aims to recreate the chain of events prior to the incident in order to identify the chronological order of the factors which led to the instability of the structure. Moreover, besides the sources of failure inserted in Table 1, dam safety depends of human factors, such as maintenance, monitoring and surveillance. The rupture of the structure affects mostly the immediate vicinity. Nonetheless, the subsequent uncontrolled flood leads to further destruction downstream. Thus, security and stability of dams are of great importance to the safety of the general population.

Table 2: Outline on dam failure sources

NATURAL		MAN MADE	
CAUSE	Example	**CAUSE**	Example
GEOLOGICAL		**DECISIONS**	
❖ ROCK WEAKENING	ST. FRANCIS, *U.S.A.*	❖ DESIGN	BOUZEY, *FRANCE*
❖ INSTABILITY ABUTMENT	MALPASSET, *FRANCE*	❖ FOUNDATION	WOODHEAD, *UK*
		❖ SPILLWAY	DALE DYKE, *UK*
❖ SUBSIDENCE	BALDWIN HILLS, *U.S.A.*	❖ MATERIAL	EIDON, *AUSTRALIA*
❖ RAPID DRAWDOWN	EIDON, *AUSTRALIA*	❖ STRUCTURAL	VEGA DE TERA, *SPAIN*
❖ LANDSLIDE	VAJONT, *ITALY*	❖ OPERATIONAL	VAL DI STAVA, *ITALY*
❖ EROSION	TETON, *U.S.A.*		
CLIMATE		**STABILITY**	
❖ TEMPERATURE	AYERES ISLAND, *U.S.A.*	❖ SLIDING	GLENO, *ITALY*
		❖ SHEARING	BOUZEY, *FRANCE*
FLOODING			
❖ OVERTOPPING	SEMPOR, *INDONESIA*	❖ UPLIFT	HABRA, *ALGERIA*
		DEFORMABILITY	
		❖ DIFFERENTIAL	BALDWIN HILLS, U.S.A.
		❖ CYCLIC LOADING	
SEISMIC INSTABILITY	FUJINUMA, *JAPAN*	**DELIBERATE**	
		❖ BOMBING	MÖHNE, *GERMANY*

Moreover, as seen from Table 2, a significant percentage of the possible causes of dam failure is the geology. The bridge between geology and engineering consists of three area: stability of foundations, watertightness of the reservoir basin and the availability of natural material for construction. Therefore, the choice of a certain dam type is severely influenced by the underground structure, rock quality, topography and earth movements.

The purpose of this research is to answer the question: To what extent do the geologic setting and rock characteristics at the site location influence the longevity of a dam and reservoir project? This rather complex topic will be explained by looking at three case studies which deal with dam and reservoir preliminary demise, which refers to failure which is reached before the end planned lifetime of a structure. Accidents in dam engineering happen everywhere and they are due to multiple causes. However, this paper will emphasize the direct influence of both regional and the local geology in the design of a secure, viable and cost-effective dam and reservoir project.

The reason behind focusing on geotechnical disasters rather than successful projects is that lessons can be drawn from each incident, and the modern structures are just upgraded versions of former dam constructions. This could indicate whether engineers today are designing sufficiently safe structures, and to determine what is left to improve. The ideology of Prof. Henry Petroski, an American professor in civil engineering and expert in failure analysis, is that each time something fails, there is the opportunity to understand something more:

"Failure is central to engineering. Every single calculation that an engineer makes is a failure calculation. Successful engineering is all about understanding how things break or fail." (Interesting Engineering, 2018)

2. Engineering Geology

The safety, feasibility and cost of a dam are all dependent on the geology at the site. Hence, an engineering geologist should be aware of the regional and local geology in order to give the best advice to the design team. The purpose of this chapter is to provide the reader with a succinct overview of the most important geological features which come in direct relation to the construction of the dam or reservoir, most of which will be mentioned later in the case studies.

The preliminary site investigation gives a valuable insight into the regional geology. Location of major faults, layering, deformation processes, age of the sediments, erosional agents etc. can all reveal key features about the rock mass. Regional geology could additionally provide hints about potential paleolandslides, large scale karstic formations or any unconformities found at the site location. Moreover, ground movements should be acknowledged, such as earthquakes, regional subsidence or tectonic activity.

On the other hand, local geology provides more specific details about the subsurface at the site location. This information is used in the design of the foundation and it is essential to be known for the longevity of the project. For instance, surface geology describes the orientation of the discontinues in the rock formation and the dipping of the bedding planes, but also includes information about the extent of weathering. However, geologists encounter difficulties when the rock is not exposed at the surface and either invest in costly drilling activities, or rely on their expertise. Apart from surface investigation, geophysical exploration such as seismic and electric surveys can be used to determine the layering or some other features of the subsurface. Drilling proves to be necessary as it gives very precise data about the stratigraphy and petrophysical properties of the foundation rock. However, drilling is done at one spot, so geophysical surveys can be further used to interpolate between the unknown locations in order to provide a better visualization of the subsurface.

With respect to the types of rock that affect the dam construction, Walters (1962) classified the material depending on its the suitability as a foundation. Ideally, the dams are found on strong rocks (found close to the surface), little pervious and as compact as possible. Thus, here we can mention the next categories:

- *Granite* is sound rock for foundation due to its strength, however, engineers should beware of fissures, disintegration and china clay intrusions. A common feature is sheeting due to relief of tectonic stresses and sometimes ‘onion peel’ formation (Thomas, 1979). An example of a dam site located on very fissured granulite rock is Lavaud-Gelade Dam in France. The solution to its high permeability was to conduct extensive consolidation grouting injection with cement clay and bentonite (Walters, 1962).

- *Gabbro, andesite and basalt* are in general not recommended for foundations because they do not support water-retaining structures. For instance, Zerbine Dam in Italy founded on compacted serpentine (gabbro exhibiting serpentinized olivine) highly fractured, collapsed due to sudden increase in precipitations (ICOLD, 1974).

- *Metamorphic* rocks are satisfactory for foundation; however, grouting is essential as it improves the bearing capacity and it reduce the surface seepage to permissible levels (Wahlstrom, 1975). Here the most common types of dam are rock filled and buttress.

- *Gneiss* and mica schists are acceptable for sustaining bearing pressure and watertightness, however they facilitate sliding if mica minerals are present in excess. A representative example for a site where gneiss is associated with schist, which exhibits a large quantity of silicate minerals, is the Forks Dam in California which was shortly abandoned after construction.
- *Limestone* usually poses difficulties features such as slip planes, karstic elements, large fissures and weak zones. Grouting is again found as a solution to reduce the large-scale permeability in this type of rock, however, in some cases this operation might cost as much as the dam itself. Hales Bar Dam in U.S.A had a cavernous foundation carved in Mississippian highly soluble limestone, where seepage developed in vertical channels (ICOLD, 1974).
- *Sandstone* can be a very porous, permeable and erodible rock which often alternates with weaker sedimentary beds such as claystone, shale and siltstone. One of the major disasters in France was the Bouzey Dam found on soft Lower Triassic sandstone which was scoured out under the dam. Grouting the foundation could have reduced the perviousness of the soil, however, this geotechnical technique was unknown in 1895, the year of failure (Walters, 1962).
- *Clay*stone, when found in massive thick beds sometimes interbedded with sandstone or limestone, only allows earth or rockfill dams due its low bearing strength. A significant example of failure is the Eildon dam in Australia, which is founded on clay. The incident involves a slip circle failure as a result of pore pressure in the clay foundation, which experienced liquefaction (ICOLD, 1974).
- *Sand, gravel and clay* in uncompacted form are soft materials, permeable and only appropriate for small dams with agricultural purpose. The importance of compactness of the soil is exemplified by Apishapa Dam in the U.S.A, where the ground was sandstone alternating with shale and clay beds. The material in the earthen dam settled and formed cavities which made a perfect passage for the water to infiltrate (ICOLD, 1974).

Knowing the strength and the continuity of the rock mass comes as an additional information to the geological interpretations and it is a decisive factor which might impact the longevity of the project. Thomas (1979) classified the rocks in relation to their mechanical properties (criteria which will be often used when describing the rock mass in the upcoming case studies):

- *Uniaxial Compressive Strength:* weak (less than 35 MPa), strong (35 to 115 MPa) and very strong (greater than 115 MPa)
- *Failure characteristics:* brittle and plastic
- *Homogeneity*: massive and layered
- *Continuity:* solid (joint spacing greater than 2 m), blocky (joint spacing 1 to 2 m) and broken (highly fragmented)

Another significant property of the rock which can directly influence the choice of the design is weathering. Here, it can be distinguished between fresh rock, moderately weathered, completely weathered, and a soil. The degree of degradation of the rock is directly influenced by climate, weather, groundwater, humans etc.

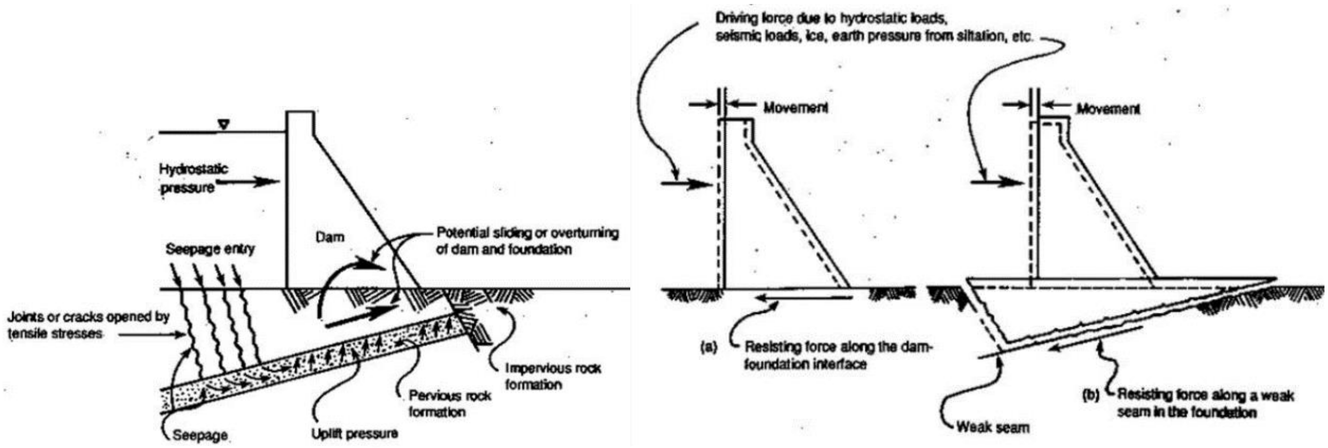


Figure 2: Uplift pressure in a dam (left) and sliding failure (right) (Empson, 2013)

However, when the rock has satisfactory strength, weakness could arise from the orientation and dip of the discontinuities (faults and folds, cracks, joints, foliation planes etc.) as a potential result of the increase in load pressure. The following list is a selection of terminology briefly described which will be used regularly throughout this paper when describing the geology:

- *Bedding planes* mark the interface between the two layers and represent a weakness for the rock as slip planes can form
- *Folds* result from tectonic movement and are the primarily factor which shapes the topography. Typical types of folds are synclines and anticlines and it not uncommon for faults to breach these formations and create a more complex picture of the subsurface.
- *Faults* are fractures formed due to movement of the earth. Depending on which side went up of down, there is normal, reverse and strike-slip faulting. With respect to faults, slickensides, gouge material, crushing degree and offset are essential aspects to be considered when choosing the right foundation for the dam structure.
- *Joints* are a type of fractures at a smaller scale not caused by earth moments. However, every rock exhibit joints which shall increase its permeability, or is case clay intrusions are present, sliding could freely take place.

When a dam has the objective to store water/tailings, several effects should be additionally taken into consideration such as hydrostatic pressure, seepage, uplift, corrosion, piping etc. The rock is tested in laboratory, but it is also required to be analyzed in situ, under fully saturated conditions and under load. All these concerns should be taken into consideration when designing a dam.

Figure 2, left sketch, illustrates the effects of the uplift pressure present in the cracks within a dam but also in the pores and joints of the foundation rock. Figure 2, right sketch, puts emphasis on the devasting effect of sliding if the driving force is greater than the resisting force. Both uplift and sliding are drivers of instability for dams and can lead to failure, as exemplified in Table 2.

Therefore, geology and rock mechanics play in important role in the design of a dam. Detailed field mapping and sufficient site investigation procedures should provide the engineering geologists with the necessary information about the subsurface.

3. Case study: Saint Francis Dam

The St. Francis Dam failure is the first of the three case studies chosen as a remarkable example of a major geotechnical disaster. Geology is the key factor which influenced the dam to collapse sooner than its end life. This chapter will prove that lack of knowledge and professionalism of the engineering team led to this catastrophic incident, followed by a hasty incomplete post disaster investigation subject to extensive discussion.

3.1 Overview Failure

Saint Francis Dam was found north-west of Los Angeles, Southern California, U.S.A. (for exact location see Figure 3). The construction of the 56 m high curved concrete gravity dam was finished in 1926. The owner was the City of Los Angeles Bureau of Waterworks which acknowledged in 1918 a water crisis in the region (ICOLD, 1974). The officials assigned designer of the LA aqueduct, William Mulholland, to be the chief engineer for the Saint Francis Dam. The project was aimed to expand the available water supply in the arid Californian climate by impounding water in a reservoir which can be seen behind St. Francis Dam in Figure 4. The reservoir is located in San Francisquito Creek between Power House No. 1 (upstream) and Power House No.2 (downstream) and the natural constitution of the canyon facilitated the construction this large gravity dam.

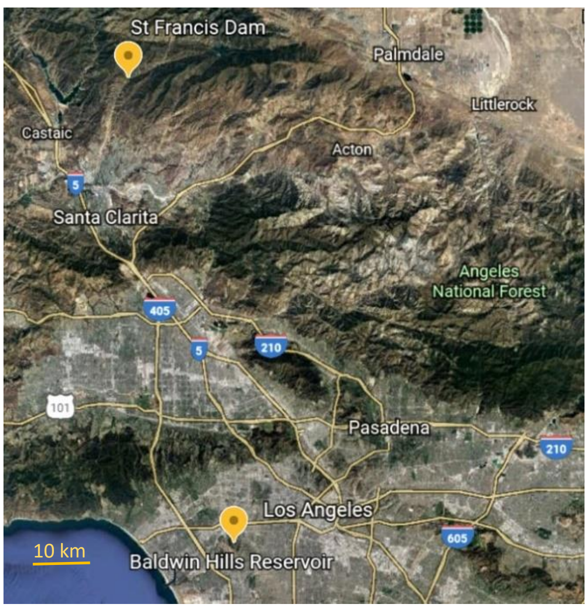


Figure 3: Location of Saint Francis Dam and Baldwin Hills reservoir ("Google Earth," n.d.-a)

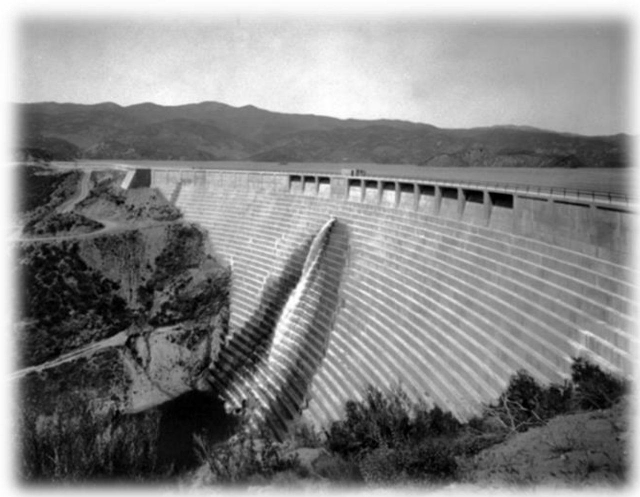


Figure 4: Photo of Saint Francis Dam after construction (SCVNews, 2018)

Two years after the construction, the reservoir was finally full on March 11, 1928. One day later, at midnight between 12 and 13 of March, the dam burst open unexpectedly, and the muddy water rushed downhill. The disaster cost the American government over $10 million, and the consequences of the flooding are the loss of life of 432 people, the destruction of several properties such as the Power Plant Number 2, Saugus substation and construction site, highway bridges etc., making the event the deadliest American Civil Engineering failure of the 20th century (Rogers, 2006).

3.2 Geology

In order to understand why the concrete structure suddenly failed, it is important to have a grasp of the geology of the area. From the regional geology, we know that the dam is located between San Gabriel and San Andreas faults, in an area with intense folding and faulting. The canyon was structurally controlled by the ancient Francisquito Fault, which put together the older late Cretaceous Pelona schist against Oligocene Vasquez formation of conglomerate (Rogers, 2006). Thus, the structure is founded on two formations with different properties which were unfortunately unknown at the time Mulholland designed the dam. Figure 5 shows a section of the geologic map of the canyon with current material exposed nowadays at the surface, which identifies the location of the St. Francis Dam, the LA aqueduct and the San Francisquito Fault.

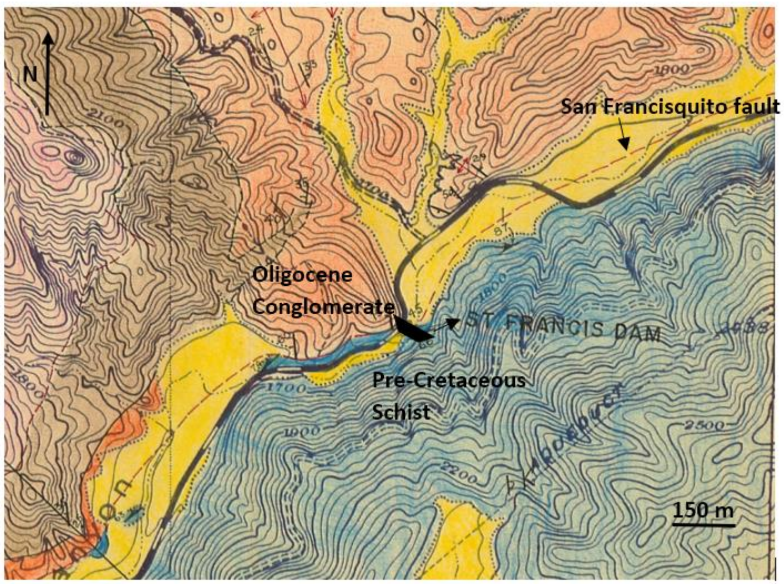


Figure 5: Geological map of the San Francisquito Creek (Ruiz-Elizondo, 1953)

For the local geology, the cross-section in Figure 6 gives the stratigraphy at the site location. The upper right abutment consists of the Vasquez formation, later renamed Sespe. The reddish conglomerate sandstone is soft and plastic in unsaturated conditions, and susceptible to slacking when submerged (Rogers, 2013), reaching a strength of only 3.5 MPa (Stapledon, 1976). However, in dry state, the material is much firmer but exhibits fractures filled clay or gypsum gauge (ICOLD, 1974).

The two meter wide Francisquito fault is filled with gravelly clay gauge (Stapledon, 1976) of comminuted schist which is hard when dry and oily when wet. The fault lays halfway the abutment and has a strike parallel with the course of the canyon (ICOLD, 1974). The strike-slip fault juxtaposes high-grade Pelona Schist on the south with non-metamorphosed Mesozoic rock units on the north such as the Sespe Formation (Bunker, 2001). The fault can be seen also a dotted red line in Figure 5 in the geological map, creating a clear boundary between Sespe and Pelona Formation.

The lower section of the right abutment consists of moderate strength Pelona formation which consists of quartz, mica and feldspar minerals. The layer continues under the dam structure and forms also the left steep bank (see Figure 6). The metamorphic rock is sheared and thus its strength is considerably reduced. The lamination is parallel to the steep slope of abutment (ICOLD, 1974). Figure 7 shows the surface at the dam site after the water eroded the weathered rocks after the failure event. It can be clearly distinguished between the reddish conglomerate on the left side, and the grey schist on the right, as well as a sharp interface between the two formations.

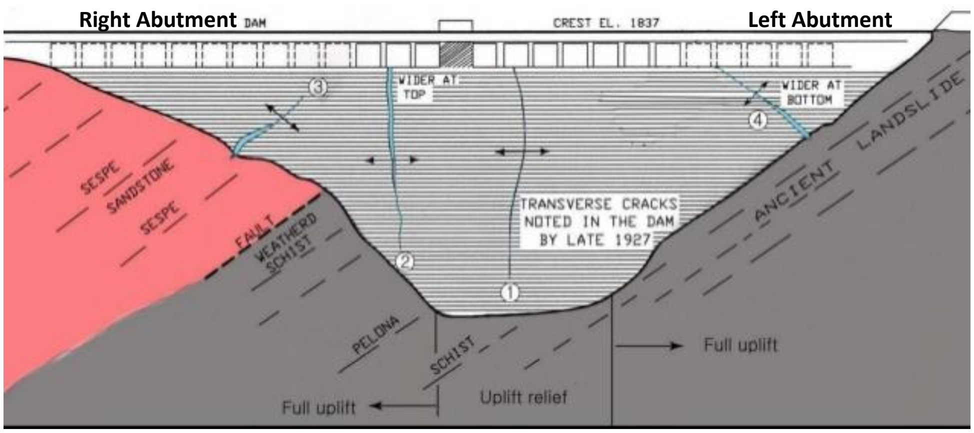


Figure 6: Cross section of St. Francis dam looking downstream (Hartley, 2016)

Part of regional geology are the paleolandslides displayed also in the cross-section in Figure 6. The ancient landslides take place in the Pelona Schist Formation. Proof of their existence are the exposed topographic benches developed in the Sierra Pelona Ridge surrounding the reservoir which are relicts of large landslide grabens. These sudden movement of masses would have blocked the San Francisquito Creek at various locations which promoted the development of a less steep valley where trees can grow and fluvial sediments can be deposited on the creek's floor (Rogers, 2013). In the geologic map from Figure 5, we can observe this Quaternary alluvium material that is heavily deposited in the creek (the yellow layer). The ancient landslide can be clearly observed from Figure 8, where the curved sliding surface can be depicted, as well as the graben formed in the heavily foliated Pelona Schist.

Figure 7: Picture of the Dam site after failure looking downstream (Rogers, 2013)

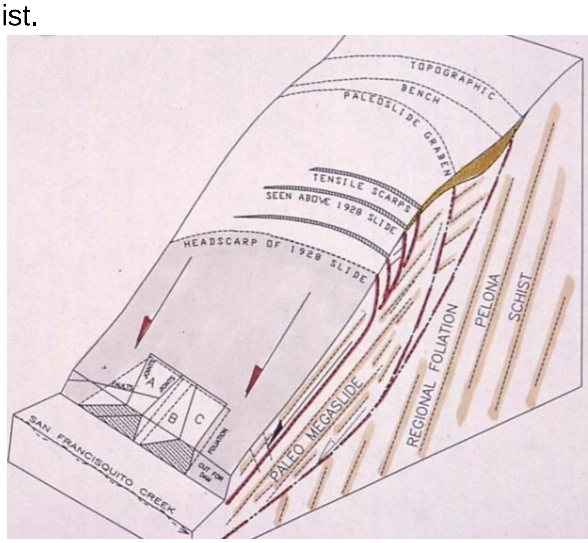


Figure 8: Left Abutment structure with the paleolandslide (J.D. Rogers, 2013)

3.3 Interpretations

The most cited investigation is the one requested by the Governor CC Young which consists of a team of 4 engineers and 2 geologists (Rogers, 2006). They explored the site on the day following the incident, then in five days, the investigation report was complete. Thus, the *"Report of Commission to Investigate the Causes leading to the Failure of the Saint Francis Dam Near Saugus, California"* finds a rather trivial explanation on what happened and what went wrong in the engineering project.

The investigation presented to the Californian Governor stated that the red conglomerate underling the dam's right abutment was unsuitable for a dam foundation and that the failure began in that area, along the old San Francisquito fault (Rogers, 2013). Their interpretation is that the Sespe Formation softened when saturated, allowing seepage at the right side of the structure. The fault was considered inactive, thus fault movement would not be the reason for the dam failure. (ICOLD, 1974). However, the initial failure of the right abutment triggered the landslide on the opposite side in the Pelona Schist, as it freed one of many slip planes formed due to foliation. Moreover, last minute changes of the design were made during construction, which increased the height by 11 percent without modifying the base width, thus drastically reducing the factor of safety against overturning (Rogers, 2013), a possible explanation why the construction was washed away.

However, there are many reasons to believe that the chain of event was slightly different than the one presented by the official commission. J. David Rogers, formerly at University of California-Berkeley and now at Missouri University of Science and Technology (Hartley, 2016) showed using the modern techniques of the 1980s why the St. Francis Dam actually collapsed:

- Firstly, the reservoir had been impounding water for 2 years, since the construction of the dam finished. Thus, saturated conditions were long before present at the site location. Thus, it is unclear why the commission decided to lay the blame of the collapse on the unconsolidated sandstone slacking very suddenly, because in practice the material had been slowly dissolving for more than a year prior to the catastrophic event.

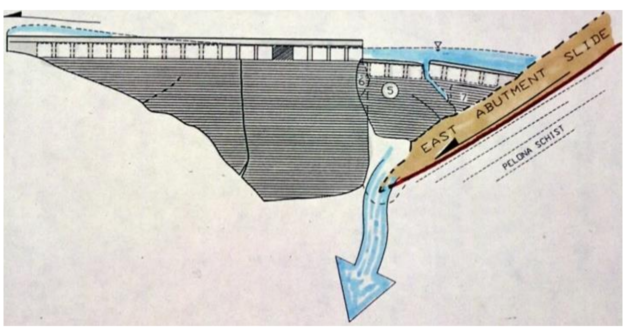
- Additionally, Rogers also pointed out problem with the electricity line that got interrupted the night of the incident. The power towers were connected to Hydropower No. 1 and from the next day inspection on the site, one picture is the proof that the commission misjudged one important aspect. The only towers still standing are those on the right abutment, while the opposite bank is totally collapsed due to the landslide (so there are no remaining power towers standing). Also, by putting together the time when the electricity was interrupted and the location of the missing towers missing, it is clear that first the left bank was the one that collapsed and stopped the power line. This is the first clue which indicates that chain of catastrophic events might be different than the one officially reported.

- Finally, a witness testified that while he was driving 3 hours before the incident on the road on the felt abutment, he realized that the road slid 30 cm down the hill. This is another indication that the landslide already started on the left bank and that could have caused the disaster. Also, another clue is that the dam keeper and his girlfriend were found dead all dressed up among concrete blocks down the valley after the event. This proves that they were not in the house, but next to the dam, looking for something. It can be speculated that they came to inspect the dam in the middle of the night as a response to the warnings concerning landslides received from the people passing on the road.

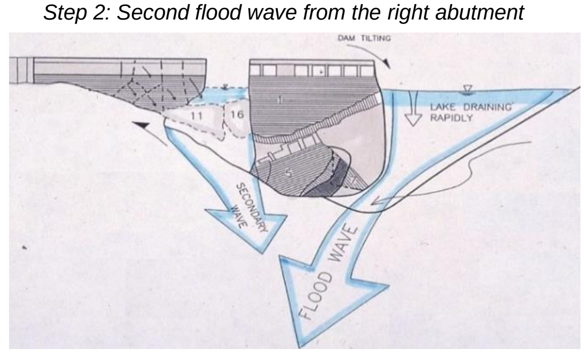
From all of the above arguments, Rogers concluded that the failure could have happened due the landslide on the left abutment first, which is in direct contradiction to the findings of the Enquired Commission. It is also known that for the past two years; several cracks were formed in the dam structure (also included in the cross-section in Figure 6) that would have led to the instability of the construction.

All in all, the chain of events that led to St. Francis Dam failure can be depicted from Figure 9 which illustrates the interpretation given by Rogers. The incident started with a small landslide of 30 cm before the event which created uplift at the bottom of the east abutment slope and allowed high velocity water to create an orifice. The schist got easily eroded by the concentrated orifice flow and a massive landslide happened in the left abutment (step 1). The middle part of the foundation was the only part of the structure anchored in the subsurface, however, since the left side of the dam had been flushed downstream, the middle block started to tilt and the water entered through another shrinkage crack on the western side of the dam (step 2). Thus, a secondary wave broke through the cement structure at the base of the right abutment and the weakened structure, crumbled and disintegrated as it was not anchored to the bed neither. In the end, only the middle block is left in place, slightly tiled clockwise (step 3).

Step 1: Land slide in the left abutment



Step 2: Second flood wave from the right abutment



Step 3: Ruins of the dam found after the failure

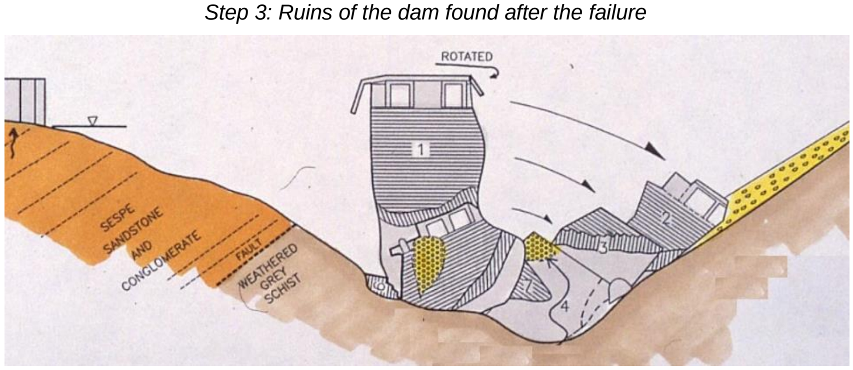


Figure 9: Sketch with the sequence of events which led to the rupture of Saint Francis Dam (Rogers, 2013)

Founding the construction on paleolandslide slope without any reinforcements caused a lot of problems which have great consequences on the project. The main body of the dam (the one that remains standing) had uplift relief valves, while the abutments did not exhibit any and thus allowed the water to infiltrate and create an overpressure. Additionally, until 1945, most engineers assumed that concrete was sufficiently impermeable (and dry) to resist complete saturation, and that dams founded on low permeability strata would not be subject to hydraulic uplift (Rogers, 2013).

The commission requested to investigate the failure also concluded that the cement was in satisfactory condition. The design was rather ambitious, as the plans were changed last minute, e.g. the height of the dam was raised by 11% without increasing its base width (Rogers, 2013), however the chef engineer did not account for their consequences. The concrete gravity dam was a solid structure, however there were multiple cracks in the cement remarked before the incident, all of which were considered by the engineers non alarming (Rogers, 2006). Furthermore, the design for Mulholland Dam (built 2 years before) was the only reference used to design to the St. Francis dam (Rogers, 2013), both looking similar in terms of size and shape. This proves that very dam is unique in its own way, and an engineering project which turned successful on one site, can be a total catastrophe placed on different grounds.

3.4 Conclusions and lessons learnt

Therefore, from all the interpretations made by different commissions investigating the causes which led to the rupture of St. Francis Dam, it can be concluded that there was a totally inadequate foundation for the dam. In the design and construction phase, no geological advise was requested by Mulholland and his team (Stapledon, 1976), thus no information on the suitability of the foundation. No core tests were taken, and no geological mapping was undertaken (Rogers, 2013). All the interpretations therefore agree that the failure was due to weakness of the foundation, however, tilting and sliding was the ultimate cause of failure of St. Francis Dam.

With regards to the designer and chief engineer, William Mulholland faced criminal prosecution and willingly took responsibility for the disaster (Rogers, 2013). The lessons learnt from this incident is that in the safety of a complex engineering project should not be left to the judgement of one man, but to a team of experts which should communicate with each other. There should be also a mandatory input of geological features prior to choosing the location and the design which should explicitly state the suitability of foundation.

Lastly, the engineers learnt another lesson about the importance of hydraulic uplift. The danger of uplift in gravity dams was recognised since 1882 in the design of Vyrnwy Dam in the UK, while in the U.S.A. Olive Bridge Dam finished in 1911 was the first to include drains in both the structure and foundation. Arizona Code in 1932 was the first to include a rule about how to make a design of a gravity dam by accounting for the hydrostatic static pressure (Thomas, 1979). A debate about the effects of the uplift under a dam was continued until the 1950s. However, the destabilizing impacts of the hydraulic uplift on the steep sided abutments was not fully appreciated until the Malpasset failure (Rogers, 2006).

4. Case study: Malpasset Dam

The Malpasset Dam case study is a great example of how engineers did not assimilate the important lessons from previous incidents (such as the St. Francis gravity dam), as they thought that arch dams were not sensitive to uplift effects due to their slenderness and the quality of the rock foundation. Approximately 30 years after the Californian structure collapsed, the French experienced a catastrophic geotechnical disaster themselves with the Malpasset Dam. Nevertheless, the origins of this incident are much more complex and yet again, the engineering team encountered one of their greatest concerns: uncertainty in the geology at the site location. This chapter will focus on explaining the causes behind another great dam failure, with a particular attention given to why it happened despite having knowledge on previous similar accidents.

4.1 Overview Failure

Malpasset Dam was situated in south-east of France, near the city of Cannes, please refer to Figure 10 for the exact location. Its completion year was 1954, and designer Andre Coyne was the man behind this 66 m high thin double arch concrete dam. The purpose of the dam was to provide a permanent supply of water to the city of Fréjus and for agricultural irrigation. The structure was quite conventional and at the time, it was considered to be one of the safest designs (see Figure 11 for a picture of the dam), with typical structural elements such as spillway and spillway apron, outlets and inlets, wingwall and a concrete buttress on the left abutment. Andre Coyne wrote down in his class notes that the abutments are the most important part of the construction, and he considered that if they stand up, the arch dam can categorically withstand anything (Goodman, 2013). The dam is located in the Provence Alps region on river Le Reyran, having the A8/E80 La Provençale motorway intercepting the river's valley downstream, road which was under construction when the dam was completed.

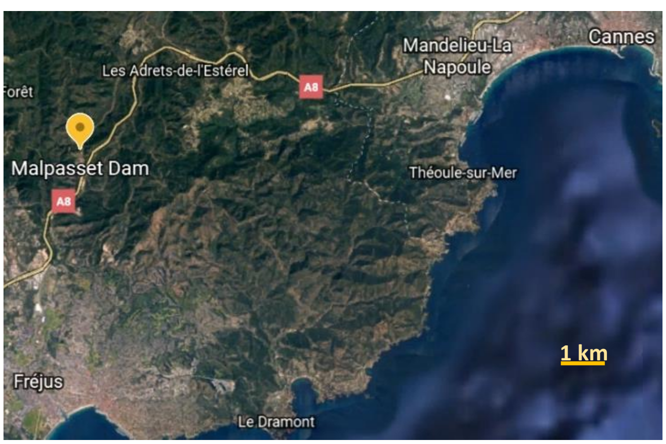


Figure 10: Location of the Malpasset Dam, France ("Google Earth," n.d.-b)

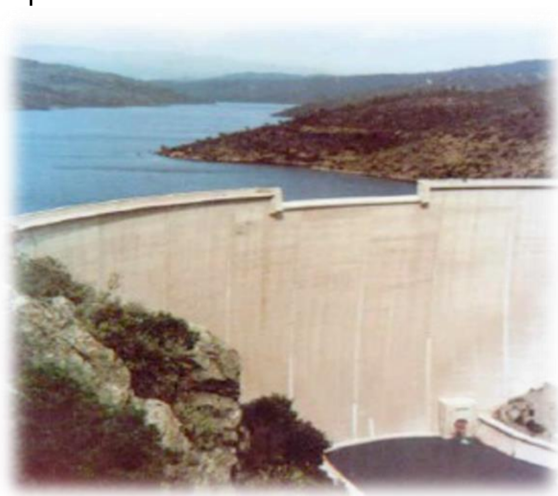


Figure 11: Malpasset Dam in 1954 (Goodman, 2013)

On 2nd of December 1959, at 9 pm, the so proclaimed indestructible piece of engineering, gave away. The storage was 30 cm from being full for the first time (Thomas, 1979), consequence of several heavy rain days. Unimaginable impact followed this failure: the loss of 400 lives, destruction of the highway construction site, several small towns and the destruction of Fréjus city, and on top of all of this, 400 million euros debt at today's value for the French government

(Luino, 2010). The dam cracked, then an enormous wave escaped, travelling downstream at high velocity. The whole arch was wiped out instantaneously, and the exact moment of the collapse is recorded by the interruption of the electricity lines crossing the valley (ICOLD, 1974). Several pre-disaster events need to be noted. In November, cracks were observed at the spillage concrete apron, but no measures had been taken. Additionally, on the day of the collapse, a team of engineers who inspected the dam remarked an almost full reservoir for the first time in its history. The last 4 m of water were filled in very fast by exceptional rains in one day, which caused a chock effect to the construction and its foundation (Duffaut, 2003). The engineers considered how to release the water without damaging the highway bridge in construction just downstream, so they only decided to open the outlet valve 3 hours before the disaster stroke (Goodman, 2013).

4.2 Geology

This major dam hazard is due almost entirely on geological causes (Walters, 1962). As a result of this, special attention will be given to the geology of the area in order to clarify what went wrong and how a more thorough site investigation could have prevented this collapse. From the regional geology, it is known that the dam is situated in the Provence Alps, in a synclinal Carboniferous zone enclosed by metamorphic horizons of the base of Massif de l' Esterel (Walters, 1962), an area with intense tectonic movement. The geological map enclosed in Figure 12 shows that the dam was founded in an area of Rhyolite flows and Carboniferous rocks, known as the Houiller du Reyran formation. The gneiss formation (Gneiss du Tanneron) overlies much of the site merges with the carboniferous sediments downstream in the valley (Jaeger, 1979).

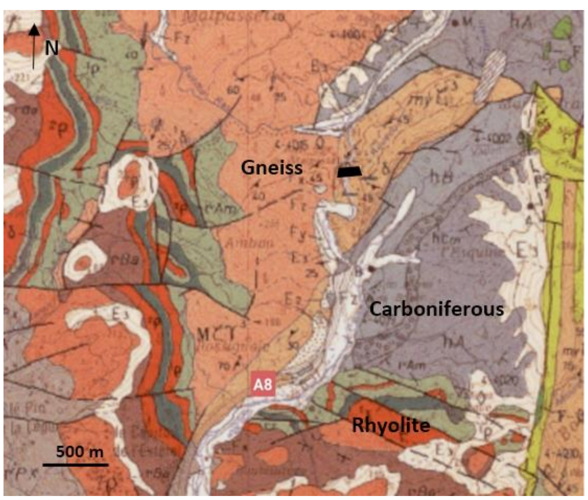


Figure 12: Geologic Map in the part of the Provence Alps, where the Malpasset Dam is located ("BRGM | FRENCH GEOLOGICAL SURVEY," n.d.)

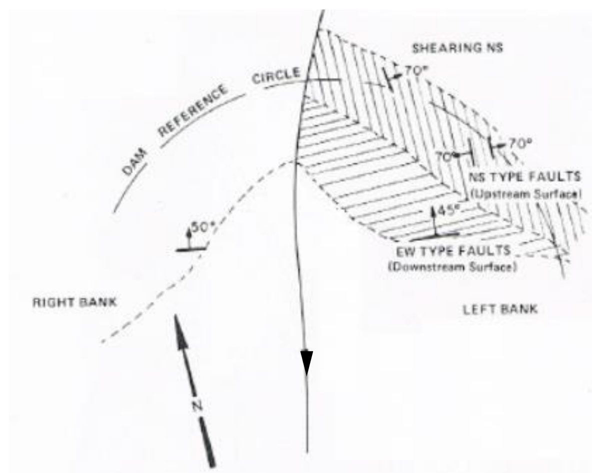


Figure 13: Geological Sketch of dam area (Goodman 2013)

Local geology comes with a comprehensive description of the rocks at the site. In general, banded gneiss and schist of Pliocene age are exposed at surface at the site location and the primary rock present is augen gneiss, phyllite rich, poor in feldspar and with a schistose structure. The schist is prominent on the left bank and lower part of the right side with the foliation dipping downstream 40 degrees into the right abutment. The rocks exhibit minerals such as sericite and chlorite (Goodman, 2013), which decreases the mechanical strength of the rock. The sericite schist has unconfined compressive strength tested in laboratory ranging from 30 to 50 MPa in contrast to the in-situ measurements of 1 MPa rock mass strength (Thomas, 1979). This difference can be explained by looking at the anisotropism of the rock. Primary anisotropism is due to the origin of the material and secondary anisotropism owes to the development of fractures and solution alteration, weaknesses which are not accounted for in laboratory tests (Wahlstrom, 1975). The mass of rock is very seamy and jointed with minor faults of 30 cm thickness, unfortunately undetected during site investigation (Thomas, 1979). Other studies confirm the very high deformability of the rock mass in comparison to other dam built near Malpasset: ten times less than that on most sites, hundred times less than that on best sites (Duffaut, 2013b).

Professor Corroy from University of Marseilles was the geologist responsible for the preliminary site investigation. He remarked several noteworthy features such as pegmatite intrusions in the gneiss (which turned to be less alarming than his forecast); the downstream dipping layering on the dam site; thin cracks in the gneiss in the foundation area, for which he suggested extensive grouting (Jaeger, 1979). From literature, it is said that amphibolites such as gneiss or mica schists, may be considered sound for sustanining bearing pressure and water tightness. However, when these rocks are associated, a weak zone of desintegrated rock may form at the interface of gneiss and mica schists (Walters, 1962).

Without being predicted before the accident, the collapse exposed various significant geological features of the rock mass at the site. On the left abutment, a large open book shape gash of width 40 m and depth 30 m, named the dihedral, appeared in the rock showing two almost perpendicular walls (ICOLD, 1974). The description of this triangular wedge of rock is essential to the understanding why the disaster happened. Both Figures 13 and 14 show the missing monolith. Figure 13 illustrates a 2D sketch with the main directions of faults near the dam and in particular, the zone of interest, the dihedral, and clearly distinguishes between the upstream and the downstream walls.

The downstream surface of the dihedral coincided with the face of a fault which belongs to the most recent type of east-west faults, with an irregular thickness of clayey breccia gouge up to 80 cm (Goodman, 2013). Please refer to the left picture of Figure 14 for the downstream wall (D arrow), pointing to the seam that is breaking out to the surface at an angle of approximately 45 degrees. The fault has a strike which can be traced across to the opposite bank, parallel to the foundation of the dam, as its thickness remains less than 1 m (Duffaut, 2013a) . The fault had no morphological feature visible from the ground surface before the dam failure. Moreover, no gouge was found in the site investigation boreholes. It could be that borehole quality was not as high as nowadays. The rock shows fractures (see figure 14, right picture), it is easily breakable and mechanically unable to resist sliding due to the presence of sericite.

The upstream wall of the dihedral is a structure which shows a great number of potential shear surfaces (foliation structure) dipping from upstream to downstream, almost parallel to the layering of the schist, and tangent of the arch (ICOLD, 1974). The rock is coated with fine mylonite and shows fractures which form blocks of big size (see arrow D in Figure 14) and it is clear that the rock had undergone a lot of pressure (Goodman, 2013), please refer to Figure 14, right picture.

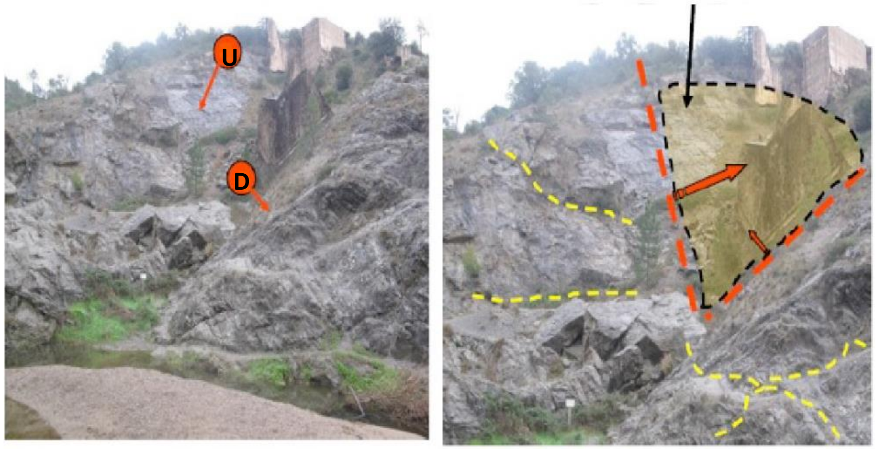


Figure 14: Dihedral shape gash in the left abutment appeared after the collapse (Goodman, 2013)

4.3 Explanation of failure

Keeping in mind the geology at the site as exposed after the failure, an investigation on the cause of failure of Malpasset Dam can be made. The Report of the Commission of Enquiry set up by the owner of the dam, the French Ministry of Agriculture, stated that the most probable cause of the disaster was the slippage of the rock mass on the left abutment due to the presence of an undetected fault. The already high deformability of the foundation was locally increased by the presence of this slip plane (Thomas, 1979).

As a more detailed clarification on how was this possible, the structure was unable to adapt to the increased deformability, so both the concrete buttress and the arch dam shell would fail in this order. The Commission also explains that the force which ruptured the root of the lateral wing wall plus the resistance of the abutment itself is far greater than the total arch thrust, making the arch move relative to the rock (Jaeger, 1979). Because the shear planes found in the upstream surface were almost parallel to the tangent to the arch, the dam forces, instead of spreading out in the foundation, remained concentrated generating high compressive strength (ICOLD, 1974). The concrete shell of the dam is a rigid body and could not be deformed as such, thus the required thrust was meant to be transmitted to the rock in the abutment, which soon became overloaded. Thus, the first phase of the rupture was actually the redistribution of the forces in the dam shell which could have lasted days, and then later a more instantaneous collapse of the dam took place (Jaeger, 1979).

After the collapse, the left abutment had been shifted 2 m horizontally with little vertical movement. The base of the dam and the right bank rotated almost 80 cm without the structure breaking up (ICOLD, 1974). As a consequence of this, a large crevasse opened between the concrete and the rock (Duffaut, 2013a), depicted at the base of the right abutment. The post disaster picture of the dam (Figure 15) exhibits only the base and the part of the right side of the dam as a standing staircase.

Figure 15: Malpasset Dam site after the failure (Goodman, 2013)

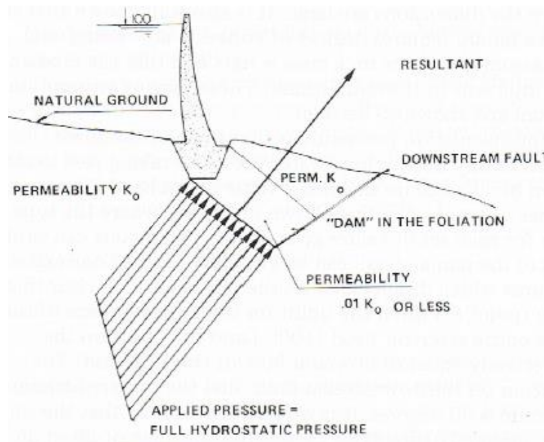


Figure 16: Failure of the Malpasset Dam (Goodman, 2013)

However, the commission did not explain how exactly the slippage happened. Thus, later investigators provided with solutions that relate the implications of the dam construction to the local geology. Under load, highly jointed schists show large decrease in permeability. Dam thrusts of 2.5 MPa were sufficient to create a less impermeable surface under the dam foundation (see Figure 16, where area under the dam has permeability 0.1 K_0). Thus, hydrostatic forces were building up against the '*underground dam'* and the previously mentioned dihedral reacted by creeping upwards and downstream (Thomas, 1979). Grouting was carried out during construction, but stopped, because the material would not absorb enough quantities, which prove the imperviousness of the foundation material.

Moreover, while the dam reservoir was filling in, the water exerted a hydrostatic pressure on the dam and on the foundation rock downstream, but not on the upstream side due to its lack of tensile resistance. A slot opened and deepened, helped by the favorable orientation of the rock foliation and the high deformability of the terrain. The extension at the upstream face provided a direct connection to the reservoir. Figure 16 illustrates that the resultant of the dam thrust and the hydrostatic pressure resultant could cause rapid sliding out of the wedge (Thomas, 1979). Thus, in this way, the arch was uplifted, was able to rotate and finally crack from its right abutment.

4.4 Conclusions and Lessons Learnt

The Malpasset Dam failed due to several factors. Inadequate site investigation led to incomplete knowledge on the geology of the area. Thus, a sound concrete structure failed due to instability of the abutment. The collapse could have been prevented, however the lack of professionalism of the team did not predict the instability of the abutment.

The first conclusion for this case study is that due to underfunding, preliminary site investigations study was not done properly. Jean Goguel explains in his report on the impossibility to study the fracturing on the rocks due to their lack of exposure at surface. He was concerned that he would not find the mylonite in gneiss even after numerous exploratory wells (Thomas, 1979), partially due to lack of quality of drilling at the time. These core samples could have raised the alarm when choosing the dam location and proven that the Reyran Valley is an area with intense tectonics movements which requires extra caution when choosing a design.

Secondly, certain events in the life of the dam encouraged water to infiltrate and soften the block of rocks. Perhaps, certain blasting operation for construction of the highway or the heavy rains which proceeded the disaster may have caused the seepage (Walters, 1962). Goguel also attributed the failure on the poor mechanical strength of the sericite rich gneiss in a zone where the stresses of the dam were extremely high (Thomas, 1979).

Thirdly, priority was set wrongly, as the team prioritized the preserving the bridge foundation freshly poured downstream. The priority should have been to minimize of the risk of dam failure by releasing sufficient water, a risk with disastrous consequences on human life and properties.

Also, as mentioned before, the 'underground dam' which appeared in an area of almost impervious rocks in the foundation, caused the uplift and sliding of the dam. The presence of the dihedral facilitated the redistribution of the forces in the dam shell and created a rotation in the dam structure clearly visible after the failure. Hence, this is how a geological feature such a slip plane or a material with low strength in combination with the effect of water seepage could led to the instability of the foundation.

Last but not least, there are a couple of essential aspects to be mentioned about the learnings from this geotechnical disaster. First and foremost, in situ strength and deformability measurements are imperative (it was proven that the material in the left bank was strong enough to withstand bearing pressures, but unable to carry the calculated dam thrust (Walters, 1962)) and that certain mechanical qualities of the rock should be specified aforetime, for instance the sensitivity of the rock mass to blasting vibrations. Additionally, monitoring is also essential, as in case of the Malpasset Dam, it was limited to displacement readings, recorded regularly but processed with a certain delay. Data recorded before the failure and processed after the dam failure showed that the terrain was moving significantly. This should have raised the alarm and prioritize water release in the reservoir to preservation of the bridge foundations downstream.

Furthermore, the hydrostatic uplift pressure ought to be taken into consideration as a fundamental design parameter. Due to large area on which the uplift acts, the force reaches an index sufficient to lift up the entire construction. Such powerful uplift was in case of Malpasset Dam, a combination of weakness of the rock and behavior of gneiss which became almost impermeable when compressed and unable to withstand the full hydrostatic pressure corresponding to the water level of the reservoir (ICOLD, 1974). Provided that grouting is inefficient, drains directed upstream would reduce the excessive pore pressure. Following the Malpasset Dam failure, many engineers worldwide have indeed used this technique and it has been proved to be very efficient against the uplift pressures.

A counter example for Malpasset tragedy is the Vajont Dam in Italy, which also experienced a terrible accident four years later. A catastrophic landslide near the reservoir created a massive wave of 100 m which overtopped the dam and killed 3000 people who were living downstream (Stapledon, 1976). The geology at the site is a sequence of massive and bedded dolomitic limestone, with Dogger Malm Formation as riverbed overlain by Cretaceous limestone, intensively jointed (ICOLD, 1974). The design of the dam was sound and this resulted in a standing and little damaged dam after the incident. The exceptional stability of the structure suggests that the dam was not overloaded and it was skillfully anchored in the abutments (Jaeger, 1979), something which Malpasset much needed.

5. Case study: Baldwin Hills Reservoir

The last case study refers to the Baldwin Hills Reservoir, which gave away despite the efforts of the engineering team to take extra safety precautions in order to foresee any unwanted incidents. A well-designed structure and great awareness of the geology were yet unfortunately insufficient to guarantee the reliability of the structure. This chapter will thus demonstrate how earth movements can bring down a solid structure, concluding with remarks on what could have been improved in order for the prevention of such disaster.

5.1 Overview Failure

Baldwin Hills Reservoir was situated in the southwest of Los Angeles, southern California, U.S.A. (see Figure 3 for the exact location). The reservoir was completed in 1951, and operated as anticipated for 12 years. The reservoir was built by curving a small basin on a hilltop in the Baldwin Hills. The storage facility had rhomboid shape and was confined on three sides by compacted earthen embankments, and on the fourth northern side by Baldwin Hills Dam of 40 m height (see Figure 17 for the shape of the reservoir) (ICOLD, 1974). The construction was owned by the Department of Water and Power, and like St. Francis, it had the purpose to provide water for the city of Los Angeles. The Inglewood oil field adjoined the reservoir on the south and southwest and occupied an oval area which extended diagonally across the trend of the Baldwin hills along the axis of the Inglewood Fault (Hamilton & Meehan, 1971), which it will be later extensively discussed. The proximity of the reservoir to the oil field and to the main fault will have serious impacts on the longevity of the structure, see Figure 21 for the location of the oil field and the Inglewood Fault.

With respect to the design of the reservoir, the engineers decided on a flexible structure with a watertight lining and an elaborate system of drains which would keep the foundation dry at all times. Thus, here we can mention layers such as 75 mm bituminous concrete, overlaying a 4 m thick drainage layer of compacted earth lining of clayey and sandy materials. Under it, a drainage layer of 100 mm of "peagravel" consisted of a network of drainage conduits which lead to an inspection chamber. Lastly, the floor of the foundation of 5 mm layer of bitumen was found. (Stapledon, 1976). Please refer to Figure 18 for the visualization of the structure.

When put into service in 1951, the dam was considered a model of engineering excellence in design, construction methods, and monitoring systems (Leonards, 1987). On the 14th of December 1963 the failure occurred. The caretaker heard the unusual sound of running water in the spillway discharge pipe while making his daily inspection. The toe drains and underdrains system were discharging muddy water, thus the decision to drain the reservoir was shortly made. Next, a seepage crack was discovered on the downstream slope of the northern dam, please refer to Figure 23. Immediately, evacuation measures were initiated as the flow was steadily increasing and only four hours later, the Baldwin Hills Dam breached (ICOLD, 1974).

Consequences of the disaster were minimal compared with what would have occurred if no warning had been provided, despite this, the damage consisted of five lives lost, $12 million property, and loss of the reservoir itself (Hamilton & Meehan, 1971). The investigation that followed the disaster was conducted by the State Engineering Board of Inquiry and a consulting board (ICOLD, 1974). The crack can also be visualized in Figure 18, which coincides with one of the faults, as will be later explained, and Figure 17 shows the crack on the northern abutment as the water was rushing downhill

Figure 17: Aerial view on the Baldwin Hills Reservoir when it collapsed (MacDougall, n.d.)

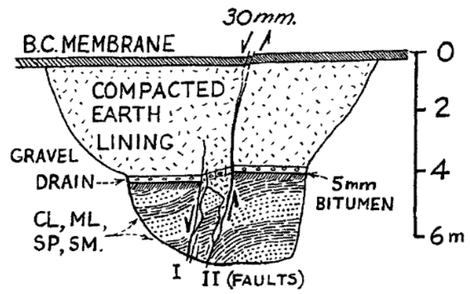


Figure 18: Foundation of the Reservoir after the failure (Stapledon, 1976)

5.2 Geology

This incident is another example of failure due to geological reasons. Therefore, it is of the uttermost importance to define the environment of depositions of the sediments, as well of what type of geological formations which are found in the Baldwin Hills region. Regional geology shows that Baldwin Hills Reservoir is found in an area of low hills, the highest of the Newport-Inglewood ridge (ICOLD, 1974) which rise in striking contrast to the surrounding flat terrain of the Los Angeles basin. Ground movements such as folding and faulting have contributed to the uplift of this chain (Hamilton & Meehan, 1971). The hills are underlain by near-horizontal sediments of late Tertiary and Quaternary age. Some of the older sediments are slightly consolidated, however, the majority of materials are sands, silts and clays, readily erodible and of low density (Stapledon, 1976). The environment of sedimentation is marine and sea-shore of Pliocene and Pleistocene origin, with the layers generally dipping 5 to 7 degrees (ICOLD, 1974).

Figure 19 shows the geological cross-section which illustrates the stratigraphy in the Baldwin Hills. At the surface, the soil mantle is clayey silt for less 1 m, overlaying another thin layer of alluvium layer, a mixture of silt, clay, sand and gravel. The first formation encountered is the Palos Verde from upper Pleistocene, about 20 m thick, fine to coarse sands occasional interbedded with silts or soft sandstone (Hudson & Scott, 1965). It is highly erodible, proof given by the deep gullies found next to the reservoir (ICOLD, 1974). Below Palos Verde lays Inglewood Formation which dates from lower Pleistocene. It is 30 m thick and is composed of thick fine sands, interbedded with silts and clays (Hudson & Scott, 1965). This formation ranges from hard rock to loose, as shown by the powdery sands and silts which were eroded and exposed at failure (ICOLD, 1974). The deepest formation is Pico Formation dating from upper Pliocene which is the thickest (more than 500 m) and composed of silt, very fine sands and clay interbedded with soft siltstone (Hudson & Scott, 1965). It is more massive than the overlying formations and less erodible than Inglewood formation (ICOLD, 1974).

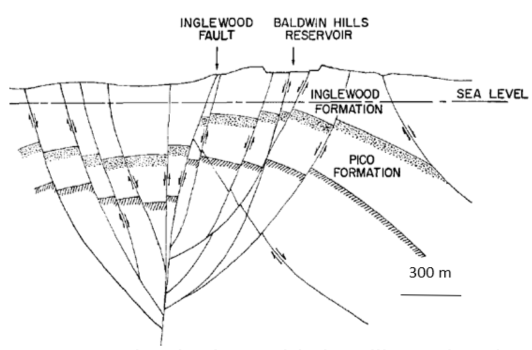
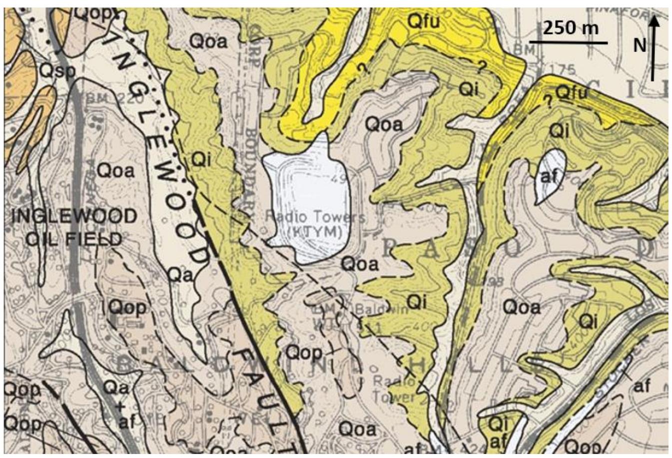


Figure 19: Geological Cross section in the Baldwin Hills region (Hudson & Scott, 1965)

The interpretation of the layers in the Baldwin Hills area is confirmed by looking at the geologic map administered by the U.S Geological Survey (USGS) in Figure 20. We can identify the '*Qoa'* to be the Palos Verde formation, upper Pleistocene, slightly consolidated and very erodible. The Inglewood Formation is exposed at the surface near the site and it is depicted as '*Qi*' in the geological map. The last layer that the reservoir encounters on the northern face is '*Qfu'*, which is described by the USGS as the Upper Fernando Formation. From literature, we know that the upper Fernando formation is from the same age as the Pico formation, Pliocene and Pleistocene (Blake, 1991), thus it can be assumed that both the cross section and the geologic map may refer to the same layer. Moreover, Blake (1991) provides numerous cross sections with the stratigraphy in the Los Angeles Basin and it is clear to see that around Inglewood fault (referred to as the Newport Inglewood fault), the material found at the surface is composed of Quaternary sediments overlaying the Pico formation. The other interpretation would be to assume that the Upper Fernando Formation is part of Inglewood, thus only Quaternary material is exposed at the surface in the northern embankment of the Baldwin Hills reservoir.



Legend

- **Qoa:** *Older alluvium* sandy gravel, Upper Pleistocene
- **Qi:** *Inglewood Formation*, finer grained sandstone with beds with soft grey siltstone, weakly consolidated and eroded, early Pleistocene
- **Qfu:** *Upper Fernando Formation*, part of Inglewood, soft grey massive siltstone, shallow marine sediments, weakly consolidated, eroded, early Pleistocene
- **Qop:** *Paleosoil*, erosion resistant, Upper Pleistocene
- **Qa:** *Alluvium* sand clay gravel, unconsolidated, surficial sediment
- **Af:** *Artificial Fill*

Figure 20: Geological Map of the area near the Baldwin Hills reservoir situated in the center (Dibblee & Ehrenspeck, 1991)

The Baldwin Hills Reservoir lays on top of the primary anticlinal fold structure which has been modified by faulting, especially by lateral and dip-slip displacement along the Inglewood fault, which bisects the hills (Hamilton & Meehan, 1971) and it is part of the four principal faults in the Newport-Inglewood uplift (ICOLD, 1974). The reservoir is about 300 m from this major fault, and several minor faults were found to cut through the site, during site investigation (Stapledon, 1976). These near-parallel, north-striking faults that splay outward from the Inglewood fault south of the Baldwin Hills Reservoir were probably formed as an array of tear faults developed in response to strike-slip displacement along the dominant Inglewood fault (Hamilton & Meehan, 1971).

Fault number I was discovered during construction and named the Reservoir Fault, please refer to Figure 21 for the visualization of the location of the faults. Reddish-brown clayish gauge of 0.3 to 10 cm thickness, fresh slickensides (which implied that the fault was active) and cementation of silts and fine sands were found along the fault (ICOLD, 1974). Horizontal displacement could not be recorded, however since completion, a vertical displacement of about 20 cm for fault I and 8 cm for fault II (Hamilton & Meehan, 1971) was established, classifying the faults as normal or extensional faults.

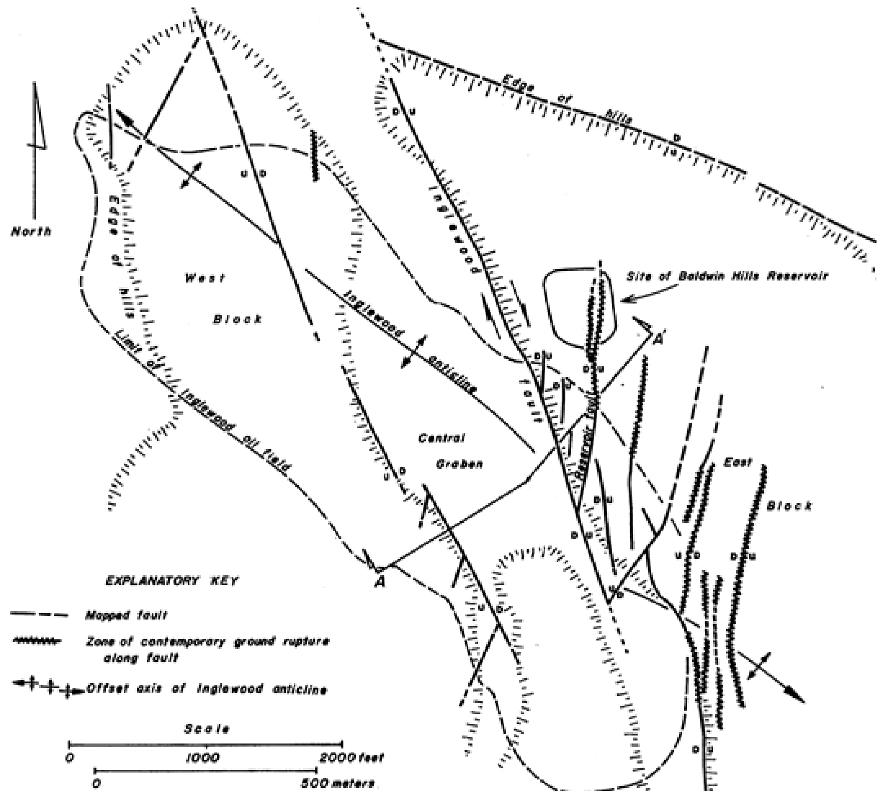


Figure 21: Map of Baldwin Hills and the main geological structures (Hamilton & Meehan, 1971)

5.3 Earth Movement Analysis

Having a clear view on type of rocks found under the foundation, the reasons behind the earth movements recorded at the site location need to be discussed. Ground rupture and earth cracks were observed adjacent to the reservoir after completion. Possible causes for the deformations are separated in two categories:

- Related to the Inglewood Oil field activities such as hydrocarbons withdrawal, operation activities, fluid injection (in 1954, the Standard Oil Company started a secondary program to recover the remains in the eastern part which implied injection of brine) (Hamilton & Meehan, 1971).
- Tectonic origin such surface rupturing along active faults (Hamilton & Meehan, 1971).

The first cause for ground movement is subsidence, which has been drastically accelerated due to the proximity of the oil gas field. The Los Angeles Department of Water and Power recorded the ground movements between 1910 and 1964 and the regional subsidence decreases with increasing distance from the center of the oil field. Thus, a so call subsidence bowl of radius of 800 m was created (ICOLD, 1974) and, at the location for the reservoir, a subsidence of more than 1 m was recorded at the end of the mentioned period.

To be more explicit, the surface rupturing occurred at the edge of the subsidence bowl. This stretching of the rim is an mechanical consequence of the subsidence (Hamilton & Meehan, 1971), and cracking of material found under tension will inevitably happen. Subsidence also exerts a downwards movement of sediments in the bowl, and where faults or other weak surfaces are present, elastic strain will be relieved by upward movement of the ground at the edge (Hamilton & Meehan, 1971). Please refer to Figure 22 for a visualization of this theory which explains that the outer parts of the subsidence bowl will lift upwards, whilst the center compresses due to depletion of the hydrocarbon accumulation.

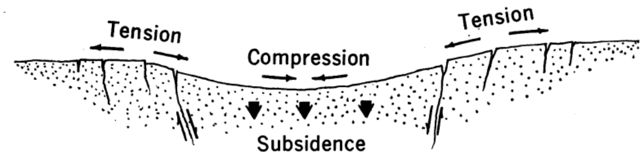


Figure 22: Fault activation due to subsidence in the Inglewood Oil Field (Hamilton & Meehan, 1971)

Apart from subsidence, differential compaction of the material of each side of the fault as a result of reservoir loads needs to be considered as well. The penetration-needle tests carried out on each side of Fault I by the Department of Water and Power after the failure are not conclusive with respect to the relative softness or stiffness of the material. Other researches such as those made by Casagrande, interpreted that material on the downthrown side of the fault is highly fractured, broken and fissured and less stiff than the material on the upside of the fault which is intact (Scott, 1987). Settlement records indicated that consolidation both of the dam and foundation was completed by about 1955, which implies that there is no reason for differential compaction of the material across the faults to continue throughout the whole life of the reservoir. Thus, only differential settlement is there to continue for years after the reservoir failed and was emptied, and can no longer be attributed to the compaction of the soil adjacent to the fault (Scott, 1987).

This differential settlement could trigger a fault movement; however, it is unclear why the fault activation happened exactly on the day of the failure. Therefore, there was probably a gradual culmination of effects that built up over the years. Most probably, one or more cavities, formed during the operational years of the reservoir, broke through the foundation and led to the failure (Scott, 1987).

Moreover, another theory speculates that there is a remarkable coincidence between the crack growth rate below the reservoir and the sequence of injection episodes relating to the oil field nearby (Hamilton & Meehan, 1971). These changes in volume and in strength of soil with fluid-filled voids are shown to be fundamentally related to change in stress within the solid skeleton. An increase in the effective stresses accompanying fluid withdrawal and a decrease in fluid pressure due to hydrocarbon extraction clearly lead to subsidence. Thus, the fault gets activated through the response of stressed ground to artificially induced fluid pressure in the ground (Hamilton & Meehan, 1971).

Figure 23: Initial crack formed in the embankment (on the left) and the development of the crack until collapse (on the right) (DHS and FEMA, 2002)

5.4 Conclusions and Lessons Learnt

The Report produced by the State Engineering Board of Enquiry stated that the failure is due to the development of displacement in its foundation. Having eliminated the potential cause of an earthquake induced movement, the fault system under the reservoir is hold responsible for the failure, and it is considered to be the weak spot for the structure (Thomas, 1979). Moreover, the commission concluded that what triggered the earth to move was the subsidence which was happening in the region for a while. The acceleration of the cracks opening could be due to the fluid injection used in the oil field in order to increase the yield of the reservoir. Regional movements, differential compaction of the foundation strata due to reservoir loading, and slow seepage of water from the reservoir into the underlying erodible soils contributed to the failure.

The exact explanation of this event is that the porous concrete drain was damaged by the early small movement of the fault and seepage along the fault started. During the life of the reservoir, erosion was taking place in the fault under the bitumen blanket, while the fault permitted the porous concrete drain to widen the openings that were developing. However, this was a rather gradual process, and the failure only happened when there was full pressure from the reservoir on the fault. At the moment of failure, the flow and erosion increased exponentially, a cavernous opening formed under the foundation, facilitating the embankment to collapse into this opening (Thomas, 1979).

As a conclusion, the safety of the reservoir depended on preventing the water to get into the faults zone and the soil under the foundation (Leonards, 1987). A smaller reservoir would have been a much safer reservoir in an active earthquake fault zone (Scott, 1987) and a steel lining could have extended the life of the structure.

Therefore, there are numerous aspects which could have prevented the failure to happen and are lessons to be learnt from this incident. An estimation of settlement (non-existent at the time of the design), calculation of amount of seepage and possibility to relate it to the number of drain pipes used in the actual design of the foundation, or to the time it takes to make it through the drainage system (Scott, 1987) are just a few measures which could have foreseen the disaster. Additionally, it is important not only to have an inspection and a surveillance system but also a prior determination of course of action whenever the measurements hit pre-determined critical values. Thus, a more advanced monitoring system should be have adopted that would warn of the impeding danger way ahead in time to avoid the imminent failure followed by a decisive emergency plan in case the recorded values would reach the potential hazardous limit (Leonards, 1987).

6. Conclusion

In conclusion, based on the initially posed questions, the following remarks can be drawn. Below, in Table 3, there is an overview of the main causes of geotechnical disasters and the corresponding lessons to be learnt with respect to the three main case studies.

Table 3: Summary of the three case studies

CASE STUDY	REASONS OF FAILURE	LESSONS LEARNT
ST. FRANCIS DAM	❖ INADEQUATE FOUNDATION ❖ UPLIFT, SLIDING ❖ LACK OF PROPER SITE INVESTIGATION	❖ MANDATORY GEOLOGICAL INPUT ❖ MORE THAN ONE PERSON SHOULD CHECK THE SAFETY OF THE STRUCTURE ❖ IMPORTANCE OF UPLIFT
MALPASSET DAM	❖ INSTABILITY ABUTMENT ❖ UPLIFT, SLIDING ❖ CONCENTRATED STRESSES UNDER THE LOAD OF THE DAM IN HIGHLY ANISOTROPIC ROCKS	❖ COMPULSORY IN-SITU MEASUREMENTS ❖ REDUCE UPLIFT BY USING DRAINS DIRECTED UPSTREAM EVEN IN ARCH DAMS ❖ INCREASE AWARENESS ON THE HIGH SENSITIVITY OF PERMEABILITY IN FOLIATED ROCKS TO PRESSURE
BALDWIN HILLS RESERVOIR	❖ DISPLACEMENT OF FOUNDATION ❖ SUBSIDENCE ❖ DIFFERENTIAL SETTLEMENT	❖ BETTER CALCULATION OF SETTLEMENT AND SEEPAGE ❖ MONITORING SYSTEM WITH COURSE OF ACTION PLAN IN CASE OF EMERGENCY

All of the above historical cases are emblematic events which show that there were serious implications with regards to geological features in engineering projects. Whether or not the engineering team took into account the regional and local geology, definitely influenced the longevity of the project. By all counts, the geology at the site location along with the availability of the materials are clear indication of the feasibility of the construction. Other factors such as hydrological, human and geographical could also affect the outcome of a project. The value of these unfortunate experiences is that they provide a legacy of knowledge to future generations of engineers, as remarked by an international publication:

The combination of ageing dams, retirement of experienced dam engineers and increased consequences of dam failure due to downstream development underscore the need to better ensure the future safety of dams worldwide. Past experience must not be forgotten and lessons learned must be captured for future generations. ("Lest we forget: learning from international dam incidents - International Water Power," 2010)

Therefore, there is no doubt that, at the present time, there is an ongoing pressure to ensure the safety of both old and new dams in order to foresee unwanted accidents. The remaining question in this conclusive part of the paper is with regards to the degree of safety in modern buildings of dams and reservoir. The Camará and Pirris dams are two examples of 21st century structures in South America which could answer the question whether society is building carefully enough nowadays. The former is another representative case when inadequate geological input and misguided decisions of the management team led to failure. On the other hand, Pirris Dam is a counter example which shows the professionalism of the engineering geologists who considered all the geological features in the construction of the dam and were able to prevent a disaster. Both of the cases are briefly discussed in the Appendix.

In comparison with the discussed historical cases, there are certain aspects in modern constructions which have undoubtedly improved, but some which remain an issue. The Camará and Pirris dams are built on foundations which have discontinuous beds, oriented in such a way that could facilitate the sliding of the rock mass. It is well known from the St. Francis Dam disaster that these orientations are dangerous, prone to slippage and might result in instability, as it happened when the left abutment of the Camará Dam gave away. However, the team of engineers from the Pirris Dam did understand the importance of effective communication between geologists and the design team (as highly recommended after the Malpasset disaster) and made an early change in the reinforcement of the structure which prevented the imminent sliding of the embedment when fully saturated.

All things considered, past experiences influence current geotechnical projects to a surprisingly great extent. The most common types of failure of present dams are as follows but not limited to: lack of maintenance of the structures; unpredictability of precipitations; unfortunate compromises with respects to the materials and surveillance systems due to lack of funding. And yet still, insufficient geological site investigation remains a recurring cause for dam failure in modern times, mostly in countries where less strict regulations leave this obligation open to interpretation. Maintaining dam and reservoir stability is one of the many responsibilities of our society and it is in our duty to find better ways to ensure the security of these engineering masterpieces.

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Appendix

Case study: Camará Dam

Camará Dam, located in eastern Brazil, is an originally designed embankment dam turned into a 50 m high gravity dam after the majority of field explorations had already been completed (USBR & USACE, 2015). The dam was finished in 2002, and filled up only in 2004 due to a long period of droughts followed by sudden heavy rains. During the last month of the filling, several abnormalities were observed, such as materials in the drain flows and wet spot at the toe in the left abutment. The designer recommended the reservoir to be emptied, however, the owner did take his advice and prioritized the high demand water over his recommendation. In June 2004 the dam gave away resulting in five deaths and 500 homeless (USBR & USACE, 2015).

The geology at the site consists of a base rock of gneissic migmatites with foliation dipping 30 degrees toward the right abutment (USBR & USACE, 2015). The abutment is stable during construction, but unstable under the uplift pressure due to reservoir filling up. From core sample, geologists could have recognized the weak layers which coincides with the failure plane (please see Figure 24), and on top of this, the weak plane was also exposed during construction, but was wrongly considered to be very shallow (it was excavated and filled with concrete) (Kanji, 2017).

Camará Dam failed by the foundation of the left abutment, where a layer of totally weathered biotite schist about 30 cm thick was exposed, having a friction angle of only 21 degrees (Kanji, 2017).

Figure 24: Camará Dam foundation affected by geological aspects (Kanji, 2017)

Case study: Pirris Dam

Pirris Dam in Costa Rica is a concrete gravity dam finished in 2011 (RCC Dams, n.d.) founded on highly cemented mudstone, of strength greater than 100 MPa. The rock is highly jointed and exhibits possible weak planes which could endanger the stability of the structure (Kanji, 2017). At one site visit, the team acknowledged joints 12 cm wide infilled with plastic clay parallel to stratification, with little friction angle. They have modeled the discontinuities using finite element analysis and established a Factor of Safety less than 1 in many areas (Kanji, 2017). The same as in Camará Dam case, the structure is stable during construction, but potential hazardous when the reservoir is full. The team found a solution to build a shear key in the abutment (please refer to Figure 26) and since then, the dam has been under regular operations.

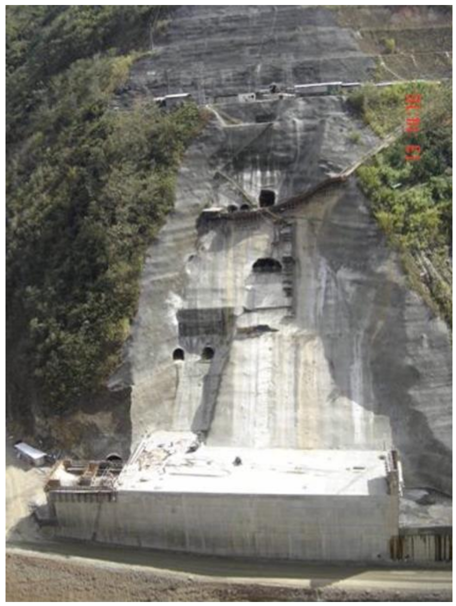

Figure 25: Shear key completed for Pirris Dam (Kanji,2017)