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Dam failures - an analytical and statistical approach

MASTER'S THESIS

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Manuel Ovalle Iglesias, Ing.

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Ass. Prof. Dipl.-Ing. Dr.techn. Helmut Knoblauch

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Supervisor:

Ass. Prof. Dipl.-Ing. Dr.techn. Helmut Knoblauch

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Statutory declaration

I declare that I have authored this thesis independently, that I have not used other than the declared sources/resources, and that I have explicitly marked all material, which has been quoted either literally or by content from the used sources.

Graz, Mai 2019 ……………………………………………………

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Abstract

Dams are hydraulic structures built to provide wellness and development in society. The failure of these structures could carry an impact on the environment, the economy and fatalities. Therefore, it is crucial a proper design, maintenance and monitoring of such important infrastructures.

The present work tries to implement the research published in the ICOLD Bulletin 99 of 1995, where an analysis of the dam failures was done by paying special attention in their height, age and mode of failure. Hence, the master thesis is focused on further analysis of the cases occurred during the period comprised between 1996-2018, taken as model the aforementioned bulletin. For this purpose, a database has been created to generate statistical information. In order to draw conclusions, a comparison is done among the own achieved results and those carried out about dam failures before 1995.

Moreover, the Austrian dam safety approach is presented and discussed as an example of a successful high international standard.

Finally, old and latest techniques and methods for dam surveillance and monitoring are examined, with special attention in the state of the art and worldwide tendencies in terms of dam safety. Furthermore, the current need of reevaluation of existing dams due to climate change and updated design methods is reviewed.

Kurzfassung

Talsperren sind hydraulische Strukturen, die gebaut werden, um zur Entwicklung der Gesellschaft beizutragen. Ein mögliches Versagen dieser Strukturen wirkt sich nicht nur negativ auf Umwelt und Wirtschaft aus, sondern kann auch zahlreiche Todesopfer fordern. Daher ist eine ordnungsgemäße Planung, Wartung und Überwachung derart wichtiger Infrastrukturen von entscheidender Bedeutung.

Die vorliegende Arbeit versucht, die im ICOLD Bulletin 99 von 1995 veröffentlichten Forschungsergebnisse umzusetzen, bei denen eine Analyse der Ausfälle des Staudamms unter besonderer Berücksichtigung von Größe, Alter und Art des Ausfalls durchgeführt wurde. Darauf aufbauend konzentriert sich die Masterarbeit auf die weitere Analyse der im Zeitraum 1996-2018 aufgetretenen Fälle, wobei das vorgenannte Bulletin als Vorbild diente. Zu diesem Zweck wurde eine Datenbank erstellt, um statistische Informationen zu generieren. Um Rückschlüsse zu ziehen, werden die eigenen erzielten Ergebnisse mit denen verglichen, die vor 1995 zu Dammversagen geführt haben.

Darüber hinaus wird der österreichische Dammsicherheitsansatz als Beispiel für einen international erfolgreich hohen Standard vorgestellt und diskutiert.

Abschließend werden alte und neue Techniken und Methoden zur Überwachung und Kontrolle von Staudämmen untersucht, wobei dem Stand der Technik und den weltweiten Tendenzen im Hinblick auf die Sicherheit von Staudämmen besondere Aufmerksamkeit geschenkt wird. Darüber hinaus wird der derzeitige Bedarf einer Neubewertung bestehender Staudämme aufgrund des Klimawandels und aktualisierter Bemessungsverfahren überprüft.

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1. Introduction

1.1 Goals and aims

The purpose of this master thesis is to implement the "Bulleting 99 of ICOLD about Dam Failures Statistical Analysis" [1a] launched in 1995. The thesis research tries to provide a further overview, adding more data for a comprehensive statistical approach and mainly focused on the failures occurred after 1995, which are not covered in the mentioned bulleting. Providing, in addition, a quantitative analysis as well a surveillance and monitoring methods used, in order to prevent or avoid further failures according with the available data and lessons learned.

For the present work, a database has been created, collecting dam failures occurred between the period 1996 – 2018 from several sources as scientific papers, webpages of different national institutions and universities, as well as diverse researches which are mentioned. A total amount of 191 cases has been documented and processed (see Appendix) distinguishing between large dams according the ICOLD definition. Due to the different degrees of detailed information founded and inputted into the database, a series of graphs and information were generated and extracted, as long as the quantity of the data was representative enough to be able to provide reliable conclusions.

The types of dam that are the object of study and analysis are embankment, gravity and arch dams, because are the kind of hydraulic structures which have both a higher economical and safety impacts in society and also as it was already mentioned, the ICOLD Bulletin 99 was taken as a base reference. Tailing dams are not covered in the database due to the reasons explained in the corresponding subchapter.

The present document is structured firstly in a part where the functions and types of dams studied are defined and described. Secondly, an overview and conclusions about former studies and researches about dam failures until 1995. Thirdly, when the dam failures occurred and its reasons with an analytical and statistical approach using the created database. Fourthly, about the lessons learned. Fifthly about risk management and safety aspect in the Austrian regulations. Then, the traditional and new technologies for monitoring and upgrade measures as a consequence of climate change are looking over. Finally, the conclusions are discussed.

1.2 Function of dams

1.2.1 Hydropower

In a hydropower dam a traditional layout composed with an upper reservoir where the water is stored by a dam, the potential energy which the mass of water storage is used to generate electricity by turbines and the height difference between the upper reservoir and the lower reservoir or stream.

The power that could be generated is according the basic following formula:

$$P_e = \rho \cdot g \cdot \eta_t \cdot \eta_g \cdot \eta_m \cdot Q \cdot H$$

P_e: power in Kw

ρ: density of fluid in kg/m^3

g: gravity acceleration m/s^2

η_t: efficiency hydraulic turbine

η_g: efficiency elec. generator

η_m: efficiency turbine alternator coupling

Q: flow rate in m^3/s

H: height difference in m

Figure 1. 1: Hydropower arch dam (iagua, 2017)

1.2.2 Regulation and flood control

The dams constructed with the aim of regulation and flood control, are built in order to manage extraordinary flood events and they are usually constructed in the river or stream basins to derive, retain or store the water runoff. Releasing downstream the volume with a flow that the system is allowed effort in a safety manner like the retention basins.

With these types of dams in addition tocontrolling flood events, it is also possible to manage the water levels and a sustained run in case of drought events keeping the ecologic flow in a natural regimen.

1.2.3 Water supply for human consumption and irrigation

The rainfall of a catching area is collected and stored in dams or reservoirs and later used to supply the water when it is necessary, in this way, the water supply does not depend on seasonality or weather. Therefore, with this kind of dams it is achieved safety in the demand, supply and supporting an increase in the productivity, development and transformation of the agricultural areas and increasing the wellness of all the stakeholders.

1.3 Types of dams considered

1.3.1 Arch dams

In arch dams, the shape distributes the load radially and homogeneously, that implies that these dams work mainly axially. The arch transmits the compression stresses to the abutments into a competent rock. Therefore, there are two main conditions or limitations for the design and placement of these structures:

- High bearing capacity of the abutments or flanks
- A relative symmetrical “V” shape in narrow canyons

The arch dam may have a single or double curvature in vertical. There is also another subtype called arch-gravity dam that is an alternative when there are doubts concerning the bearing capacity of the flanks as results of the stresses and thrust produced by load transmitting to the flanks. In order to solve it, the body of the dam is wider than normal arch dams, aiming that the self-weight of the arch-gravity dam helps to resist the stresses.

Figure 1. 2: Arch dam (theconstructor.org)

1.3.2 Embankment dam

The embankment dams are very versatile because they can be built of a great variety of materials, therefore, are widely spread around the world and each kind of material is placed where according to their characteristics, they fulfill a proper function. The typical section is trapezoidal and they are thicker than the other dam types.

The earth or soil is built up by compacting layers. The impervious materials are usually placed in the core and the more permeable ones are placed upstream and downstream sides. The impermeable material also can be placed upstream side of the dam. It depends on the typology of embankment dam. It is common that a riprap is built up to prevent erosion due to the wind or rainfall events. Composed by a wide concrete spillway usually on the crest.

The self-weight helps with the stability and to resist the forces. Nevertheless, the structural behavior is different from the gravity dams.

Usually these dams are constructed where the bearing capacity of the ground is not that strong due to their foundations requirements are not that stringent than other type of dam. In addition, they are built in wide valleys with not stiff abutments. The height of the dam dependent on the characteristics of the foundation materials.

There are two main categories of embankment dams:

- **Earth-fill embankment dams**

This kind of embankment dams is the most common and are those which contain more than 50 percent, by volume, earth-fill materials. They are designed as a non-overflow section, otherwise it would lead to a failure with a separate spillway. Some features are:

- The foundation requirements relatively light
- Local soil used as construction material
- Relatively easy to build. Technology and machines required

Earth-fill dams can be classified in two different ways.

Based on the construction method:

- Rolled fill earth dams
- Hydraulic fill dam

Based on material characteristics:

- Homogeneous earth dams
- Non-homogeneous earth dams (with inclined impervious zone with artificial material o with soil with low permeability)

- **Rock-fill embankment dams**

The dams, which contain more than 50 percent rock-fill material by volume. They consist mainly in two parts:

- Impervious core
- Pervious rock-fill supporting outside for support

They can be classified in:

- Diaphragm rock-fill dams
- Central core rock-fill dams

1.3.3 Gravity dam

Gravity dams work with the friction between the dam's body and the ground. The dam's self-weight stables the structure against sliding and tipping considering all the acting forces.

They are built of concrete mass or stone masonry, except in specific points where exist heavy stresses like galleries. They are designed to hold back large volumes of water. These dams work by compression and the traction forces must to be checked carefully.

There are two main construction typologies. Vibrated concrete and rolled compacted concrete (RCC).

In general, the best configuration regarding resistance is a triangular/trapezoidal section with a curve shape in plan. Usually, the dam base takes around the 80% of the area or volume of the whole dam body.

Gravity dams suit in wide valleys. They rely on their its self-weight to hold back water and consequently they need a solid foundation bedrock.

1.3.4 Others (tailing dams-overview)

The tailing dams are typically embankment dams built with the aim of storing the resulting product or waste of mining operations. Tailing could be in different states of in a mix of them (solid, liquid or slurry).

The failures of tailing dams are relatively common and even, nowadays, with a huge negative impact in life and environment costs.

It is not the purpose of the present thesis to present and analyze tailing dam failures because of the great amount of those events occurred and mainly due to specialized current literature about it like ICOLD 2001 Bulletin 121 and the

CSP2 with detailed information of the tailings dam failures among the years 1915-2016.

2. Worldwide dam failures – overview until 1995

2.1 Reasons

2.1.1 Large dams and failures

The ICOLD defines a large dam as „ A dam with a height of 15 meters or greater from lowest foundation to crest or a dam between 5 meters and 15 meters impounding more than 3 million cubic meters "A dam failure is defined by ICOLD 2015 as „Collapse or movement of part of a dam or its foundation, so that the dam cannot retain water".

A series of failures due to the mode of operation are presented and discussed. By earthquake, design failures, concrete deterioration and hydrological reasons which include flood events, spillway scour, overtopping and piping-seepage.

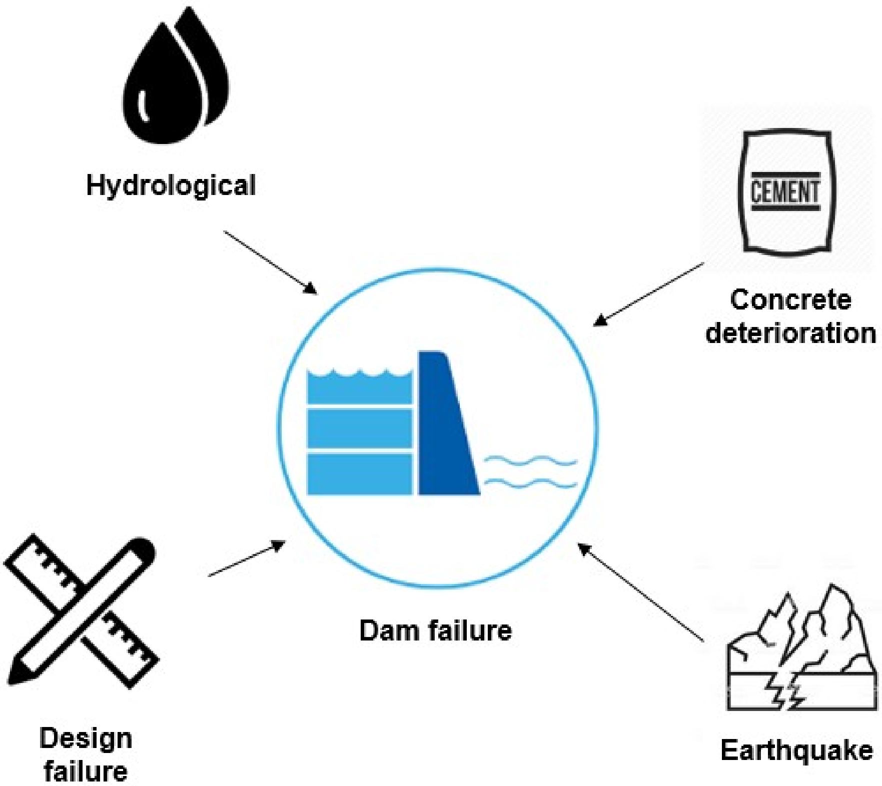

Figure 2. 1: Failures by mode of operation

The existence or not of correlation between the failure of a dam and its age and height is also evaluated through analysis of database and historical dam failures being representative enough to get conclusions.

The figure 2.2 shows the probability of failure of a dam and its year of operation and therefore, the existence of a relationship, according the age of the dam and the risk of an incident depending in which age stage is the dam.

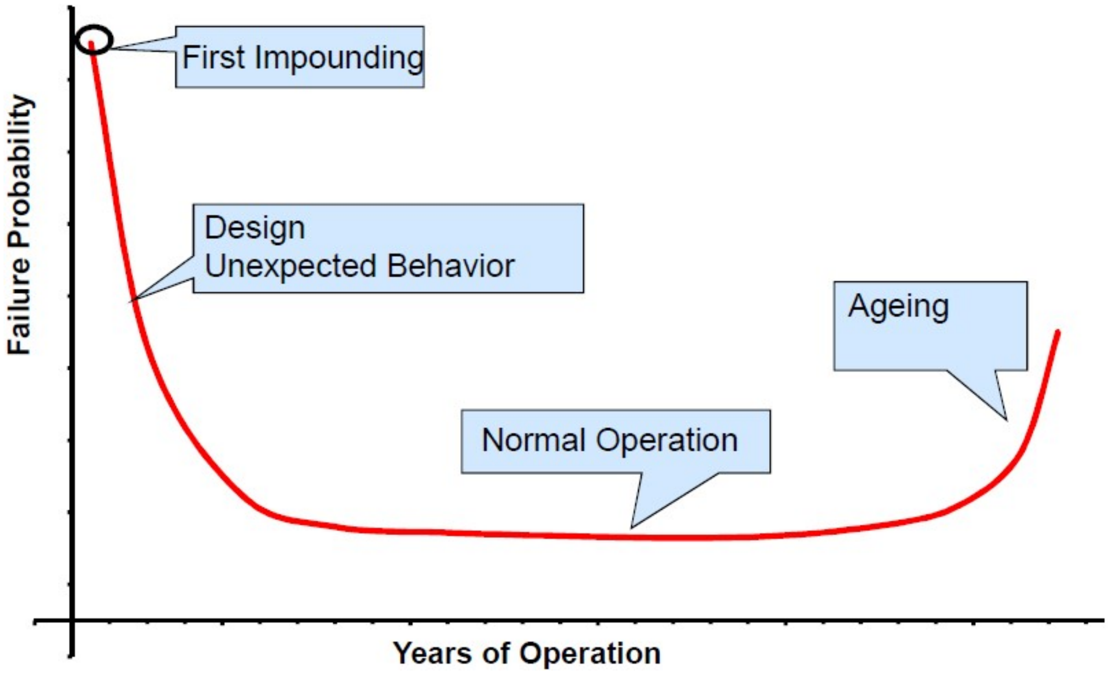

Figure 2. 2: Probability of failure along the time (R. Melbinger, 2018)

2.1.2 Earthquake

One of the reasons of dam failures are earthquakes. Nevertheless, these structures in general, present a low rate of failures, even when the old ones have not been designed to resist seismicity or the method used is considered nowadays obsolete. Although, new regulations and many expert committees have published measures to increase the security level in case of earthquake events such as ICOLD Bulletins 62 (1988 rev. 2008); 72 (1989 rev. 2010); 112 (1998); 113 (1999); 120 (2001); 123 (2002); 137 (2010) 148 (2010).

Approximately earthquake has been responsible or involved of around 2% of dam failures worldwide [1b]. Those failures include the structure by itself or the associated facilities such as power intake, penstock, powerhouse, fish ladder, etc. Mainly they have occurred in Asia, to be more specific in China, Japan or India.

The most common hazards associated to earthquakes are [2b] [3b]:

- Ground shaking produce vibrations and structural distortions, added facilities and foundations.
- Fault movements or discontinuities in the dam foundation causing structural distortions.
- Fault displacements could cause water waves in the reservoir or affect the freeboard.
- Rockfalls and landslides may cause damage to gates, spillway piers (cracks), remaining walls (overturning), electro-mechanical equipment and other facilities.
- Mass movements into the reservoir may cause impulse waves in the reservoir.
- Mass movements can block rivers or forming landslide dams that in case of failure can lead to overtopping or facility damages.
- Ground movements and settlements due to liquefaction, densification of soil and rockfill causing distortions in dams.
- Abutments movements causing sliding and distortions.
- Seiches or reservoir oscillations, caused by resonances of water that have been triggered by seismic activity are less important for dam engineering.

According the dam type, the behavior facing earthquakes is different. In general, arch dams, rock fill dam with clay cores and concrete rock fill dams have better response against earthquakes. Consequently, they are more suitable for seismic areas than concrete buttress dams, that have not good response with regard earthquakes. At middle way, the concrete gravity dam could show horizontal cracks towards the crest on the up and downstream faces.

2.1.3 Design failure

In the cases of dam failure due to design reasons, there are several causes that involve the failure mechanisms. Usually, not only a single factor in a poor design is the cause of an incident, it could come from a combination factors. Nowadays, the standards worldwide have developed to a high security parameters combined with the computer simulation of the structures and its responses. However, in former times it was not always like this.

The main reasons for design failure are [1c] [2c]:

- Substandard construction materials, the use of low quality construction materials is a factor of dam failures. The use of the right construction materials and with certified quality is critical. Gleno dam in Italy is one example, in 1923 where a wrong mortar was used and improper reinforcement as well.

- Poor maintenance, dams require regular and frequent maintenance in order to ensure in safety operation along their life cycle. Regular maintenance and inspections must to be carried out by qualified professionals and right design of the maintenance plan. One example is the Val di Stava Dam in Italy, where in 1985 the structure collapsed due to lack of proper maintenance.

- Outdated designs, dam design has improved over the past century, but in many cases due to financial reasons, the existing dams cannot be upgraded. In some cases, with a simple better weather forecasting, dam's operators can be better prepared to face adverse situations.

- Design error, these kind of failure occurs when essential factors are not incorporated into the construction of dam design, like the right monitoring. For example, a lack of a proper system for gauging the water level in reservoirs which could lead to an overflow.

- Changes in land use, for agriculture, residential areas, or other economic activities imply that there is an impact in the way of how the water is infiltrated or it reaches the dam. The flow rate that the dams have to deal nowadays may have changed since they were estimated.

- Changes in weather patterns, shifting weather patterns can put more strain on dams, as some areas get wetter while others get drier. Designing for 100 or 500 year floods has been an imperfect process, due to the understanding of weather and climate is always improving. Time ago, in order to calculate, some historic trends or patterns were used to predict behaviors and calculations, and it was not always the best way to face some problems.

2.1.4 Concrete deterioration

The concrete deterioration can be caused by physical and/or chemical processes that may lead to a dam failure due to the

progressive reduction of concrete or cement properties. The most known ones are described below [4b]:

- Freezing and thawing, due to the internal disruption of the concrete paste caused by the formation of ice crystals in saturated or almost saturated concrete. These phenomena usually occur in zones with extreme weather or mountains. There are cases where the freezing and thawing (F/T) deterioration could reach up to 100 cycles in a year. The cracks are the result of the dilation of the water when it becomes ice, with an increase of volume of around 10%. The dams built before 1945 are susceptible to F/T, where usually the material fail after 200 cycles, afterwards, it was discovered that adding tiny air bubbles the concrete protection increase significantly until 1000 cycles.

- Alkali- Aggregate reactions, is a chemical reaction between the alkalis in cement and reactive aggregates also present, which generate a kind of gel that in contact with water expands significantly and therefore, produce cracks in the concrete. The deteriorations include swelling and cracking of the concrete due to the loss of strength and elasticity modus.

There are two types. On one hand, Alkali-Silica-Reaction (ASR) if the concrete has an alkali content greater than 0.6% and glassy siliceous volcanic rocks or potentially deleterious rock types like chert or opal. On the other hand, Alkali-Carbonate-Reaction (ACR) usually associated with limestone aggregates. Nowadays, in order to avoid such reactions low-alkali cements and pozzolans are used.

- Sulfate attack, produce an attack both chemical and physical in the internal microstructure of the concrete due to the presence of sulfates in soil or groundwater. It produces an expansive disruption in the concrete which lead to cracks. There have been reported cases of failures in less than 5 years.

  The chemical attack is a reaction between sulfates and cement hydration products that produces expansions and dissolution of the concrete paste. Furthermore, physical attacks by sulfate are a consequence of crystals precipitation within the cement matrix and disrupting its internal structure. With the introduction of sulfate resistant cements, most of the sulfate attacks were eliminated after the 40's.

- Abrasion- erosion and cavitation damage, are the result of a physical wearing of the concrete by sediments, gravels and rock that are carried by the water in concrete with low strength and poor aggregates that may sweep rocks and sediments from downstream back to into the spillway and outlet works stilling basins, resulting in particles abrading the surface in a roller-mill fashion.

  Cavitation damage could be as a result of the formation and collapse of water vapor bubbles on the concrete. It could occur in spillways and outlets due to the high-velocities of the water. A proper design and high-strength concrete are necessary to avoid these damages in the spillways and in the case of outlets the cavitation can be the consequence of insufficient air supply to gates or in the way that the gates are operated.

2.1.5 Flood events (hydrological)

There are many different paths of how a dam could fail as consequence of flood events, the main ones are briefly described.

- Flood events- spillways scour, from the hydraulic view, a spillway system are subject to scour during extreme discharge flows. A breach failure could be the result of scour and erosion on the structure or abutment. Scouring downstream of a spillway can be triggered by a hydraulic jump.

- Overtopping, as a consequence of extreme flood event, which can lead to a total or partial collapse of the dam. One factor of dam failures by overtopping were the use of not appropriate hydrological methods to estimate extreme flood events and, therefore, the discharge design for spillways and outlets.

  In the case of embankment dams, which are the most susceptible type of dams to fail by overtopping, they are not designed in general to resist the erodible action of water flow over the crest. The water flows along the dam downstream with the hazard of eroding and generate uplift pressure in the embankment that with the time lead to a breach.

Depending on the magnitude and duration of overtopping, different scenarios can occur [5b]:

- Long flows, the entire overflow might be infiltrated. If the event has a short duration, there will be hardly any rise in the saturation line.

- When the overflows increase more rapidly than the infiltrating flow, water run along the dam slope. Free surface flow will progressively erode the slope and the dam toe.

- For an event of long duration, infiltration will rise the saturation line, producing internal erosion and shallow surface sliding. The destabilizing uplift pressure generated by the seepage flow will produce an embankment slide.

In addition, the result of overtopping can cause a loss of freeboard due to the dam and foundation settlement, seiches of the reservoir water level and waves as product of failure of adjacent natural slopes

- Piping-seepage, it is one of the most common reasons of dam failure. The excess hydrostatic head can increase the seepage velocity along conduits increasing internal erosion to adjacent dam conduits. Piping occurs when water removes soil particles and transport them to unprotected exit, developing unseen channels through the dam body or its foundations. When the upstream or the intake end of the eroded hole approaches the bottom of the reservoir, the failure occurs. The seepage pressure is produced by the friction between the percolating water and the walls of the voids as dragging. Piping can be prevented by constructing a weighting berm or a pressure relief well, which means, loaded filter above the area in which the seepage emerges from the ground [5b].

2.2 Failures by mode of operation

2.2.1 During construction

During construction the dam is more vulnerable. For example, due to the diversion works or that the structure has not developed the designed total bearing capacity.

Those types of failures, usually cause significant damage at the works already executed and adjacent facilities. They also imply a high potential of fatalities.

One example is the case of Oros dam in Brazil in the 60’, where during construction a flood event happened and the dam was overtopped around nine hours with a result of dam collapse and approximately 100 fatalities. The designed was carried out without many concern thought the passing flood during its construction which should be really considered in areas where heavy rain seasons are frequent.

The database used to identify the failures was mainly from ICOLD Bulletin 99 and from Appendix 1 of Bulletin 109. Collapses occurred in China or USSR are not included. As the Bulletin stated at that moment, a statistical analysis was complicated for failures during construction (except for overtopping), due to the different positions regarding information supplied by the national Committees.

The present document has applied a simple criterion for a statistical evaluation of the database, which consist in categorize in three main groups according to the type of dam (gravity, embankment and arch dams) and its height to distinguish between large dam or not according the ICOLD definition and its analysis. In most cases, the year of construction and the year of failure match in the same year or ±1. Consequently, it is classified as failure during construction.

- Embankment dams

Table 2. 1: failures during construction. Embankment dams

Name of Dam Nom du Barrage	Country Pays	Year of Failure Année de Rupture	Height Hauteur (m)	Length Longueur (m)	Volume of Reservoir Volume de Retenue (hm^3) >$10hm^3$|	Fill Remblai	Number of Victims Nombre de Victimes
Owen	USA	1915	17	247	60	E	/
Blyderiver	South Africa	1924	21	340	2.2	E	/
Paris (X)	USA	1939	17	/	/	/	/
Cheoha Creek (X)	USA	1970	28	/	11	/	/
Elansdrift	South Africa	1975	28	600	3.2	E	10

Name of Dam Nom du Barrage	Country Pays	Year of Failure Année de Rupture	Height Hauteur (m)	Length Longueur (m)	Volume of Reservoir Volume de Retenue (hm³) >10hm³]	Fill Remblai	Number of Victims Nombre de Victimes
Waghad	India	1883	(32) 32	/	17	E	/
Goose Creek	USA	1900	(20) 64	-	/	R	/
Red Rock (X)	USA	1910	32	/	16	/	/
Little Field (X)	USA	1929	(37) 37	91	/	R	/
Puddingstone	USA	1930	55	/	22	/	/
Oros	Brazil	1961	(35) 54	620	730	E/R	/
Panshet	India	1961	(49) 51	740	214	E	1000
Hell Hole	USA	1964	(30) 125	/	2600	R	0
Mac Cloud © (X)	USA	1964	70	200	43	E/R	/
Sempor	Indonesia	1967	60	228	56	R	200
Piedras	Spain	1968	40	620	60	/	/
Odiel	Spain	1970	35	154	3.3	R	/
Toa Vaca (X)	USA	1970	65	530	68	/	/
Xonxa	South Africa	1974	(24) 48	300	158	/	/
S. Tomas	Philippinas	1976	43	/	/	/	80
Hans Strydom (C)	South Africa	1980	(18) 57	525	150	R	/

There are 21 cases reported that fulfill the criteria. All of them can be considered as large dams.

- Gravity and Arch dams

Any case registered.

2.2.2 After construction

A specific typology of failure mechanism is considered in this part, which is the first filling that is very relevant and must to be considered with special attention in embankment dams. During the first filling the water pressure and deformations play a significant role. The water level behind the structure increases and the forces vary. During the first filling, the permeable earth and rock fill zones upstream become saturated, so the water load, the compressibility core properties and downstream shoulders are susceptible to collapse. Also, there are other factors acting like time dependent deformations, such as creep and earthfill consolidation and differential settlement of the foundation [2a].

- Embankment dams

Table 2. 2: failures after construction. Embankment dams

Name of Dam Nom du Barrage	Country Pays	Year of Failure Année de Rupture	Year of Completion Année d'Achèvement	Height Hauteur (m)	Length Longueur (m)	Volume of Reservoir Volume de Retenue (hm^3) >10hm^3|	Fill* Remblai*	Foundation Fondation	Number of victims Nombre de victimes
Dale Dyke	GB	1864	1863	29	380	3.2	E	-	230
Molteno	SA	1882	1881	15	800	0.2	E	-	0
Mena	Chile	1885	1888	17	200	0.1	E	R	100
Lake Francis	USA	1899	1899	15	300	0.8	E	R	-
Bila Desna	Czecho	1916	1915	18	240	0.4	/	/	65
Ema	Brazil	1940	1932	18	370	10	E	S/R	/
Fred Burr	USA	1948	1947	16	100	0.6	/	S	/
Lower Khajuri	India	1949	1949	16	/	43	E	/	/
Battle River	Canada	1956	1956	14	550	15	E	S/R	/
Little Deer Creek	USA	1963	1962	26	110	1.8	E	R	1
Jennings Creek 3	USA	1963	1962	21	92	0.4	R	E	/
Jennings Creek 16	USA	1964	1960	17	110	0.3	R	R	/
Kedar Nala	India	1964	1964	20		17	E	-	/
Sheep Creek	USA	1970	1969	18	330	1.4	E	S	/
Manivali	India	1976	1975	18	-	4.8	E	-	/
Mafeteng	Lesotho	1988	1988	23	500	?	E	R	/

Name of Dam Nom du Barrage	Country Pays	Year of Failure Année de Rupture	Year of Completion Année d'Achèvement	Height Hauteur (m)	Length Longueur (m)	Volume of Reservoir Volume de Retenue (hm^3) >10hm^3|	Fill * Remblai*	Foundation Fondation	Number of victims Nombre de victimes
Apishapa	USA	1923	1920	35	178	24	E	S/R	/
Corpus Christi	USA	1930	1930	31	1240	79	E	S	/
Mena Teton	USA	1976	1976	93	900	300	E	S/R	11

For embankment dams that failed after construction, there are 19 cases documented in the database. One of the cases, Battle river dam with 15 m, does not fulfill the criteria of height, but on the other hand it storages 15 hm^3, therefore, is considered as large dam.

- Gravity dams

Table 2. 3: failures after construction. Gravity dams

Name of Dam Nom du Barrage	Country Pays	Year of Failure Année de Rupture	Year of Completion Année d'Achèvement	Height Hauteur (m)	Length Longueur (m)	Volume of Reservoir Volume de Retenue (hm³)	Comments Commentaires	Number of Victims Nombre de Victimes
Austin I	USA	1893	1893	18	330	/	M	?
Bayless	USA	1911	1909	16	160	1.3	C	80
Granadillar	Spain	1934	1930	22	170	0.1	M	8
Puentes	Spain	1802	1791	69	291	13	M	600
Cheurfas	Algeria	1885	1884	42	/	17	M	10
Elwha River	USA	1912	1912	33	135	9	M	0
Kundli	India	1925	1924	45	160	1.3	M	?
St Francis	USA	1928	1926	62	213	47	C	450
Ashley	USA	1909	1908	18	133	0.1	B	-
Stony River	USA	1914	1913	16	330	7	/	-
Austin II	USA	1915	1915	20	388	21	B	8
Gleno	Italy	1923	1923	35	225	5	MV	600
Komoro	Japan	1928	1927	16	96	0.1	B	7
Selford	Sweden	1944	1943	21	200	1.8	B	-
Vega de Terra	Spain	1959	1955	33	270	7.3	B	140

For this section, there are 15 cases reported.

- Arch dams

Table 2. 4: failures after construction. Arch dams

Name of Dam Nom du Barrage	Country Pays	Year of Failure Année de Rupture	Year of Completion Année d'Achèvement	Height Hauteur (m)	Length Longueur (m)	Volume of Reservoir Volume de Retenue (hm³)	Comments Commentaires	Number of Victims Nombre de Victimes
Vaugn Creek	USA	1926	1926	19	95	?	/	/
Malpasset	France	2959	1954	66	222	47	/	420
Idbar	Yugoslovia	1960	1959	38	108	1.9	/	/

For arch dams, there are 3 cases reported. One of them, is the noun Malpasset disaster, where in 1946 hydrological and geological studies shown that the location for the dam was suitable. Nevertheless, these studies and specially the geological one were not properly carried out. The lithology under the dam was composed of impermeable rock (gneiss), therefore the risk of water penetrating the ground was considered low. In 1954 the right abutment collapsed, with a consequence of more than 400 fatalities.

The reasons of the collapse where the combination of heavy rainfall with the consequence, increasing of seepage and the presence of a tectonic fault in combination with a fault and also the forces associated with the water accumulating behind the dam. All those factors caused that the gneiss along the abutment entered in a compressive state. Uplift pressure at the abutment was great enough due to the filling of the reservoir to dislodge the thrust block and producing the cracks and consequently the collapse.

2.2.3 During operation

- Embankment dams

  By internal erosion:

Table 2. 5: failures during operation. Embankment dams

Name of Dam Nom du Barrage	Country Pays	Year of Failure Année de Rupture	Year of Completion Année d'Achèvement	Height Hauteur (m)	Length Longueur (m)	Volume of Reservoir Volume de Retenue (hm³) >10hm³]	Fill * Remblai*	Foundation Fondation	Number of victims Nombre de victimes
Bilberry	GB	1852	1845	20	90	0.3	E	/	80
Cuba	USA	1868	1851	15	/	0.5	/	/	/
Mill River	USA	1874	1865	13	180	/	/	/	140
Utica	USA	1902	1873	21	/	?	E	/	0
Avallon	USA	1904	1893	17	-	-	E/R	/	/
Green Lick	USA	1904	1901	19	259	0.6	E/R	/	0
Black Rock	USA	1909	1907	21	208	18	E/R	R	/
Jumbo	USA	1910	1905	18	1200	29	E/R	/	/
Hatchtown	USA	1914	1908	18	240	15	E	/	0
Horse Creek	USA	1914	1912	17	5000	21	E/R	/	0
Hebron I	USA	1914	1913	17	1120	/	E	S	/
Lyman	USA	1915	1913	19	250	43	E	S	8
Lake Toxaway	USA	1916	1902	19	120	13	E	R	0
Leeuw Gamka	SA	1928	1920	15	540	10	/	-	0
Castlewood	USA	1933	1890	28	183	4.3	E	R	2
Lake Francis II	USA	1935	1901	24	400	2.3	E	R	/
Anaconda	USA	1938	1898	22	/	0.2	E	S	0
Sinker Creek	USA	1943	1919	21	330	3.3	E	R	/
Stockton	USA	1950	1950	28	100	0.5	/	-	/
Toreson	USA	1953	1898	15	96	1.4	E	/	/
Ahraura	India	1954	1954	26	650	61	E	/	/
Pampulha	Brazil	1954	1941	18	350	18	E	S	0
Mill Creek	USA	1957	1889	20	84	0.3	E	R	/
Alamo Arroyo	USA	1960	1960	21	-	6.6	E	S	/
Hyogiri	Korea	1961	1940	15	110	0.2	E	R	139
Smart Sindicate	SA	1961	1912	28	2800	98	E	R	/
Emery	USA	1966	1850	16	130	0.5	E	R	/
Nanak Sagar	India	1967	1962	16	19300	210	E	/	100
La Laguna	Mexico	1969	1912	17	675	4.3	E	S/R	0
Caulk Lake	USA	1973	1950	20	134	0.7	E	R	/
Wheatland	USA	1973	1960	13	2000	11	/	/	/
Hinds Lake	Canada	1982	1980	12	5200	7500	E	R	0
Kantale	Sri Lanka	1986	1869	27	2500	135	E	R	127
Quail Creek	USA	1988	1984	24	610	50	E	/	0

Name of Dam Nom du Barrage	Country Pays	Year of Failure Année de Rupture	Year of Completion Année d'Achèvement	Height Hauteur (m)	Length Longueur (m)	Volume of Reservoir Volume de Retenue (hm^3) >10hm³|	Fill * Remblai*	Foundation Fondation	Number of victims Nombre de victimes
English	USA	1883	1878	30	100	18	R	R	/
Graham Lake	USA	1923	1922	34	335	200	E	S	/
Baldwin Hills	USA	1963	1951	71	198	11	E	S	5
Walter Bouldin	USA	1975	1967	51	2268	/	E	/	0
Ghattara	Libya	1977	1972	38	217	5.5	E	S/R	0
Ruahihi	New Zealand	1981	1981	32	67000	31	R	S/R	/
Embalse Aromos	Chile	1984	1979	42	220	60	E	S/R	0

By overtopping:

Name of Dam Nom du Barrage	Country Pays	Year of Failure Année de Rupture	Year of Completion Année d'Achèvement	Height Hauteur (m)	Length Longueur (m)	Volume of Reservoir Volume de Retenue (hm^3) >10hm³|	Fill Remblai	Number of victims Nombre de victimes
Torside	GB	1855	1854	31	270	6.7	E	S
Walnut Grove	USA	1890	1888	33	120	11	R	129
Sweetwater	USA	1916	1911	35	200	54	E	/
Lower Otay	USA	1916	1901	46	170	52	R	30
Graham Lake	USA	1923	1922	34	330	220	/	/
Kaddam	India	1959	1957	40	/	215	E	G
Rincon (X)	Paraguay	1959	1945	50	1100	9000	/	/
Swift	USA	1964	1914	57	225	37	E/R	19
Dantiwada	India	1973	1969	61	137	460	E	/
Euclides da Cunha	Brazil	1977	1960	61	312	13	E	G
Sales de Oliveira	Brazil	1977	1958	40	660	25	E	U
Tous	Spain	1982	1980	77	780	50	R	G 20

Name of Dam Nom du Barrage	Country Pays	Year of Failure Année de Rupture	Year of Completion Année d'Achèvement	Height Hauteur (m)	Length Longueur (m)	Volume of Reservoir Volume de Retenue (hm^3) >$10hm^3$|	Fill Remblai	Number of victims Nombre de victimes
Blackbrook I	GB	1799	1797	28	160	0.2	E	/
Killington	GB	1836	1820	18	250	3.4	E	/
Tabia	Algeria	1865	/	25	/	/	/	/
Iruhaike	Japan	1868	1633	28	700	18	E	1200
South Fork	USA	1889	1852	21	/	18	E/R	2200
Johnstown	USA	1889	1842	22	/	19	/	G
Chambers Lake	USA	1891	1885	15	213	/	E	S
Lake Vera	USA	1905	1880	15	51	/	R	/
Chambers Lake II	USA	1907	1885	15	240	10	E	S
Wisconsin	USA	1911	1909	18	80	24	R	0
Hebron II	USA	1914	1913	17	1120	x	E	/
Lookout Shoals	USA	1916	1915	25	830	49	E	/
Mammoth	USA	1917	1916	23	/	13	E	S 1
Oversholser	USA	1923	1920	16	247	18	R	/
Oklahoma	USA	1923	1920	16	365	26	/	/
Scott Falls	Canada	1923	1921	15	250	11	R	U
Mac Mahon	USA	1925	1924	17	137	0.1	E	/
Dykstra	USA	1926	1903	15	/	/	R	/
Balsam	USA	1929	1927	18	91	?	E	/
Briseis	Australia	1929	1926	17	137	1	R	11
Wagner Creek	USA	1938	1912	15	98	0.7	E	1
Heiwaike	Japan	1951	1949	22	82	0.2	E	100
Ashizawa	Japan	1956	1912	15	/	/	E	/
Kaila	India	1959	1955	26	213	14	E	S
Kharagpur	India	1961	1956	24	/	55	E	/
Ogayarindo	Japan	1963	1944	24	100	0.16	E	/
Cazadero	USA	1965	1906	21	55	16	R	/
Ovcar Banja	Yugoslav	1965	1952	27	/	3	E	G
Wesley E Seale	USA	1965	1958	25	1804	374	/	G
Pardo	Argentina	1970	1949	15	60	0.1	R	25
Lake Barcroft	USA	1972	1913	21	62	3	E	0
Whitewater Brook	USA	1972	1949	19	137	0.5	E	/
Lower Idaho	USA	1976	1914	15	275	/	/	U
Bolan	Pakistan	1976	1960	19	530	89	E/R	20
Dhanibara	India	1976	1975	20	/	61	E	/
La Paz	Mexico	1976	/	10	1600	x	/	80
Machu	India	1979	1972	26	3900	101	E	G 2000
Gotvan	Iran	1980	1977	22	710	x	/	200
Dibbis (X)	Irak	1984	1966	17	650	50	/	/
Noppikoski	Sweden	1985	1967	19	175	0.7	E	G 0
Spitskop	SA	1988	1975	17	760	61	E	0
Bagaudo	Nigeria	1988	1970	20	2100	22	E	/
Tierpoort Dam (X)	S. Africa	1988	1923	20	116	33	/	/
Mitti (X)	India	1988	1982	17	900	19	E	0
Belci	Romania	1991	1982	18	420	12	E	G 20

Name of Dam Nom du Barrage	Country Pays	Year of Failure Année de Rupture	Year of Completion Année d'Achèvement	Height Hauteur (m)	Length Longueur (m)	Volume of Reservoir Volume de Retenue (hm³) >10hm³|	Fill Remblai	Number of victims Nombre de victimes
Torside	GB	1855	1854	31	270	6.7	E	S
Walnut Grove	USA	1890	1888	33	120	11	R	129
Sweetwater	USA	1916	1911	35	200	54	E	/
Lower Otay	USA	1916	1901	46	170	52	R	30
Graham Lake	USA	1923	1922	34	330	220	/	/
Kaddam	India	1959	1957	40	/	215	E	G
Rincon (X)	Paraguay	1959	1945	50	1100	9000	/	/
Swift	USA	1964	1914	57	225	37	E/R	19
Dantiwada	India	1973	1969	61	137	460	E	/
Euclides da Cunha	Brazil	1977	1960	61	312	13	E	G
Sales de Oliveira	Brazil	1977	1958	40	660	25	E	U
Tous	Spain	1982	1980	77	780	50	R	G 20

There is a total amount of 110 cases listed in the database, 41 of them correspond to failures due to internal erosion, only one case does not fulfil neither the height or water storage to be considered as large dam. The rest 69 cases are dam failures by overtopping which shows that the highest number of failures documented.

- Gravity dams

Table 2. 6: failures during operation. Gravity dams

Name of Dam Nom du Barrage	Country Pays	Year of Failure Année de Rupture	Year of Completion Année d'Achèvement	Height Hauteur (m)	Length Longueur (m)	Volume of Reservoir Volume de Retenue (hm³)	Comments Commentaires	Number of Victims Nombre de Victimes

Sig	Algeria	1885	1858	21	/	3	U M	10
Bouzey	France	1895	1880	22	520	7	M	100
Angels	USA	1895	-	15	120	/	M	1
Elmali	Turkey	1916	1893	23	298	1.7	F M	?
Tigra	India	1917	1917	25	1340	124	F M	1000
Elguiau (W)	GB	1925	1908	12	1000	4	C	10
Zerbino	Italy	1935	1924	16	70	10	F C	100
Pagara	India	1943	1927	27	1440	100	F M	?
Chikkahole	India	1972	1966	30	670	11	F M	?

Fergoug I	Algeria	1881	1871	33	300	30	FM	200
Fergoug II	Algeria	1927	1885	43	300	30	FM	0
Mohne (war) (X)	Germany	1943	1913	40	/	134	M	1200
Eder (war) (X)	Germany	1943	1914	48	400	200	M	100 ?
Xuriguera	Spain	1944	1902	42	165	1.1	FM	7
Khadakwasla	India	1961	1879	33	1400	137	UM	1000

Rutte (X)	Italy	1965	1952	15	/	0.3	MV	-
Leguaseca	Spain	1987	1958	20	70	0.1	MV	7

There are 17 cases reported of gravity dams that failed during operation, all of them large dams.

- Arch dams

Probably the most famous case is the failure of Vaiont in Italy in 1961. An arch dam with 262 meters of height.

Around 2000 people died as a consequence of a 25 m height wave above the dam crest product of the overtopping due to a massive landslide of about 260 millions of cubic meters. A lack of proper investigation and how it was used (negligence) about the reservoir slopes were the reasons of the catastrophe.

2.2.4 Warfare

In present times, there have been many concerns about some dam failures due to act of wars or associated reasons in conflicts occurred in the Middle-East like Irak and Siria as well, as some cases in Africa and South Asia, but luckily no case has been reported.

Nevertheless, before 1995 the cases happened during the second world war, as part as the strategy from the Royal Air Force (UK) to damage some vital german infrastructure during the conflict. It led to air raids that produced the breached of dams Eddersee, Möhne and Sorpe dams and following floods in rivers Ruhr and Eder.

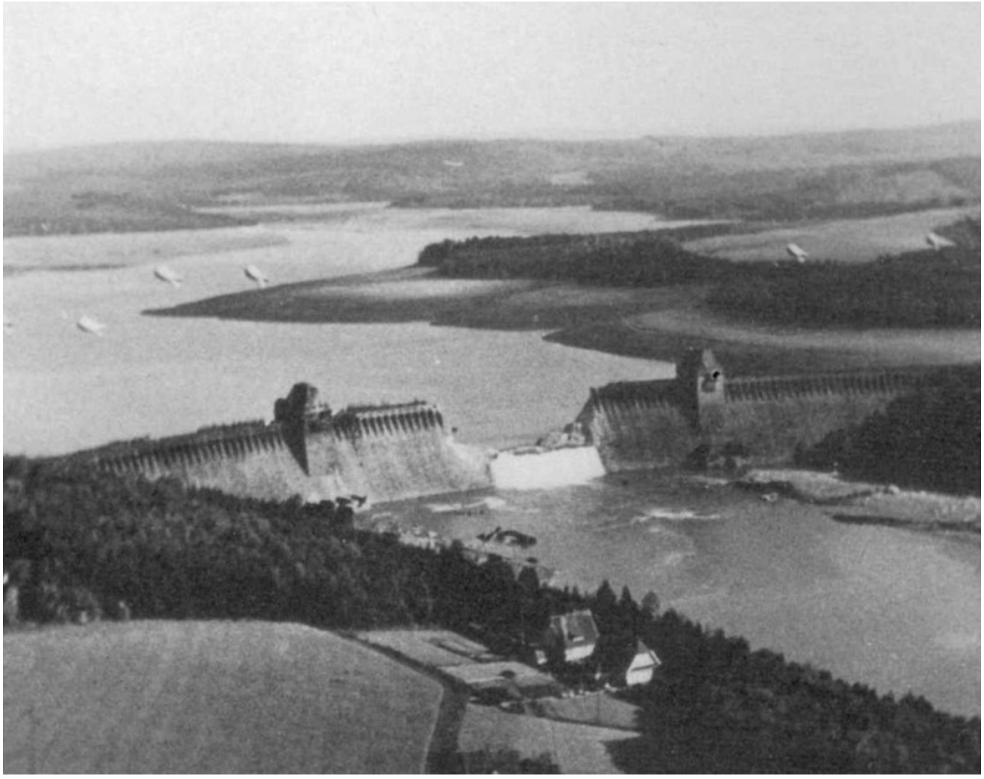

Figure 2. 3: Möhne dam failure (Wikipedia)

2.3 Statistical approach

According the consulted data, the constructions of dams has risen during the first half of the past century, reaching a peak in the 1970s in developed countries, in total, in the 20th century there are registered around 45,000 large dams worldwide distributed in 140 countries.

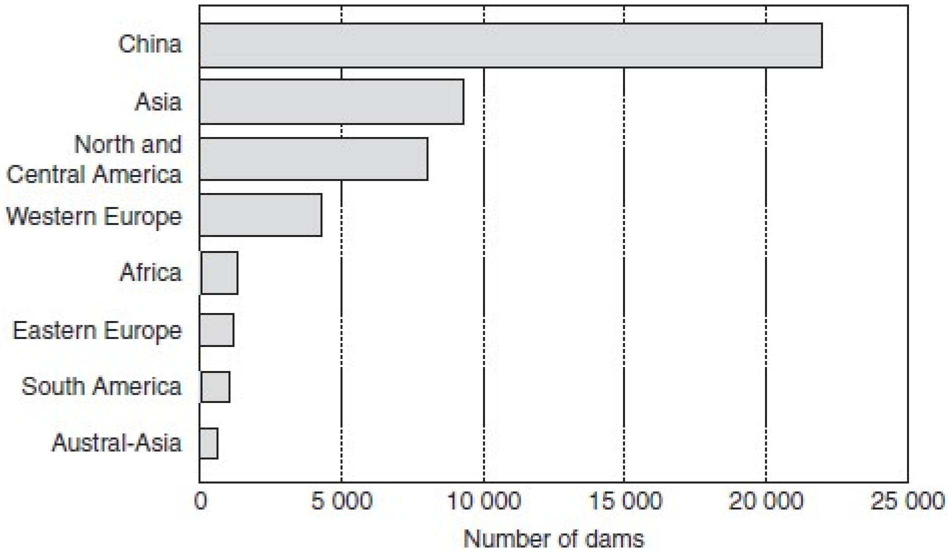

Figure 2. 4: Distribution of large dam's end 20th century (WCD- Dams & development, 2000)

Following the dam's failures according to its age and reasons of collapse are analyzed since a statistical view in the literature.

2.3.1 Age

A compilation of different authors and mainly from the Bulletin 99 are submitted according the collected data. All the graph come from the ICOLD Bulleting 99 unless it is explicitly mentioned.

The analysis of the data shows that most of the failures occurred before the 50´s corresponding to a 2.20 %, 117 failures registered of 5,268 dams constructed in this period, being the highest failure rate between the years 1910 – 1920. One more time, it is important to mention that the incidents carried out in China and ex USSR are not included.

One of the reasons that the failures after the 50's decreased to 0.50 %, is that less dams have been built. The reduction of the ratio can be attributed as well to the improvement of the technologies, construction methods, design by itself, regulations, more rigorous controls and avoiding some materials that- have been mentioned in the present thesis.

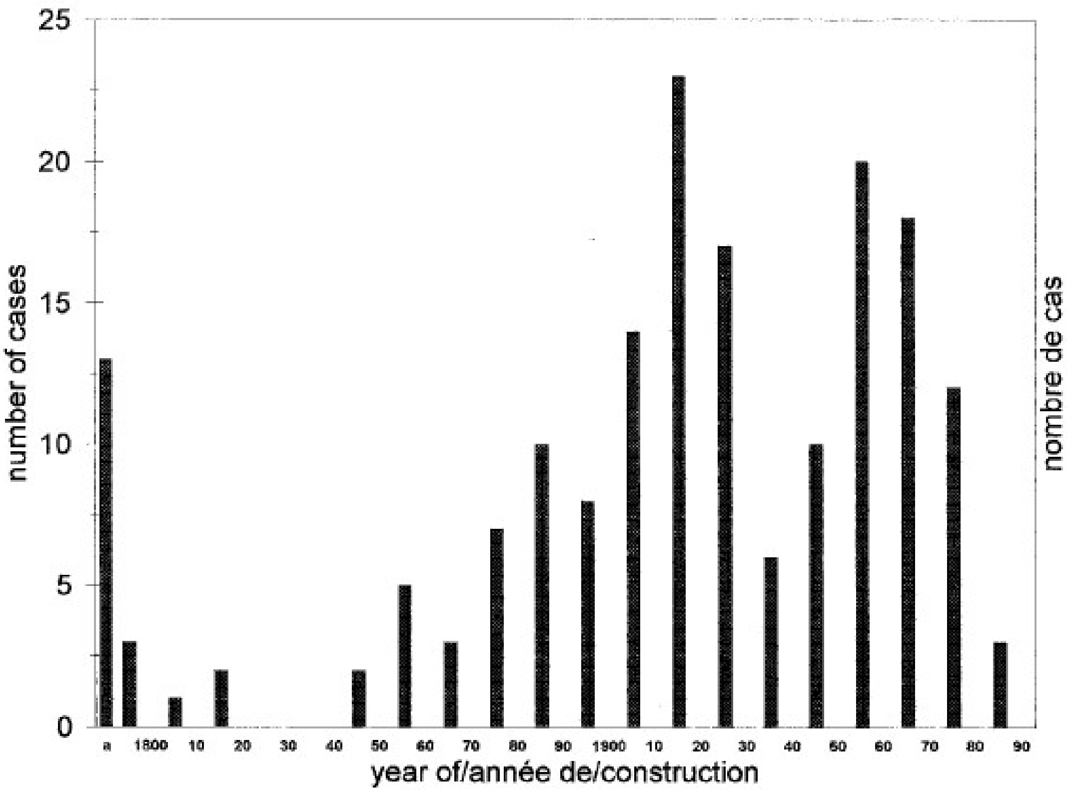

Figure 2. 5: Failures by year of construction (ICOLD Bulletin 99, 1995)

Most of the failures were in dams with moderated height within the large dam's definition. Although, there is a majority in number of "small" dams constructed. The most failures correspond to relatively new built dams within the 5 first years after construction and if the period is extended until the 10 years, the proportion reaches around a 70% of all the incidents.

In order to make a comparison of the result of the Bulletin 99, the following graph correspond by a research made by L.M. Zhang [7b] were more than 900 dam failures were collected and analyzed. All the cases correspond to earth dams. In contrast with the database of the Bulletin 99 with includes also other dam types. Nevertheless, in the ICOLD data, most of the dam failures reported were of earth dam as well, so the comparison can be considered relatively valid.

Construction year range	Case number	Percentage (%)
Before 1800	8	1.3
1800-1849	11	1.9
1850-1859	8	1.3
1860-1869	14	2.4
1870-1879	5	0.8
1880-1889	21	3.5
1890-1899	32	5.4
1900-1909	38	6.4
1910-1919	50	8.4
1920-1929	41	6.9
1930-1939	31	5.2
1940-1949	22	3.7
1950-1959	38	6.4
1960-1969	53	8.9
1970-1979	36	6.1
1980-1989	9	1.5
After 1990	2	0.3
Unknown	174	29.6
Sum	593	100.0

Figure 2. 6: Construction time of failed earth dams (L.M. Zhang et al, 2009)

The table shows that for the periods 1910-1919 and 1960-1969 also occurred the higher percentages of failures with 8.4% and 8.9 respectively. The percentage of unknown cases is very high.

This graph below shows and corroborate that the period with highest incidents is before the 10 years old. With a huge difference to the following period that decrease till the order of 20 cases.

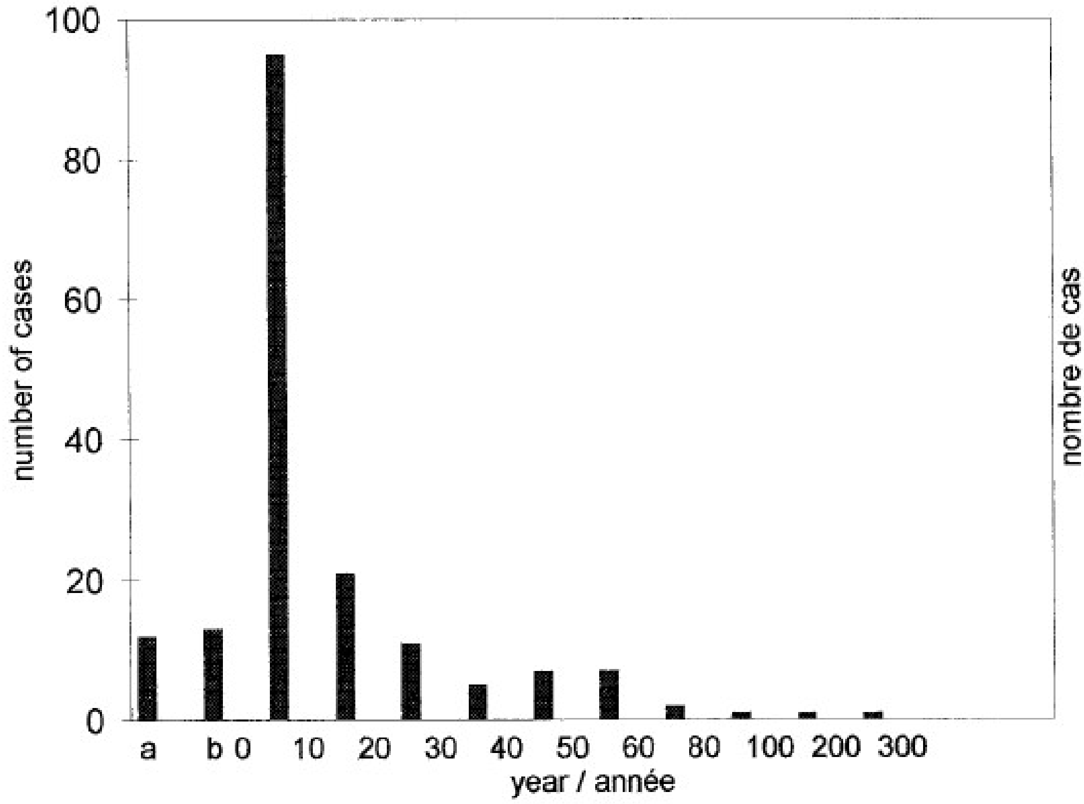

Figure 2. 7: Failures by age (ICOLD Bulletin 99, 1995)

Age range	Case number	Percentage (%)
0-1	85	14.3
1-5	96	16.2
5-10	36	6.1
10-20	62	10.5
20-40	58	9.8
40-60	31	5.2
60-80	16	2.7
80-100	7	1.2
100-150	10	1.7
>150	6	1.0
Unknown	186	31.3
Sum	593	100.0

Figure 2. 8: Failures by age. Earth dams (L.M. Zhang et al, 2009)

The table carried out by Zhang et al, also shows that the greatest percentages are in the first 10 years.

In more detail, within the first 10 years, the highest number of failures reported are before the first year, as it is shown below. This could be due to the first filling.

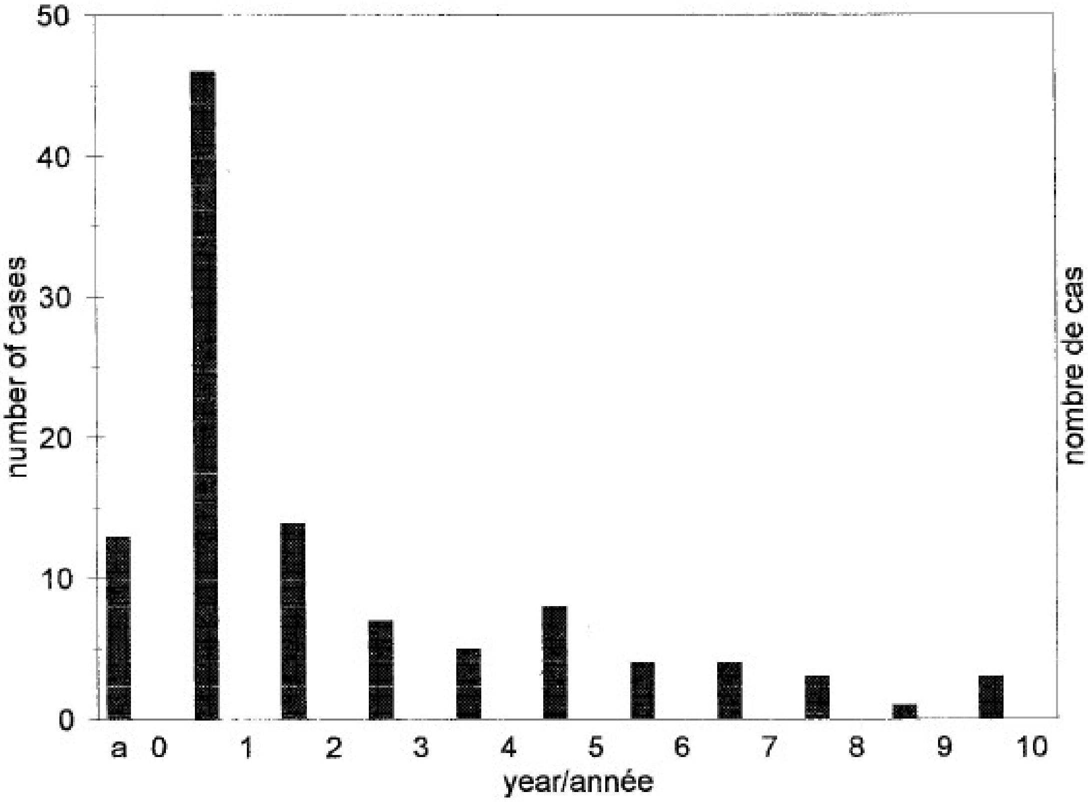

Figure 2. 9: Failures within 10 years. Less <1 year. (ICOLD Bulletin 99, 1995)

2.3.2 Height

Another of the big topic of the ICOLD Bulletin 99 was to make a statistical analysis of the dam failures, according its height, to verify or not if there is a correlation with this factor.

In the figure below, a ratio between failed and existing dams and its height is compared. It shows, that the height does not play an important role in the dam's incidences.

Ratio:

$$\frac{\text{failed dams of height H}}{\text{total failed dams}} \text{ and } \frac{\text{existing dams of height H}}{\text{total existing dams}}$$

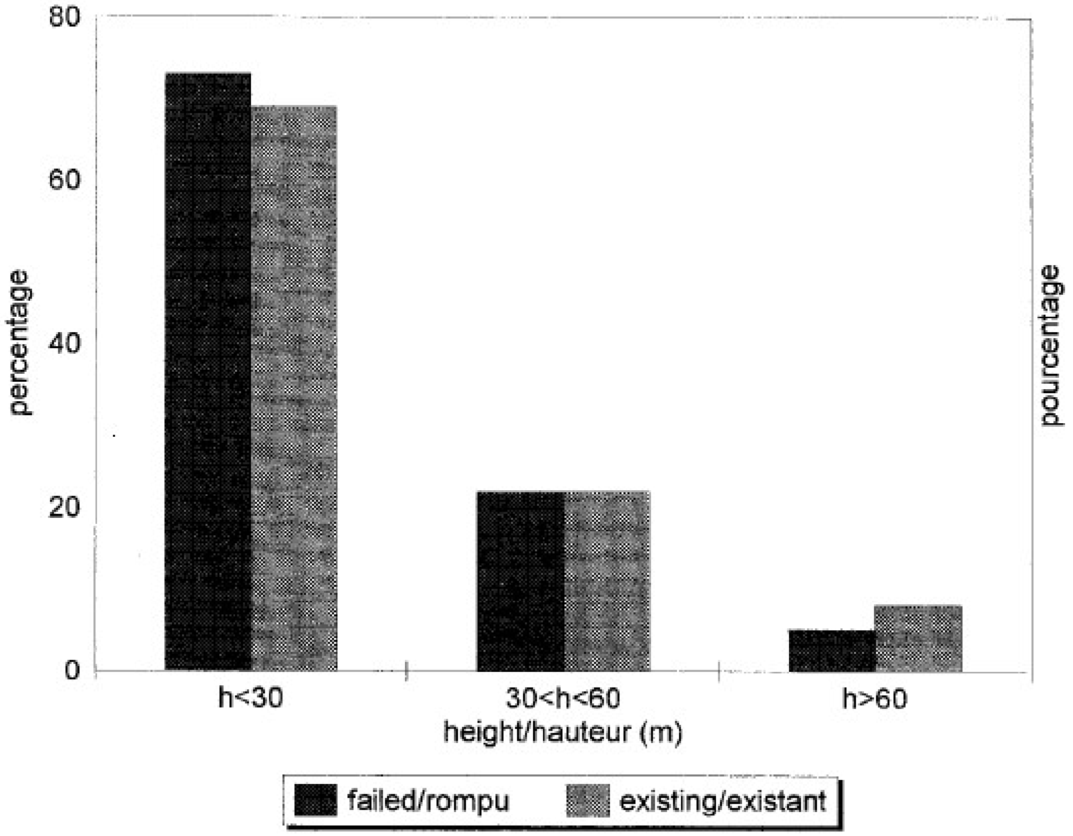

Figure 2. 10: Comparison of ratios (ICOLD Bulletin 99, 1995)

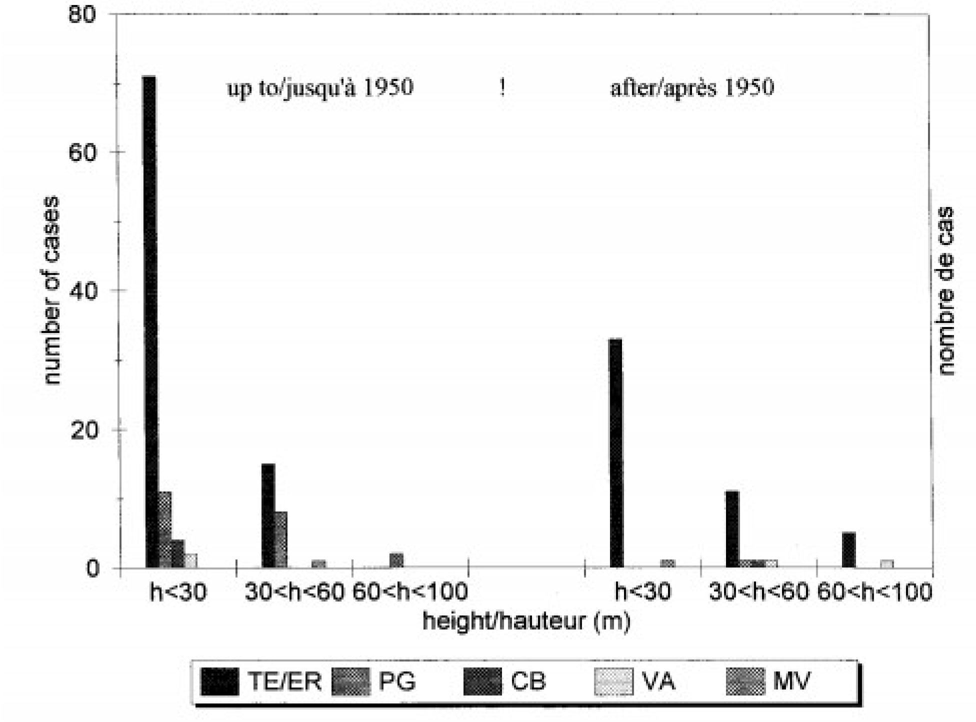

Figure 2. 11: Number of failures by height and type (ICOLD Bulletin 99, 1995)

The figure 2.12 demonstrates what it was already mentioned in previous sub-chapter. There is a reduction in the in the cases of failures after the 50´s. With a clear highest number of registered cases of earthfill and rockfill dams respect to the other types of dams.

Where:

PG = Concrete gravity; CB = Concrete buttress; VA = Concrete arch; MV = Concrete multi-arch; TE = Earthfill; ER = Rockfill.

According the ICOLD Bulletin 99, the ratio between failed dams of height H and dams built of height H shows that there is no correlation between the failures and the heights of the dams, due to the ratio varies very little. The following graph says that most of the incidents occurred in dams lower than 30 m.

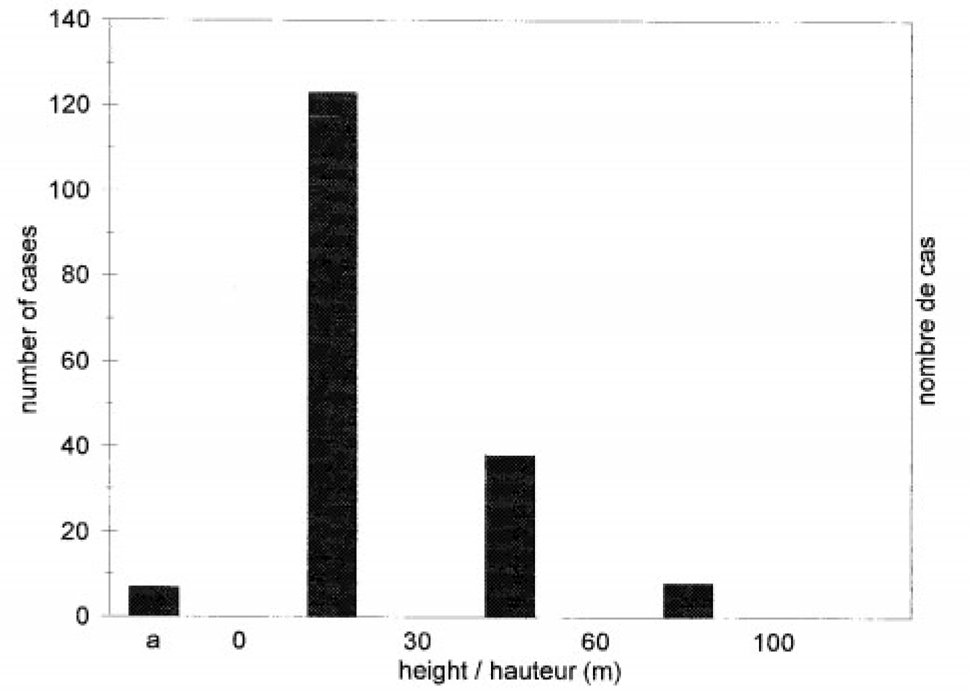

Figure 2. 12: failures by height (ICOLD Bulletin 99, 1995)

Height range (m)	Case number	Percentage (%)
>100	4	0.7
100-60	10	1.7
60-30	44	7.4
30-15	135	22.8
<15	301	50.8
Unknown	99	16.6
Sum	593	100.0

Figure 2. 13: Failures by height. Earth dams (L.M. Zhang et al, 2009)

The study by Zhang et al, also shows the same information but only considering earth dams. The dams with height among 15-30 m take almost the 23% of the cases in their database, on the other hand, if the dams with <15 m, which are probably not large dams are considered, the numbers rise until 51%.

2.3.3 *Comparison dam – failure reasons*

Different authors and commissions have carried out several investigations about failure causes and break mechanisms and the consequent publications like Gruner 1964, Babb & Mermel 1968, Biswas 1971, ICOLD 1974; Schnitter 1976, Goubet 1979, Vogel 1982, ITCOLD 1991; Schnitter 1993, ICOLD 1995).

The DSDF-VIENNA [3a] has registered 712 failures of small and large dams in 50 countries. If the ICOLD criteria for large dams is applied, the resulting number of dam failure cases is 315.

The 77% of these cases correspond to earth or rockfill dams, a 17 % to masonry or concrete dams, where the main causes of failure were overtopping with a figure of 36% and foundation or piping reasons with a 34%. Sliding failures correspond a 9%. Breach dimensions and failure times are breach parameters, which classify the intensity of failure process. That means that the values of them contribute to the evaluation of the risks of a dam type in relation to an expected failure cause according to DSDF-VIENNA.

Table 2. 7: Percentage of main failure reasons

Reason	**Percentage [%]**

Overtopping	36
Foundation and pipping	34
Sliding	9

For failures time due to overtopping in their full height:

Table 2. 8: Overtopping time failure by dam type

Dam Type	**Time [hrs]**
Earthfill	0.2 – 3.0
Rockfill	0.2

In cases of flash floods, the earthrockfill dams are washed away over their full length quickly. In the cases or masonry or gravity dams they withstood overtopping for longer time, but it also could occur abruptly.

For the failures due to piping in all dam types:

Table 2. 9: Piping time failure in all dams

Dam part	**Time [hrs]**
Foundation	0.5 – 4.0
Body	0.3 – 0.3

The large embankments failed over their full height, on the other hand, the lower homogeneous earth dams made of dispersive clay, only erosion tunnels in the dam body were left as results of the failure process connected with low rates of outflows.

The conclusion reached by L. Berga [3a], is that the investigations of failure-cause-specific break-mechanisms of dams show different behaviours of types of dams in failure processes, which attribute to the same failure causes. This fact should influence risk classifications of existing dams or of dams in construction.

3. Worldwide dam failures – after 1995

3.1 Common approach

In the chapter 3, information is extracted from the created database in a statistical approach. Thus, only dam failures among the period 1996-2018 are considered. The cases from China and Russia are not included in the database, but some examples and data are discussed.

Firstly, only large dams by its height are considered. According the volume storage definition is not taken into account due to the general lack of information available. This simplification is also made in order to make further feasible comparisons. The total number of cases considered are 28 which will be divided and classify in four different categories depending on the moment or circumstance when the failure took place and by its dam's typology. The information is presented with the next format:

Country	Name	Height [m]	Year of construction	Year failure	Failure reasons	Dam Type

Subsequently, statistical information and results are generated from the failure list. In each table or figure, it is specified either only large dams or which cases listed in the database are considered (from a total of 191).

In general, the results are classified by the dam type and they show the distribution of the failures by country, height comparison, year when the failure happened and its construction period.

Later on, a time dependent analysis is carried out to find patterns and failure cycles in time from the database.

Finally, a comparison is carried out from the main results of chapter 2 (failures until 1995) and the ones that were observed among 1996-2018.

3.2 Failures by mode of operation

3.2.1 During construction

The failures during construction in the studied period can be assorted in two categories. One by hydrological events like rainfall and the other by design problems. As it was explained in chapter 2, the hydraulic structures are vulnerable during construction because they have not fully developed their designed bearing capacity. All the registered cases are embankment dams.

Table 3. 1: Failure during construction. Embankment large dams

Country	Name	Height [m]	Year const.	Year failure	Failure reasons	Dam Type
New Zealand	Opuha dam	50.0	During const.	1997	Heavy rain. Breached.	Embankment
USA	Carl Smith	17.2	During const.	1998	Quality problems. Structural failure by sliding.	Embankment
Laos	Attapeu-saddle dam D	16.0	During const.	2018	Heavy rain. Collapse	Embankment

The three cases registered for dam failures were directly or indirectly result of hydrological events. The cases of Attapeu saddle dam D (Laos) and Opuha dam (New Zealand) were as consequence of heavy rains which led to failure of the dam. On the other hand, Carl Smith dam in the USA a heavy rain triggered a slide on the right side of the dam causing a breach and consequently the reservoir was running through it. Associated failures occurred in the irrigation diversion structures that were washed out.

For the Opuha dam, a heavy rain with an occasional heavy burst occurred in 1997 during the construction of the dam. A cut was opened in the left abutment over a rock saddle of the dam to avoid an uncontrolled breach of it [6b]. Nevertheless, the water started to flow over the cut and progressively widening reaching a size of 20 m width and 6 m depth.

In 2018 the saddle dam D collapsed with casualties between 40 and hundreds according the sources. It was an auxiliary structure of a hydroelectric project in Laos in a tributary of the Mekong River. Although the triggered reason of the failure of the saddle dam was a heavy rain, there are still some investigations in process and they indicate that there were also substandard during the construction and the dam was not planned to deal with extreme weather events during its construction.

3.2.2 After construction

Failures occurred after construction also refer to the ones by first filling. There are only three cases founded in total.

The Gararda dam in India is a clear example of failure due to the first filling, where a breached in the central body with a length of about 100 m appeared due to the lack of central clay core and devoid of designed downstream horizontal and inclined sand filters [8b].

The case of Brazil is special, due to the main reason of the failure was the collapse of a tunnel which was supposed to be independent in structural terms to the dam. Nevertheless, during the first and the second filling of the reservoir, deformation and cracks were detected. At the beginning, they were in expected magnitude order, but they developed to the point that is was necessary remediation measures. After 6 months of the first filling (3 – 4 months of the second filing) the dam collapsed with a transversal crack of 300m along the dam body triggered by the „unrelated“ tunnel. Therefore, it is included in the category of after construction failures.

The case of Tokwe Mukorsi in Zimbabwe is not well documented, but the failure occurred during the first filling.

Table 3. 2: Failure after construction. Embankment large dams

Country	**Name**	**Height [m]**	**Year const.**	**Year failure**	**Failure reasons**	**Dam Type**

Brazil	Compos Novos	202.0	2006	2006	Tunnel collapse leading to cracks and breach	Embankment
India	Gararda	32.0	2010	2010	Breach in the dam body	Embankment
Zimbabwe	Tokwe Mukorsi	90.3	2008	2014	Downstream slope failure	Embankment

Table 3. 3: Failure after construction. Gravity large dams

Country	**Name**	**Height [m]**	**Year const.**	**Year failure**	**Failure reasons**	**Dam Type**
Turkey	Köprü	109.0	2012	2012	Spillway failure	Gravity

In the case of the unique gravity dam failure registered, during the first filling, a gate in the diversion tunnel broke during a heavy rain event, leading to an uncontrolled discharge with fatal consequences.

The low number of failure cases can be achacable to the lessons learned from former events and that in recent times less dams are being constructed. The monitoring measures, improving of construction techniques, regulations and the awareness of this specific factor surely have played an important role in the decrease of incident too.

3.2.3 During operation

Table 3. 4: Failure during operation. Embankment large dams

Country	**Name**	**Height [m]**	**Year const.**	**Year failure**	**Failure reasons**	**Dam Type**
USA	Timberlake	33.0	1920	1996	Overtopping	Embankment
Austrialia	Dartmouth	15.0	1977	1996	Unlined rock steps damaged. Erosion during low spill	Embankment

USA	Nine Mile	25.7	1908	1996	Quality problems. Culverts and embedded structures	Embankment
USA	Moss Creek Lake	20.4	1939	2000	Quality problems. Seepage or piping erosion	Embankment
India	Jamunia	15.0	1921	2002	Piping leading to breaching	Embankment
Syria	Zeyzoun	32.0	1996	2002	Cracks appeared and the dam collapsed during flooding	Embankment
USA	Big Bay	15.6	1992	2004	Hole in the dam which lead to failure	Embankment
India	Nadgavan	23.0	1998	2005	Overtopping. Weir uncapacity during heavy rain	Embankment
USA	McClure	19.5	1919	2007	Penstock rupture. Reservoir water level drown down	Embankment
India	Jaswant Sagar	43.0	1889	2007	Piping leading to breaching	Embankment
India	Chandiya	23.0	1926	2008	Breach due to flooding	Embankment
Brazil	Algodões	21.6	2005	2009	Heavy rain. Filure spillway and then brake of the wall	Embankment
USA	Delhi	18.0	1929	2010	Heavy rain, flooding	Embankment
Venezuela	Manuelote	35.0	1975	2010	Overtopping leading to breach	Embankment
Bulgaria	Ivanovo	19.0	1962	2012	Collapse of dam body	Embankment
Japan	Fujinuma	18.5	1949	2011	Failed after 2011 Tōhoku earthquake	Embankment

USA	Oroville	235	1968	2017	Heavy rain. Damages in spillway	Embankment

Of the 17 large embankment dam failures listed, 14 of them (82%) correspond to incident associated with hydrological reasons like overtopping, seepage or piping.

Mentioning some cases, we found for instance, Algodões dam in Brazil, where in 2009 the dam was struck by a flood. Nevertheless, it did not occur an overflow above the crest. Instead, the spillway at the right abutment was destroyed by erosion of the backfill behind the chute wall during the initial phase of the failure. It resulted in the collapse of the part of the dam body.

Another case happened in Bulgaria, where Ivanovo dam failed in 2012 after a period of heavy snowmelt. Before, there were existing cracks that were not repaired during years. Legal problems associated to whom belong the dam and therefore the responsibility to fix the cracks and maintenance were the primary reason of the disaster.

Perhaps, the most famous recent case is Oroville dam in 2017 where its two spillways overflows had suffered erosion after heavy rainfall event and the never used auxiliary spillway worked at first time.

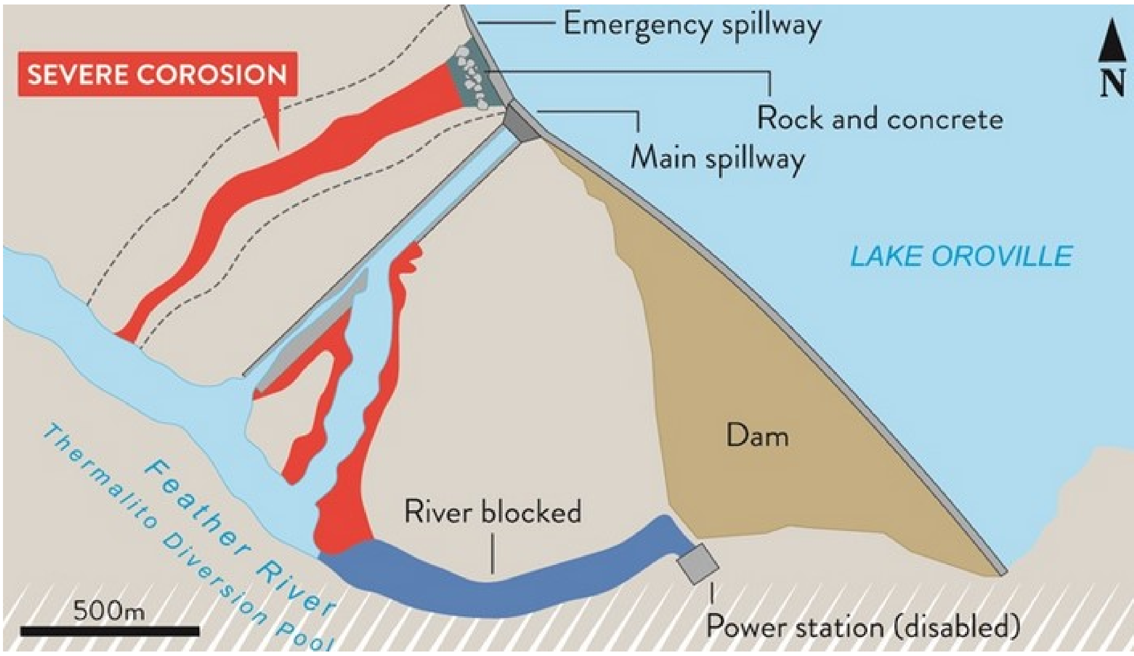

Figure 3. 1: Oroville dam, erosion plan view (newcivilengineer.com)

In the initial phase of the crisis, the main spillway was working normally, but after some days of working, some craters appeared, so in order to avoid further damages, the auxiliary spillway was putted on duty. The water flows over the auxiliary one causing erosion and damages with a greater eroding rates than expected. The authorities decided to place rocks and concrete even using helicopters to repair erosion damages in the aux. spillway. In order to make those works, the release of water through the main spillway was increased. Nevertheless, this action produces the erosion in the adjacent hillside, producing a debris dam that blocks the river and forces the closure of the hydroelectric plant (Wikipedia).

The other cases that were not as consequence of hydrological events are treated separately.

For the Big Bay dam failure, a complex process was developed. A big sinkhole appeared and it was noticed in first instance as a kind of liquefaction, but the root of the failure was never conclusive, other factors were involved like defects in the outlet conduit, lack of effective seepage control, poor design, construction seepage filter and monitoring.

There is just one case of failure due to earthquakes, that happened in 2011 in Japan. The Fujinuma dam failed after the Tōhoku earthquake. The investigation revealed that the foundation of the dam was not a proper one. The breach occurred at the tallest section of the dam.

For Chandiya dam failure not further information in addition of simple breaches was found, therefore, it was not possible to assess it.

Table 3. 5: Failure during operation. Gravity large dams

Country	Name	Height [m]	Year const.	Year failure	Failure reasons	Dam Type
USA	Folsom	100.0	1956	1995	Spillway gate failure	Gravity
Brazil	Camará Dam	50.0	2002	2004	Internal erosion in the abutment	Gravity
USA	Taum Sauk	38.0	1963	2005	Computer/operator error and also piping.	Gravity
USA	Tallulah Falls	42.7	1913	2010	Equipment failure. Uncontrolled spillway discharge	Gravity

In regard the gravity large dam failures, four cases are registered. Three of them (75%) were as result of equipment failures or malfunctions, which led to an uncontrolled discharge. However, in the case of Camará dam in Brazil the burst by internal erosion in the abutment was the result of a poor maintenance with a consequence 3000 people without homes.

Table 3. 6: Failure during operation. Arch dams

Country	Name	Height [m]	Year const.	Year failure	Failure reasons	Dam Type
Russia	Sayano-Shushenskaya	242.0	1978	2008	Waterhammer led to explosion of hydropower plant	Arch

The only arc dam failure registered, is the Sayano-Shushenskaya dam in Russia, which at the moment of the accident was the sixth largest hydroelectric plant in the world [5c].

The reasons of the failure were the sudden closing of wicket gates that led to a heavy water hammer in the spiral case and penstock and consequently its collapse. The associated civil structure was destroyed. A heavy reverse water hammer (draft tube) also produced the elevation of the turbine cover, shaft, etc. The powerhouse was rapidly flooded.

The consequences were the total destruction of two turbines and another two were damaged. There was big environmental impact due to oil leakage and 75 people died because of the accident.

Dam failures in China are not covered. Nevertheless, two specific cases are mentioned, but not included in the database due to they did not failed exactly. They are the Zipingpu dam, a concrete faced rockfill dam (CFRD) with 156m height and Shapai arch dam with 131 m height. In 2008 the Wenchuan earthquake with a magnitude of 8 M_w, and both dams suffered cracks in their dam body and associated facilities and structures got severe damages. Although, the structural integrity of the dams was not in danger. That event and the performance of the dams prove the high security of large dams facing earthquakes.

Another example, in China as well, is the world's largest power station of the Three Georges dam, which is capable to trigger earthquakes in a phenomenon called "Reservoir Induced Seismicity (RIS) when huge water reservoir meets specific geological conditions. It has been proven that this dam has induced many earthquakes and there are concerns about the consequences of the failure of such dam.

3.2.4 Warfare

In the period between 1996 – 2018, no dam failures due to warfare actions have been reported or found in the literature. Nevertheless, during the Kosovo war the NATO forces were thinking to intervene in some critical infrastructures due to the risk of failure or sabotage.

One case during Kosovo war, was Peruća lake embankment dam with 64m height, in Croatia. In 1993 the Serbian forces detonated 30 tons of explosives in the gallery. It produced heavy damages but the dam did not collapse in part for the action of the UN personnel.

Moreover, in Africa there are some reports of concerns that due to the conflicts the maintenance or right operation of hydraulic structures were neglected and the hazards of dam failures increased significantly.

Recently, the war developed in the middle-east with the Islamic State many situations arose. The Islamic State used dams as a weapon to control regions or for their own interest like the case of Tabqa Dam, which is the largest earth dam in the world. Another example is the Fallujah dam, where both bands flooded deliberately hundreds square kilometers of farmlands and villages in Iraq.

In 2014, the Islamic State took the control of the Mosul dam. There were concerns about how the terrorist could use the facility, flooding extensive areas intentionally or that the dike could break due to the lack of maintenance. In 2006, U.S. Army Corps of Engineers has called Mosul dam as the most dangerous dam in the world. In addition, there are serious doubts about the dam stability due to the known erosion in the foundation.

3.3 Quantitative Analysis and Statistical approach

3.3.1 Statistical approach

In the present subchapter, all graphs and tables were generated from the created database (Appendix 1) unless it is specified. Due to the difference level of detail from the diverse sources taken to develop the database, not all the failure cases were used to generate statistic, only when the information available was good enough to provide representative information. In each graph or table, the data that was used from the 191 cases listed in the database is specified.

For the single case of arch dam (Sayano–Shushenskaya.) No further analysis was developed, because it would not provide any statistical information.

The distribution by country of the failures is presented in the figure below.

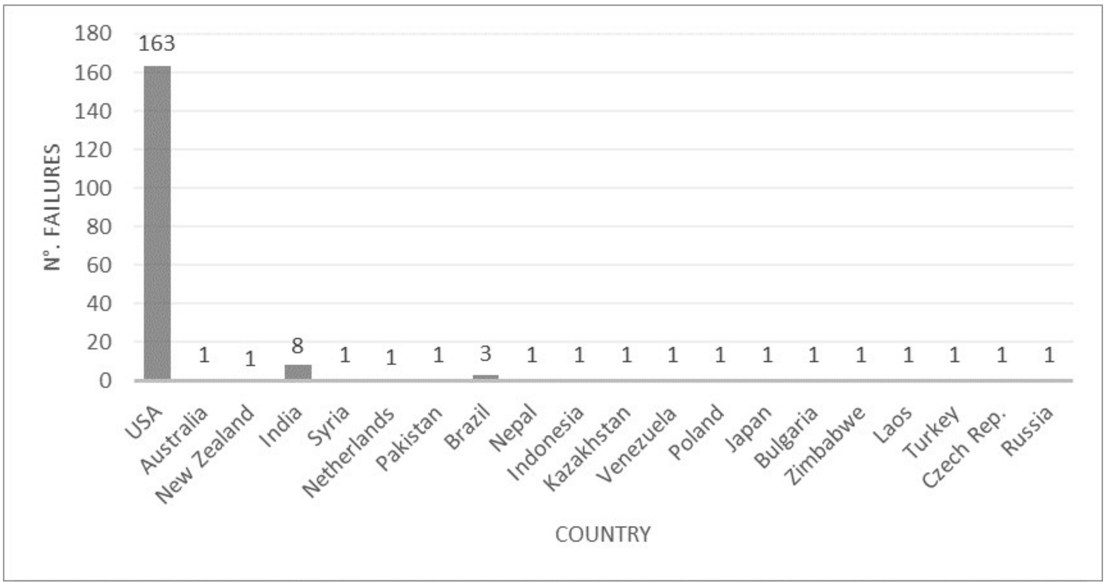

Figure 3. 2: Numbers of failures in database by country. All dams

It is important to mention, that the great number of cases registered in the USA, is probably due to in this country, in addition, that is by far the country with more constructed dams in the world (except China), there is a great network of information available with public access, in contrast with the ICOLD database. For instance, the National Performance of Dams Programs (NPDP) of Stanford University, where a massive information about dams in the USA is registered.

In the NPDP database, the information of dams out of the USA is very limited. Nevertheless, it has been of huge help for the development of the present work. The data from China and Russia are not presented.

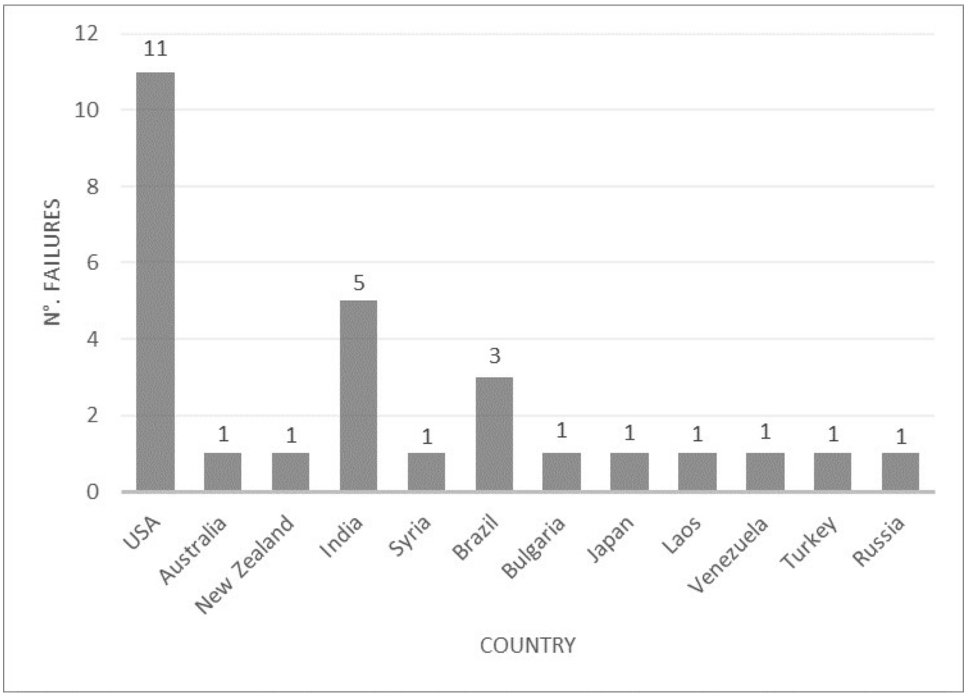

Figure 3. 3: Numbers of failures in database by country. Only large dams

For large dams, most of the registered numbers of failures correspond to the USA, followed in second position by India and Brazil. Tailing dams are not included in the goals of the thesis, but during the research process, the case of Brazil was significant. Many numbers of failures have occurred in this country, with huge impact in the environment, cost and in fatalities.

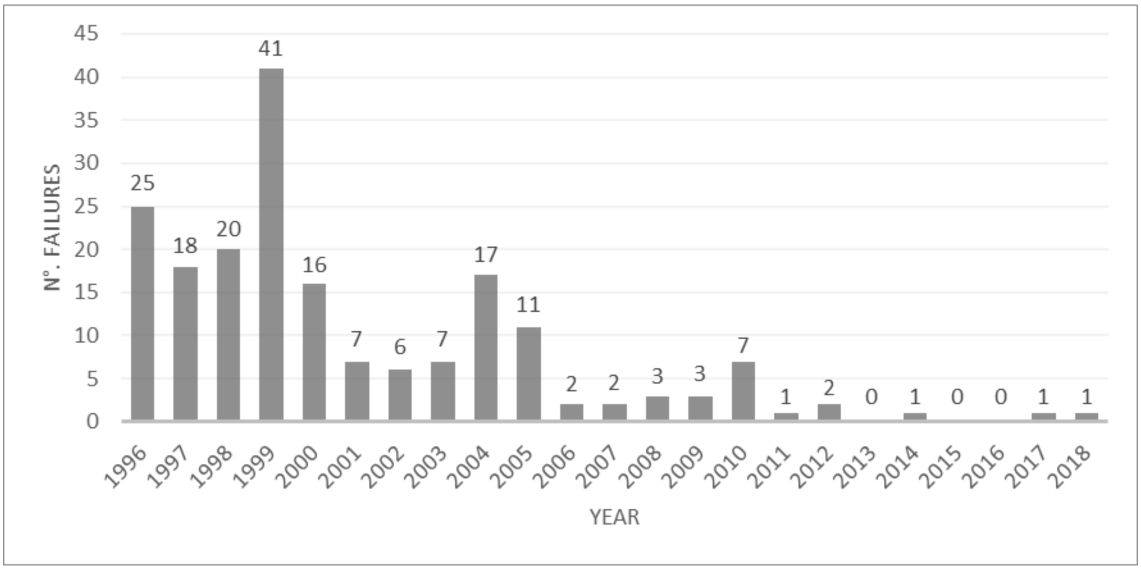

Figure 3. 4: Numbers of failures in database by year. All dams

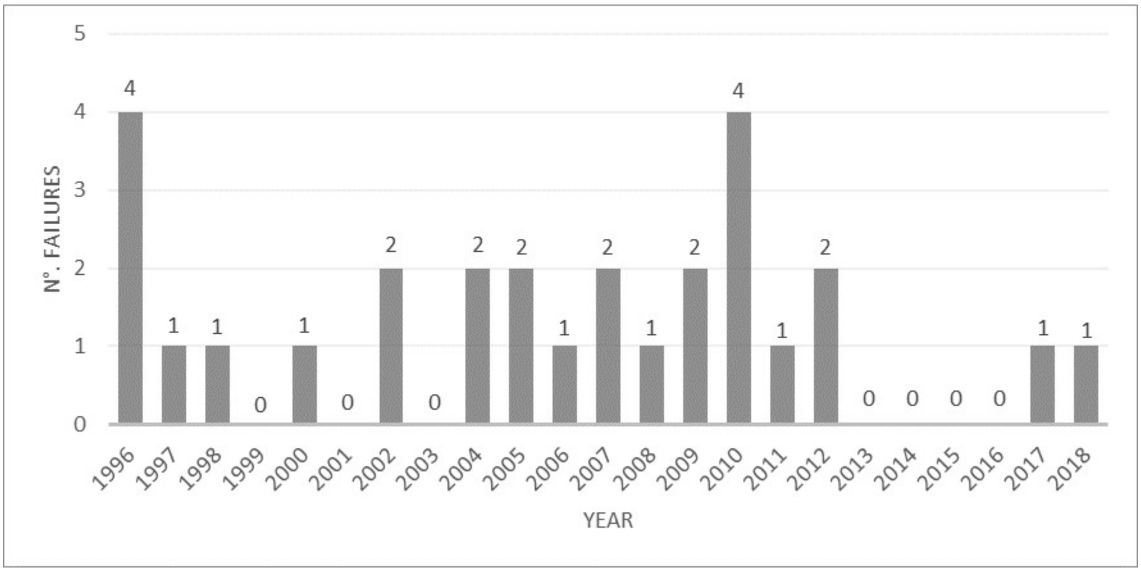

Figure 3. 5: Numbers of failures in database by year. Only large dams

In the figures 3.4 and 3.5 shows that, there is not a correlation between large and no large dam and its failures by year. In common, only in the year 1996 it is possible to appreciate a high number of cases for both. For no large dams, the year 1999 presents the highest number of incidents.

The table below shows the total amount of dam embankment failures with the corresponding percentage. Large, and no large dams are mixed in this table. The main division correspond to the dam´s height. As it was mentioned before, only the when a dam is equal to higher to 15 meters is considered as a large dam. In the database there are case higher than 100 m or even 200 m. Although, for statistical and practical reasons, they are collected in the group of dams higher than 45 m.

Table 3. 7: Height of failed dams. All embankment dams

Dam height [m]	Number of cases	Percentage [%]
<15	109	82.0
15 - 30	14	10.5
30 - 45	5	3.8
>45	5	3.8

It is very significant that the majority of the cases corresponds to small dams. It follows the same line than the conclusion made by ICOLD in the bulleting 99. It may show that the probability of dam failure in small dams is much higher than for the large ones. This statement can be barely taken into account, unless the relative comparison between the number of small dams and its failures is compared with the ratio with the large dams.

Table 3. 8: Height of failed dams. Only large embankment dams

Dam height [m]	Number of cases	Percentage [%]
15 - 30	13	59.1
30 - 45	5	22.7
>45	4	18.2

The table 3.7 shows that, there is a statistical decrease regarding the percentage of dam failures when the height increase. The data must be managed with care, it could happen that it is not representative enough, but attending to the collected information, a factor is shown. As long as the dam is higher, higher is the safety.

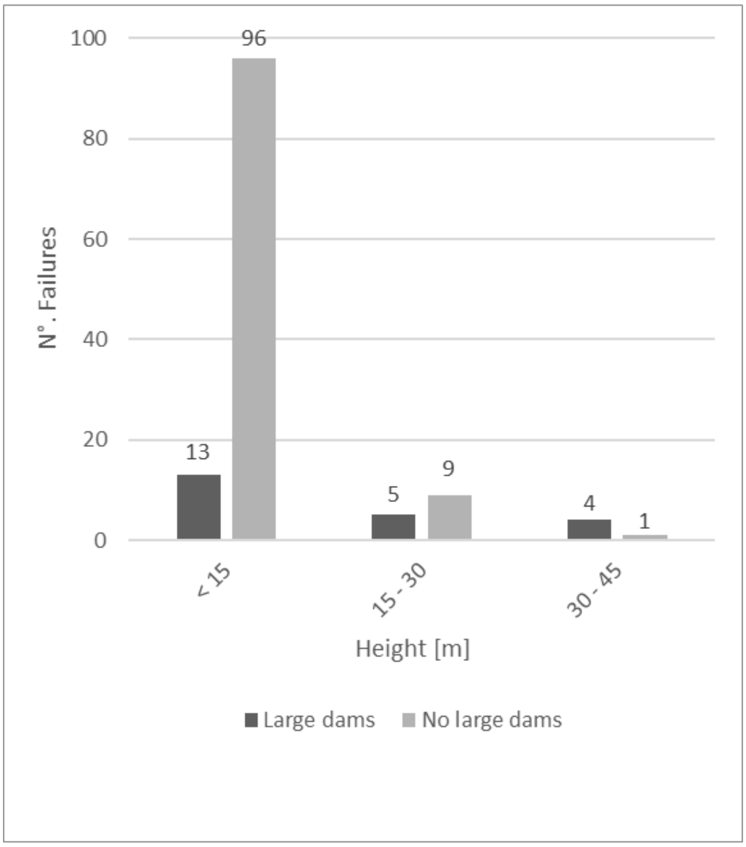

Figure 3. 6: Large and no large embankment dam failures by height

The figure 3.6, provides more indications that the height of the dam failures is a factor to be considered. The difference is relevant, representing the dams lower than 15m a 90%, quite far away of the next category that is around 4% of the no large dam failures.

Table 3. 9: Height of failed dams. All gravity dams

Dam height [m]	Number of cases	Percentage [%]
<15	2	29.0
15 - 30	-	0.0
30 - 45	2	29.0
>45	3	42.8

The gravity dams, show a different statistic, for all the large dams, the risk of failure in the database indicates that higher the dam is, the probability of failure increases.

Table 3. 10: Height of failed dams. Only large gravity dams

Dam height [m]	Number of cases	Percentage [%]
15 - 30	-	0.0
30 - 45	2	40.0
>45	3	60.0

Considering only the large gravity dams, the tendency is the same, reaching a probability of failure of 3/5 when the dam is higher than 45 m.

For all the large dam, according its height listed in the database, the year of construction is filled. Unfortunately, for the dams with a height lower than 15 m, for most of them, it was not possible to find their corresponding year of construction. Therefore, two different kinds of tables are presented, one only includes the failed large dams after 1995 and the other one, includes both, those over 15 meters and those below.

As it was stated in the subchapter 2.3, there is a statistical fact, that shows a difference between the dams constructed before and after the 50's, hence, the first line of the table, corresponds to all the dams in the database which were constructed before the 50´s.

Table 3. 11: Construction time of failed dams. All type embankment dams

Construction year	Number of cases	Percentage [%]
<1950	25	41.0
1950-1959	7	11.5
1960-1969	8	13.1
1970-1979	6	9.8
1980-1989	1	1.6
1990-1999	8	13.1
2000-2009	4	6.6
2010-2018	2	3.2

The table 3.10, provides information of the number of failures and its percentages by its decade. It can be said roughly, that the point of inflection was in the

50´s, the following three decades shows a percentage in average above the 10% of the cases and then the tendency is decreasing with the exception of the 90's.

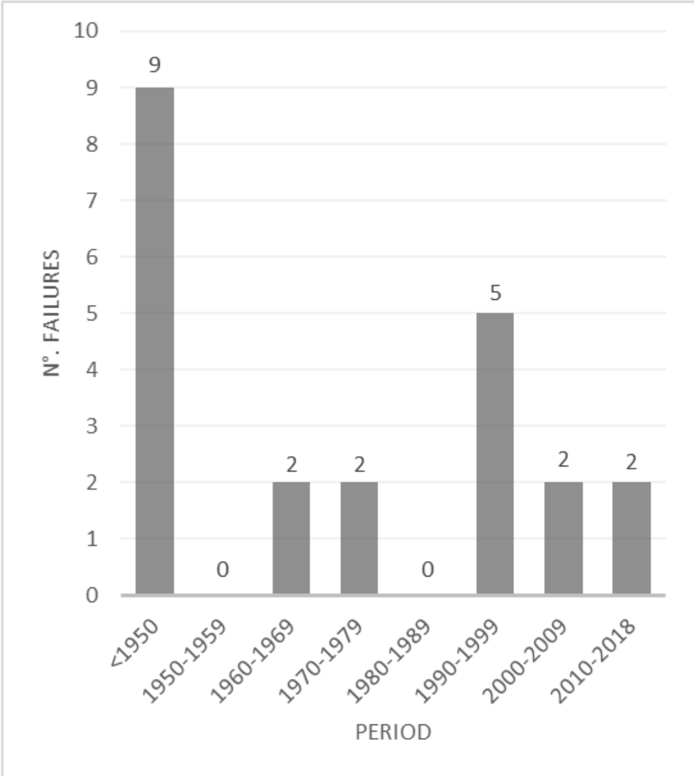

Figure 3. 7: Number of large embankment failures over period

Regarding the large dams, the figure 3.7, says in gross, the same. Before the 50's ratio of dam failures is the highest with an upturn in the 90's.

Table 3. 12: Construction time of failed dams. All type gravity dams

Construction year	Number of cases	Percentage [%]
<1950	2	29.0
1950-1959	1	14.0
1960-1969	1	14.0
1970-1979	-	0.0
1980-1989	-	0.0
1990-1999	-	0.0
2000-2009	2	29.0
2010-2018	1	14.0

From the cases registered in the database, the biggest failure incidences in gravity dams by decades was before the 50's and in the period between 2000 – 2009. However, the number of cases registered is too low to be considered totally representative.

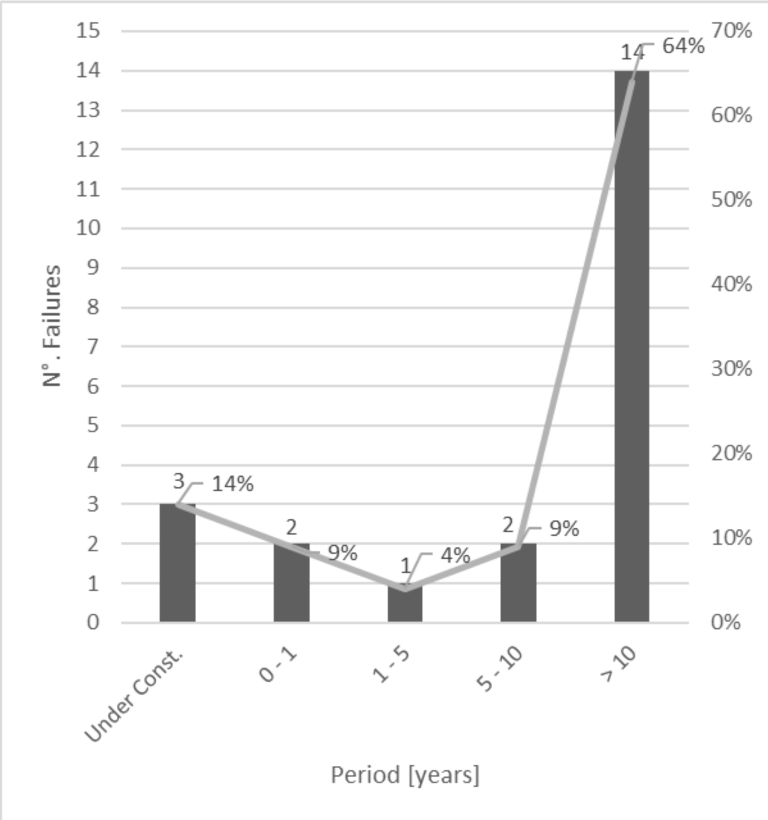

Figure 3. 8: Age of large embankment dam at failure

In opposite to the statistics shown in chapter 2, the ratio of failures in large embankment dams after 1995 is higher in dams older than 10 years. Probably, the lessons learned have been applied. Being only the age of the dam remarkable and decreasing the cases by first filling and hazards associated with early ages of the dams. See the curve shape in figure 3.8 and the similarity with figure 2.2.

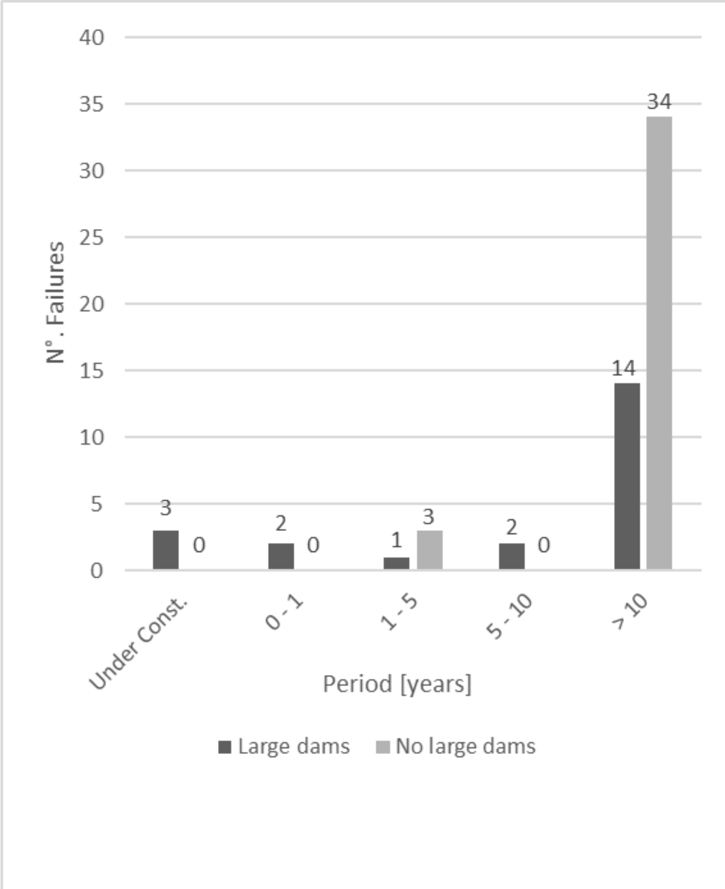

Figure 3. 9: Age comparison large and no large embankment dam at failure

The figure 3.10 shows the dam age when the failure occurred. There is a relationship between the failures and dams when they are older than 10 years (see figure 2.8).

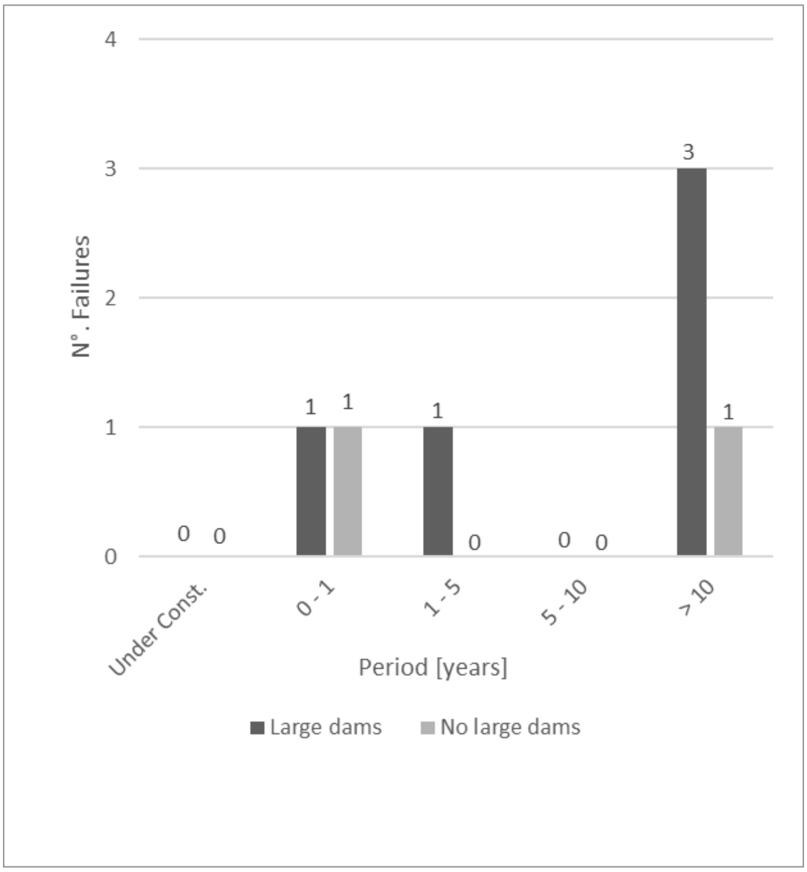

Figure 3. 10: Age large and no large gravity dam at failure

The representative number of gravity dam failures is not high. However, the graph indicates the same tendency than embankment dams.

3.3.2 Time dependent – failure approach

In order to figure the periodicity of dam failures out. A time dependent analysis has been carried out, using an analysis of the temporal distribution.

The spectral analysis is a statistical technique used for characterizing and analyzing sequenced data [3c]. The spectral analysis is a decomposition of a sequence, like the created database of dam failures, into oscillations of different lengths or scales. The data domain is converted into the spectral domain to make it easier to manipulate. The revealed scales are necessary statistical descriptors of data to indicate factors or generate data.

This methodology has been applied to analyze the distribution on dam failures by authors like a X.Y He et at, 2008 [9b], where the energy spectrum methodology is used to analyze the distribution of dam failures.

The mathematic methodology is based on the fourier transformation, where a function of time is decomposed into its constituent frequencies. The Fourier transformation formula for the number of dam failures per year N is expressed like:

$$a(f) = \frac{1}{2\pi} \int_{-\infty}^{\infty} N(t) e^{i2\pi ft} dt$$

$$E(f) = \frac{4\pi^2}{T} \left| a(f) \right|^1$$

- *a(ƒ)* stands for the Fourier Trnasformation *N(t).*
- *E(ƒ)* stands for the fluctuation energy under the frequency *ƒ.*

The spectral density *G(ƒ):*

$$G(f) = \frac{E(f)}{\Sigma E(f)} \times 100\%$$

Using the tool XLSTAT, as extension of Microsoft Excel, the spectral analysis was run with the corresponding database to all the dams and also separately, considering only for large dams data.

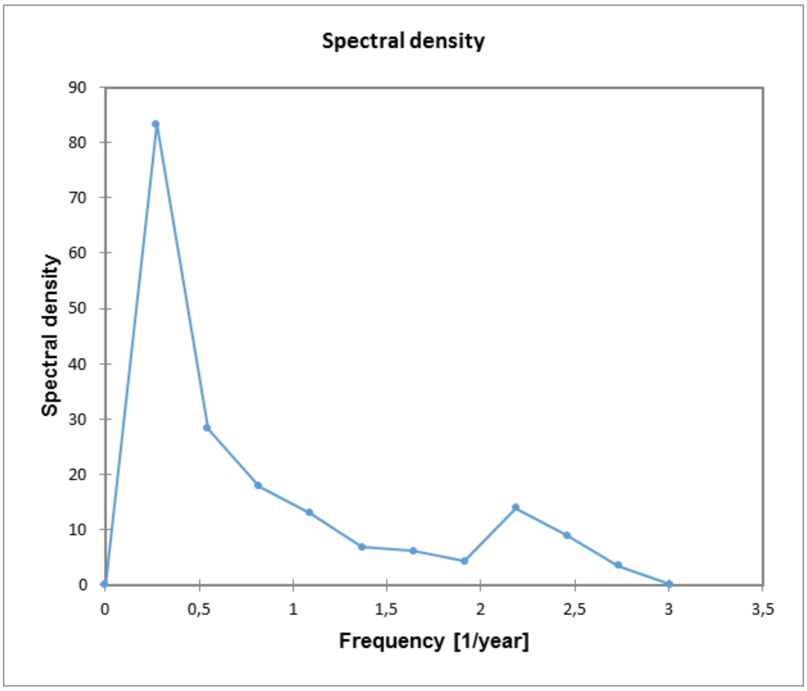

Figure 3. 11: Distribution of spectral density dam failures. All dams

For the case of all dams (191 cases), the spectral density distribution shows 2 peaks, with a corresponding frequency of 0.273 and 2.185 with represent periods of 3.7 and 0.5 years.

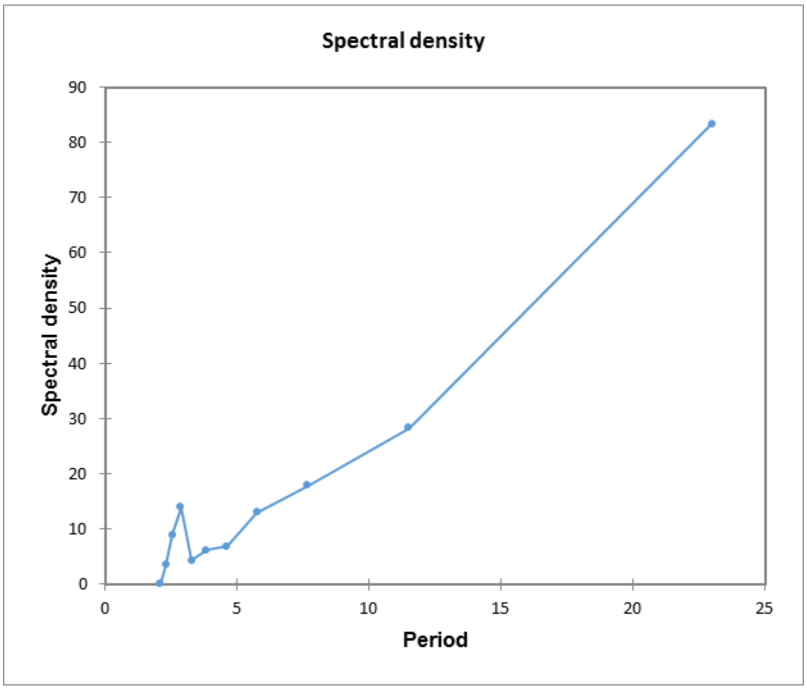

Figure 3. 12: Period of spectral density dam failures. All dams

The peak corresponds to a period of 2.9. That means a change with the regular cycles almost every three years.

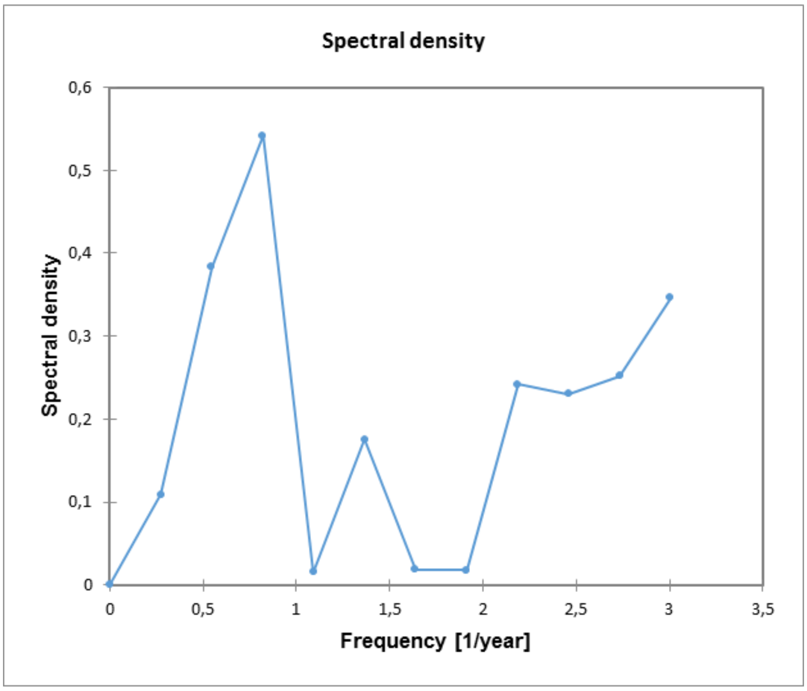

Figure 3. 13: Distribution of spectral density dam failures. Large dams

Regarding only the large dams, the figure 3.13 indicates that there are two clear peaks, with a frequency of 0.820 which means 1.2 years and 1.366 that is 0.7. Therefore, the pattern is slightly below for large dam failures in comparison with all the dam registered in the database.

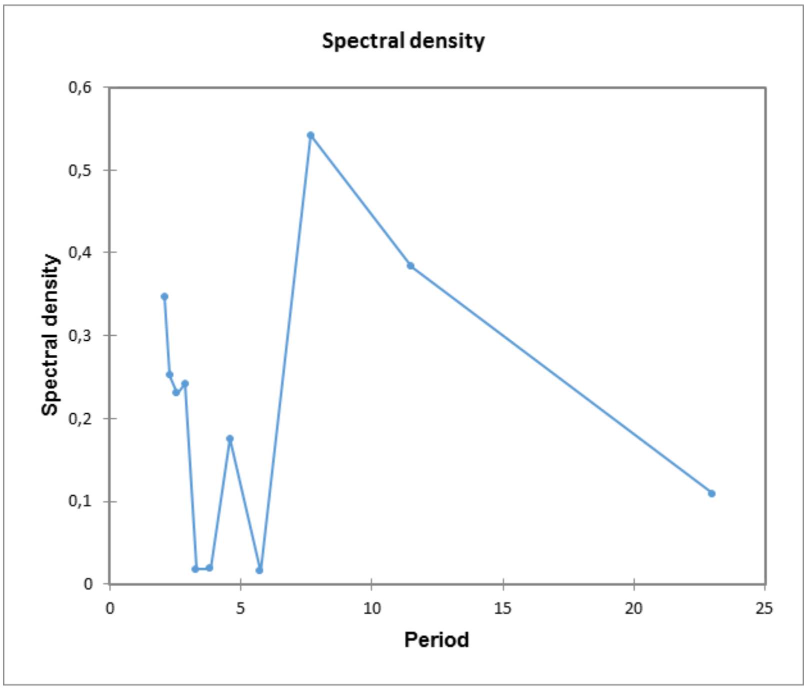

Figure 3. 14: Period of spectral density dam failures. All dams

The period when there is a change in the patter or activity of the dam failures varies in a regular cycle of 7.6 year within the years 1996 to 2018.

3.3.3 Comparison results

Some of the results obtained in the previous sections are compared with the corresponding ones published by ICOLD Bulletin 99 and presented in the Chapter 2 with the aim to find or not correlation and possible causes between the surveys.

Due to the statistics in the ICOLD Bulletin 99 are referred to large dams and the figures were graphically obtained, so an error of ±2 can be expected. They are compared only with the dams higher than 15m registered in the created database after 1995.

Table 3. 13: Comparison by height of the dam

	ICOLD Bulletin 99		Database after 1995	
Height [m]	**No. cases**	**Percentage**	**No. cases**	**Percentage**
15 ≤ h < 30	123	74%	13	47%
30 < h < 60	38	22%	9	32%
h > 60	7	5%	6	21%

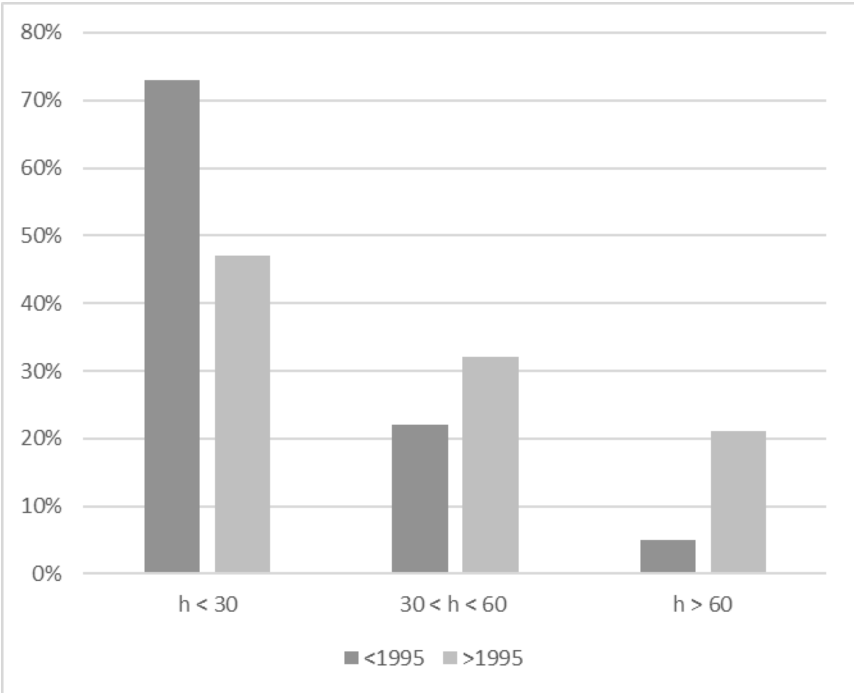

Figure 3. 15: Heights comparison in percentages

The ratios between the failed dams of height over total failed dams and the existing dams of height over total existing dams before 1995 indicate according the ICOLD Bulletin 99, that there is not a significantly variation because of the dam height and failures.

In the case of the failures collected in the database after 1995. The tendency is similar with a greater relative number of dam failures when they are smaller.

Although, the percentage of failures is almost 4 times for the created database (19%) compared with the ICOLD (5%) for dams higher than 60m.

It should be considered, in general the pattern indicates that the number new constructed dams is decreasing. Hence, the ratio can vary.

Table 3. 14: Comparison by age of the dam

	ICOLD Bulletin 99		Database after 1995	
Height [m]	**No. cases**	**Percentage**	**No. cases**	**Percentage**
During Const.	13	8%	3	11%
0 - 1	46	28%	3	11%
1 – 5	34	21%	2	7%
5 - 10	15	9%	2	7%
>10	55	34%	18	64%

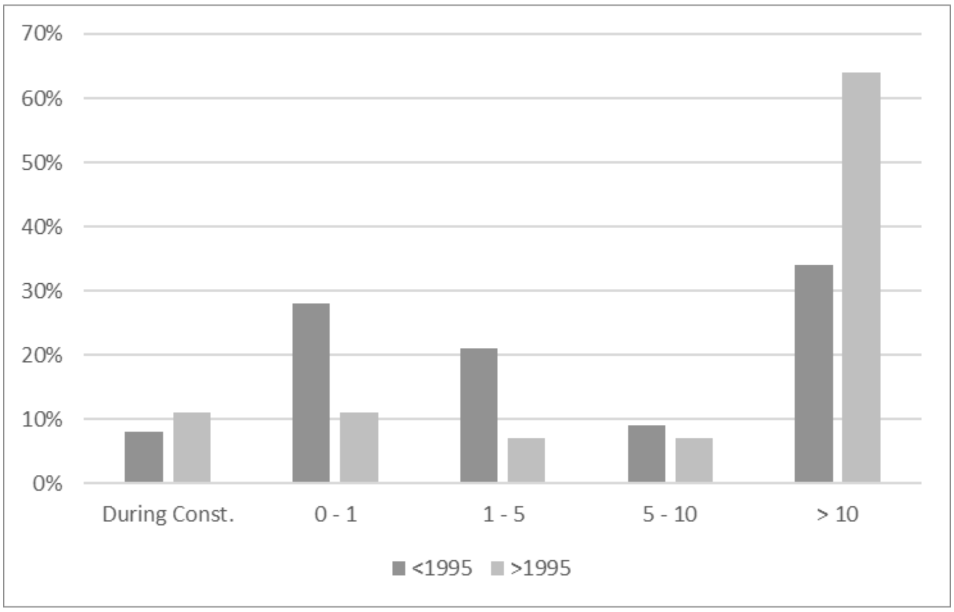

Figure 3. 16: Age comparison in percentages

The table and graph show a difference in the statistics before and after the ICOLD Bulletin 99.

It was noticed by several autors and sources, that the critical period or failures is within the first 10 years, and to be more precise, either during the construction, first filling and/or first year of lifespan. Comparing the figures during construction, they are pretty much the same. The difference comes with the significant reduction for the first year, falling the percentages from 28% to 11%. Omitting the possibility of lack of cases in the created database. The reason of such reduction (chapter 2) can be due to the current awaredness of the hazards during this period, better construction methods and material, upgraded regulation, more and higher quality monitoring techniques, etc.

The next numbers that get attention, are the corresponding to the failures after 10 years. The decrease of almost half of the percentage could confirm what it was stated in the previous paragraphs. Those dams failed between 1996-2018, but are older, so the implentations may not be totally applied when they were constructed.

The curve of failure shows an increase of the probability of incident when the dam is getting older.

To sum up, the percentage before 1995 for dams that failed between the first 10 years is a 66%. On the other hand, for the failures after 1995 for the same period is a 36%.

As it was mentioned, the data from China and Russia (former USSR), were not collected and included neither in the created database and in the ICOLD Bulletin 99. Nevertheless, two graphs are added with information that can be treated as simple reference without enter into further details.

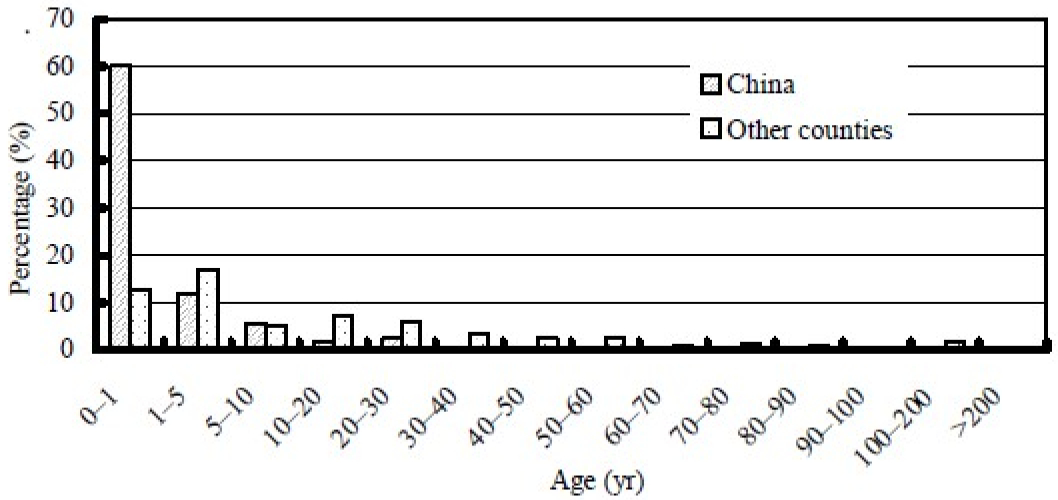

Figure 3. 17: Dam failure for different service age. China and the world (X.Y He et al, 2008)

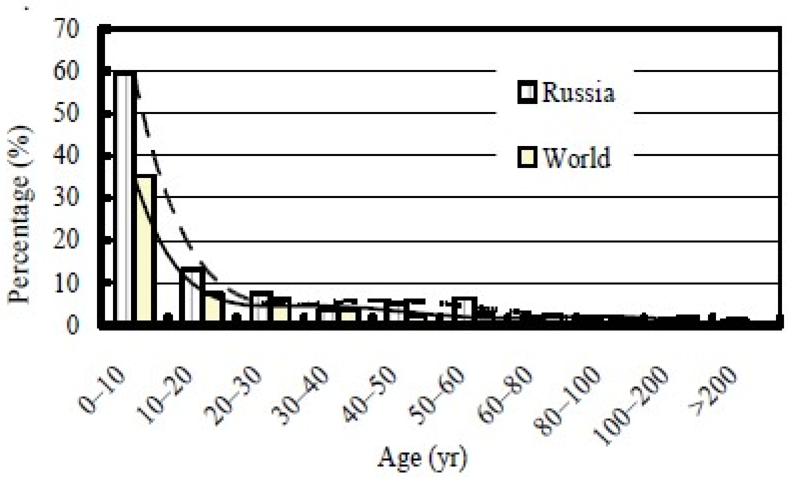

Figure 3. 18: Dam failure for different service age in Russia and the world (X.Y He et al, 2008)

The graphs show that approximately, in China the percentage of failures before the first 10 years is around the 77% and in Russia about the 60%. Which are in the range of value of the 66% of the ICOLD and far away of the 36% from the created database after 1995. In any case, the tendency shows grossly the same, during the "infant period" the probabilities of dam failure are higher.

4. Lessons learned

The division between failures occurred before the 50's and afterwards has several reasons as it has been exposed in former chapter. Like the quality of the materials, the construction procedures, the design criteria, etc.

Prior the beginning of XX century, the techniques for determining design floods were not sophisticated enough [4a] until the turning point in the middle of the century. At that time better tools were developed with the purpose to obtain a more accurate account for flood loadings and consequently, the dam's performance has clearly increased.

These tools have been of a significant help in order to decrease and prevent the number of failures, with implementations in the storage and discharge capacity to face flood events. For instance, by overtopping, that is together with seepage the main reason for dam embankment failures, and at the same time this typology of dams has the highest rate of incidents.

Coming up next, a list of lessons learned from dam incident and failures is presented. Most of them were obtained from [4c] after a deep analysis of many cases registered mainly in the USA.

The lessons learned are classified according to the approach followed in the present work. However, some lessons are difficult to catalog either during construction, after construction or during operation. Therefore, a general common category was added which include most of the lessons and recommendation which implies human factor that indeed is the greatest reasons of dam accidents. Some also can be placed in more than one table.

Table 4. 1: Lessons learned. Design and general

Lessons learned
Emergency Action Plans can save lives and must be updated, understood, and practiced regularly to be effective.
High and significant hazard embankment dams should have internal filter and seepage collection systems.

External independent peer review of designs and decisions is an effective means of providing quality assurance and reducing the risk associated with design oversights and deficiencies.
Filters and drains for embankment dams must be compatible with adjacent fill or in-situ materials.
Safety should not be sacrificed for cost.
Earth/rock cut spillways can fail by erosion and breaching and should be evaluated for integrity during passage of the design flood.
PMF magnitude floods do occur.
The use of corrugated metal pipes in embankment dams is discouraged.
Concrete gravity dams should be evaluated to accommodate full uplift.
Pumped storage embankment dams should have an emergency spillway or other redundant features to prevent overfilling and overtopping of the embankment.
All dams need an operable means of drawing down the reservoir.
Many dam failures are the result of a combination of events and can include human, natural, or structural factors.
Intervention can stop or minimize the consequences of a dam failure. Warning signs should not be ignored.
Dam owners need to address public safety at dams.
Hazardous hydraulic conditions such as hydraulic rollers can occur at dams of all sizes.
Dams located in areas of potential seismic activity need to be evaluated for liquefaction, cracking, potential fault offsets, deformations, and settlement due to seismic loads.
Alkali-aggregate reactions (AAR) can cause serious concrete deterioration and other problems at concrete dams.
Most concrete dam failures are the result of foundation stability problems. Concrete dams founded on bedrock require subsurface investigations and testing of rock properties.
Outlet works and pressurized conduits in embankment dams should be provided with a means for upstream closure.
Most dam incidents and failures can be attributed to human factors.
Geotextiles should not be used as the primary means of filtration in drainage systems for dams.

To the extent practicable, dispersive soils should not be used in the construction of embankment dams.
For high and significant hazard dams, elements that are critical for the safe operation of the dam should be designed using the "belts and suspenders" approach.
Structural underdrains can lead to internal erosion under or along stilling basins and conduits.
Non-plastic soils are far more likely than soils with some plasticity to experience internal erosion.
Cutoff walls can concentrate gradients and seepage flows
Karstic foundations are difficult to treat and can cause internal erosion issues.
Hard rock spillway channels and hard rock concrete dam foundations may erode under the right flow conditions combined with unfavorable joint orientations and other joint characteristics.
Dams may overtop for floods more frequent than the design flood if the spillway capacity is reduced (due to debris plugging or gate malfunction) or if a gated spillway is not operated as assumed in design studies.
Spillway gates should be tested to the maximum extent possible on a regular basis to verify the performance of the electrical/mechanical system and to ensure that the gates can travel freely without binding or being restricted.
In order to reduce uplift pressures in sand aquifers overlain by clay layers in earthfill dams, granular filters installed directly on the underlying foundation can be an effective solution.
Raising the normal pool or crest of an existing dam requires evaluation of existing and new potential failure modes as well as the stability of the dam for the new loading condition. Dam modifications need to be designed by a professional engineer.
Prior to cement grouting of rock zones that are highly permeable and have high piezometric gradients, it is important, when possible, to reduce the flow and the piezometric gradient in the area where the grout is to be injected.
Emergency Action Plans for dams should include a list of available resources available (materials, equipment, contractors, etc.) to help respond to emergencies.
At dams where failure or near failure has occurred, proper mitigation and/or rehabilitation should be performed to ensure that similar events cannot recur.
Pressurized pipes in embankment dams require special design features.
Safe access to high and significant hazard dams at all times is important.
The foundations for an arch dam should be properly treated by replacement of poor rock with concrete, grout-

ing, and drainage.
Because of their need to transfer load to surrounding rock, the abutments and foundations of arch dams require special geologic exploration, evaluation and treatment.
It is prudent to provide very high discharge capacity for seepage collection and control systems for dams constructed on glacial foundations.
A properly designed and executed foundation grouting program can provide an effective seepage cutoff.
Outlet works conduits with joints in embankment dams should be watertight to prevent internal erosion of the embankment.
Foundation approval should be documented by designers and geologists.
Effective communication with emergency responders is important when responding to a dam emergency.
Landslides around the rim of a reservoir can cause dam overtopping. The stability of the hillsides around the rim of the reservoir as well as the impact of potential slope failures should be evaluated.
Dams have confined spaces that need to be marked and only entered when it is determined to be safe and the entry is in compliance with confined space entry and monitoring procedures.
Drainage conduits for dams should be designed with cleanouts to accommodate future cleaning and inspection.
Peer review and collaboration during dam design can prevent design-attributed failures.
Concrete dams should be evaluated for and protected from foundation and/or abutment scour.
The presence of weak zones in natural foundation materials must be evaluated in design.
Brittle materials, such as asphalt and Portland cement concrete, should not be used as reservoir liners without high capacity filtered underdrains.
Zones of permeable soil, such as old riverbed deposits in dam foundations, should be addressed.
Grouting of earth fill or overburden material should not be used as a long-term/permanent solution to prevent internal erosion.
Dams constructed by hydraulic fill methods can be susceptible to liquefaction during seismic events.
Flooding resulting from a dam failure may extend further than noted in inundation maps.
Spillways should be designed to prevent clogging by debris.
Excessive headloss in inlet piping can be a failure mode.

Grout curtains in a formation where potential seepage paths (joints, fractures, etc.) are filled with either erodible or soluble materials are not permanent and may require periodic maintenance grouting to remain effective.
Conventional instrumentation can induce or accelerate internal erosion.
Compatibility of materials to prevent piping of materials through embankment and foundation materials needs to be considered. Special consideration is necessary for broadly graded and variable soils deposits (glacial, alluvial, and colluvial).
Critical intervention/action by personnel during a crisis can be precluded by: (1)Power failures preventing use of key equipment; (2)Site inaccessibility due to flooding or road damage; (3)Required travel time; (4)Limited staffing
Current state-of-the-art two-dimensional modeling combined with LiDAR terrain data may identify areas of inundation for inclusion in the inundation mapping not identified using one-dimensional modeling.
An emergency alarm/warning system can be an effective means of providing warning to downstream residents located in close proximity to a dam where the warning time is short
Principal spillways should be designed to convey high frequency floods without incurring significant erosion damage.
Emergency design and construction of dam modifications requires proactive involvement between the owner, engineer, regulatory agencies, and the contractor to achieve the most effective combination of constructability, conservative design, and flexibility.
Stress distribution in an arch dam is critically influenced by excessive deformation of the foundations.
Extensive and progressive cracking can seriously impair the safety and durability of a thin arc dam.
In order to reduce tensile stresses in thin arches, the structure should be thickened toward the abutments and river bed.
For new dams, weak features in rock foundations can be mitigated using concrete shear keys and dental concrete.
There is magnification of seismic accelerations not only in the upper parts of a concrete dam but also along its abutments.
For dam safety regulatory programs, it is important to have a reserve source of funding identified for responding to an emergency.
Dam systems that rely on automated computerized systems or electrical power to operate require redundan-

cy.
Supporting a dam on piles makes it more vulnerable to foundation erosion and piping.
Static equilibrium, including uplift pressures, must be considered during all phases of construction and service.
Loose rockfill should not be used as structural support in locations where it can be easily eroded.
Gravity dams on soil foundations require special features to address potential erosion of the foundation.
Failure of upstream reservoirs should be considered in the design of closely spaced dams.
Differences in the stiffness of structural materials can lead to unforeseen stress concentrations.
Unique force systems must be anticipated when designing foundations on rock having complex systems of faults and joints.
Reservoir impoundments can reduce the stability of natural slopes.
Design features that rely on specific operating procedures need to be documented and clearly communicated.
Dam failure by acts of terror should be considered as potential failure modes.
Special care is needed in placing embankment materials in order to prevent preferential seepage paths due to material variability, segregation and partial fill surfaces.
Proper development of site-specific loading diagrams and related assumptions are an important aspect for any dam design. Key assumptions include selection of dam and foundation material properties that are defensible.
Due to inherent uncertainties relating to both design and construction, appropriate factors of safety need to be applied.
Modern computers and software allow engineers to reasonably determine the potential consequences of dam failure. This has helped heighten the awareness of potential dangers as well as better equip stakeholders in emergency preparedness and response.
An important benefit of active involvement in professional societies is their forum for the exchange of information related to new technology as well as to practices which should be avoided.
Remote sensing/controls should have redundancy, and preferably via a system of a separate nature, such as visual (camera) reconnaissance to confirm reported conditions.
Low-level outlets for normally dry flood control/stormwater dams must be designed to prevent blockage by

beavers in regions where they reside.
Improperly designed or anchored spillway slabs may fail long before a spillway discharges at its designed maximum capacity.
Quality control of waterstop installations is critical to assure adequate embedment of waterstop materials into the concrete on either side of joints.
Fish containment structures must be located and designed such that they function properly in extreme runoff events without limiting the outflow capacity of the dam's spillway.

Table 4. 2: Lessons learned. During construction

Lessons learned
Seepage along penetrations through embankment dams should be controlled using a filter diaphragm instead of anti- seep collars.
A concrete encasement or cradle around or under conduit penetrations through earthen embankments allow for better compaction of earthfill.
Treatment of fractured foundation rock for embankment dams is important to prevent internal erosion of embankment material that is in contact with the foundation.
Stability of the dam foundation and other geologic features must be considered during dam design.
The design engineer(s) should be involved in the construction phase of dam projects.
Poor compaction, steep rock abutments, and other foundation discontinuities at embankment dams can lead to differential settlement, cracking, and failure by internal erosion.
Coordination between geologists, designers, and contractors is important.
Moisture content and compaction of embankment fill material must be carefully monitored for a acceptance during construction.
Special attention must be given to the compaction of earth fill around discontinuities such as along outlet structures and other penetrations through embankment dams.
Construction inspection is critical in assuring proper construction techniques and construction in conformance with approved plans and specifications
Lime treatment can be used to stabilize dispersive soils used to construct embankment dams.
Dynamic compaction can be an effective method to correct seismic deficiencies due to liquefiable embank-

ment materials in earthfill dams.
Grouted interlocking keys are an effective means to construct contraction joints in arch dams.
Vertical interlocking keys in the transverse contraction joints are highly effective in maintaining stability of individual blocks of an arch dam during an earthquake when the reservoir is partially full.
Poor design and construction practices increase the probability of a dam failure.
Sound engineering is required to build a safe dam.
Delays during dam construction can expose a dam to higher risk of failure.

Table 4. 3: Lessons learned. After construction

Lessons learned
The first filling of a reservoir should be planned, controlled and monitored.
For new dams, the as-built conditions of critical elements should be verified through direct observation and/or testing. This is particularly important for critical elements that are constructed "in the blind."
Rapid filling of clay dams after long dry periods should be avoided.
Fault activation and subsequent dam failure can be caused by high-pressure injection of fluid into subsidence-stressed subsurfaces.
Injecting air into flow discontinuities caused by small surface discontinuities and changes in high velocity flow direction can prevent cavitation.
The evolution of the pore water pressure must to be monitored
Use of a check-list during the first filling process
Verify leakages and permeabilities.

Table 4. 4: Lessons learned. During operation

Lessons learned
Uncontrolled vegetation on and around dams hinders effective dam inspection and can lead to serious structural damage, significant deferred maintenance costs and dam failure.
Regular operation, maintenance, and inspection of dams is important to the early detection and prevention of dam failure.
Engineers performing dam inspections and assessments should have a sound knowledge of the as-built features and common design practices in use at the time of dam construction to help them identify potential failure modes and other vulnerabilities.
In emergency situations, lowering the reservoir can be an effective means of risk reduction.
Many years of successful dam performance does not guarantee future successful performance
High and significant hazard dams should be designed to pass an appropriate design flood. Dams constructed prior to the availability of extreme rainfall data should be assessed to make sure they have adequate spillway capacity.
Warning time and rapid response is critical to saving lives during a dam failure emergency.
Gates and other mechanical systems at dams need to be inspected and maintained.
Dams need to be regulated by state or federal agencies.
The consequences of intentional controlled releases from dams need to be understood and carefully executed.
The hazard classification of a dam can change over time (hazard creep).
Dams should be inspected and monitored during and/or after large rainfall, seismic, or other unusual loading events.
Dams built from or on loose granular soil can be susceptible to liquefaction of the foundation material.
Settlement of earthfill embankment dams can reduce freeboard and increase the risk for overtopping. New embankment dams should be designed with allowances for settlement.
Discontinuities in grass-lined spillways can accelerate erosion and lead to significant damage or complete breaching of the spillway.
Modifications to spillways can unintentionally decrease their capacity. Any modification to a spillway should be reviewed and approved by a professional engineer.

Internal erosion can occur at relatively low hydraulic gradients.
Internal erosion can take decades to progress.
Concrete dams on rock foundations are not immune to failure during flood overtopping.
During floods that greatly exceed the design flood, spillway failure (due to exceedance of the spillway capacity) may be a more critical potential failure mode than dam overtopping.
Dam operators may operate a gated spillway at release levels less than those specified in the SOP if there is the potential for downstream consequences.
Dam performance and conveyance capacity should be re-assessed following significant changes in land use within the contributing watershed.
Maintaining dam records such as as-built plans, construction data, reports, diaries, previous studies, inspections, instrument readings, maintenance records, etc. is important.
Some dams may not have included clean out of the stream channel under the embankment. This can be a source of uncontrolled seepage under the embankment.
Aeration of the underside of the nappe is often needed to mitigate vibrations in spillways.
Siphons can be an effective method to lower reservoir pool levels during emergencies when outlet works are inoperable, undersized, or missing.
Conducting potential failure modes analyses (PFMAs) and periodic assessments of high and significant hazard dams are effective methods that help keep dams safe.
Under certain conditions at a dam site, flows exiting spillways can double back and erode and breach the dam embankment. Dam designers and inspectors need to be aware of this potential failure mode.
Uncontrolled animal activity can contribute to or result in a dam failure.
Trash racks need to be appropriately sized and cleared after large flow events. Outlet conduits at dams need to be inspected regularly to confirm their structural integrity and conveyance capacity.
A well maintained grass cover can prevent damage and breaching of embankment dams during minor overtopping events.
Seemingly small changes to dam details can cause failure of a dam.
Dam repairs should be executed promptly.
Conventional instrumentation may provide early detection of conditions within a dam that could lead to failure.

Flood control dams may not provide an opportunity to observe developing seepage and piping.
Severe piping of embankment materials can occur through a dam into the abutments or its foundation without any deficiencies being observed by dam inspection.
Subsurface inspection of the downstream toe area of run-of-river dams or any spillway structure that is normally inundated by tailwater should be conducted periodically to detect scour and possible undermining of the dam or spillway structure.
Foundation and formed drains for concrete dams should be inspected regularly and their performance verified by drain flows and uplift pressure measurements. When drains begin to plug and performance is reduced, a cleaning program should be initiated.
Installation of an under-slab drainage system can help avoid detrimental frost action while providing a method for monitoring seepage quantities over time.
Moments and stresses caused by friction within the trunnion pin connection must be considered in the structural design and maintenance of radial gates. Trunnion pin connections must also be lubricated and maintained in accordance with design assumptions.
Dam instrumentation can provide early detection of a dam safety problem and can provide an opportunity for intervention.
Reservoir sedimentation can increase destabilizing forces acting on a dam and have other detrimental effects that could contribute to a dam failure.
Reduction in minor principal stresses due to drying or differential settlement can lead to hydraulic fracturing and piping failure in embankment dams.
Erosion through a breach in erodible fill cannot be controlled.
A very well-maintained dam is not a guarantee against failure when a dam and its design is dated and does not meet current design standards.

The knowledge of the possible hazards that could play a role in the dam failure or its malfunction is vital. This implies the fully understanding of the failure modes, the possible effects of the combining of hazards, probability of occurrence in order to evaluate a plan measures to counteract or minimize them and likewise the consequences of a predicted failure. A very useful tool to identify and preview likely unfavorable scenarios are the fault trees and event trees tools for assessing probabilities of the outcomes and overall system analysis.

5. Safety aspect of Austrian dams

5.1 Safety concept

The Austrian safety concept is based on three fundamental principles: Design-Construction; Surveillance and Preparedness for emergency. The target of the principles is to reduce and minimize the consequences associated with the dam operation and to control the remaining risk [6c].

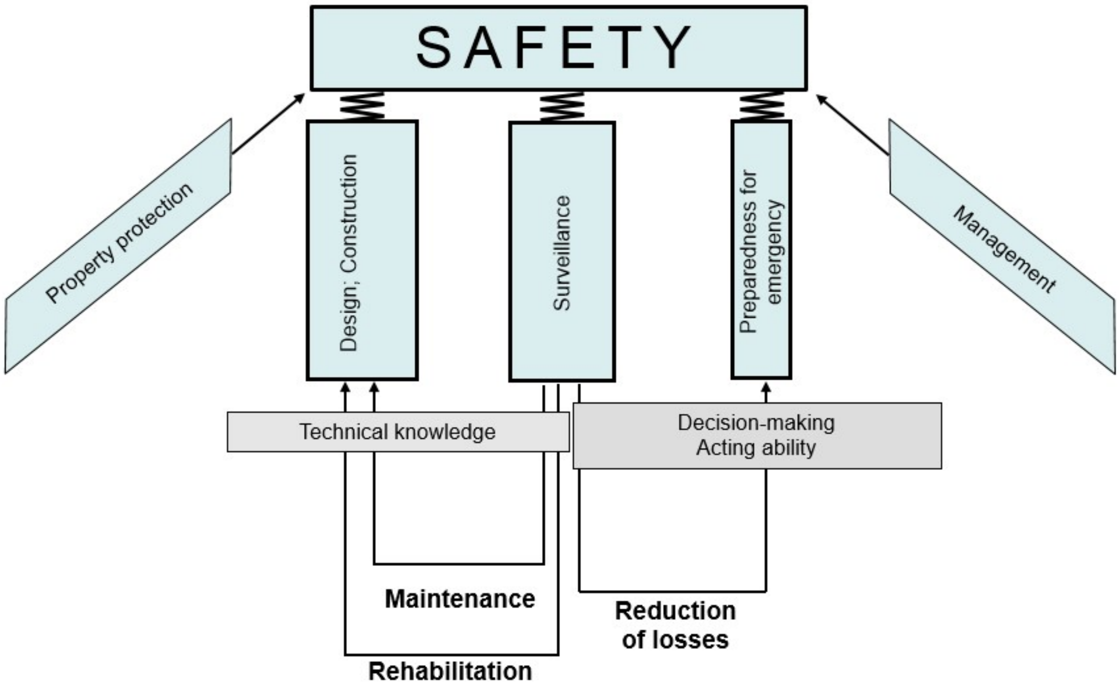

Figure 5. 1: Safety concept (ATCOLD Dam Safety Expert Seminar R. Melbinger, 2018)

For the Design and Construction principle, the goal is to ensure an optimal design of a dam. All possible operational conditions must to be evaluated, like maximum load by earthquakes or severe flood events.

The surveillance or monitoring principle aim is to identify and detect structural defects or hazards with enough time in advance to be able to take measures about. It includes monitoring devices, visual inspections and periodic report by responsible authorities and qualified professionals.

The preparedness for emergency principle goal is to respond effectively to hazards. There are 3 possible scenarios. First one, is when the hazards can be controlled. The second one, corresponds to a situation when there are doubts if the situation can be controlled, so the alarm system must to be prepared. The third one, is when a dam break cannot be avoided and alarm and evacuation plans are activated.

The definition of risk can be expressed as the probability of the occurrence of the undesirable event or probability (P) and the extent of the damage caused (D) [6c].

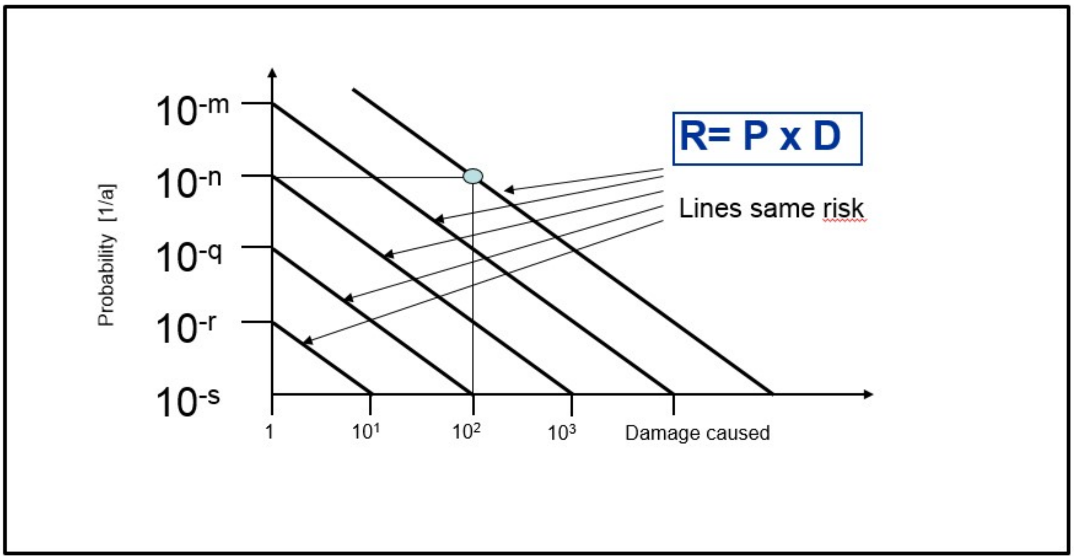

Figure 5. 2: Risk (ATCOLD Dam Safety Expert Seminar R. Rißler, 2018)

The probability can be assessed as: regular, often, uncommon and rare.

The extend of the damage as: catastrophic, considerable, low and insignificant.

In Austria, the safety aspect of dams is regulated by the Federal Water Law with specific requirements for dams higher than 15 m or with impounding reservoirs with more than 500,000m^3 [10b].

When the construction of a new dam is projected within the category explained before, the project is analyzed and checked by the Austrian Commission on Large Dams (ATCOLD). The standards about dam safety in the country stipulates that for each dam it must to be fulfilled certain requirements for operation. It is also expected that they comply with the current state of the art. Some of the requirements are:

- The owner of the dam has to appoint a Dam Safety Engineer (DSE) and deputies to ensure permanent on-call service.
- Yearly inspection by the DSE and the Dam Safety Officer.
- Every year it has to be submitted and signed by the DSE a report about the dam safety to the Federal Dam Supervisory Section.
- Every five years there is an inspection by the Federal Dam Supervisory Section.
- For each dam, there is an Emergency Action Plan (EAP).

The Supreme Water Authority that depends on the Federal Ministry of Agriculture and Forestry (FMAF), is the institution with the maximum jurisdiction about large dams in Austria. Likewise, the FMAF has a Federal Dam Supervisory Section (FDSS) that is in charge with the assistance of the ATCOLD to check the annual reports about the dam safety status that are provided by the dam's owners [5a].

The FDSS has the responsibility to supervise the construction of the dams, checking that all the standards and features approved are fulfilled.

As it was mentioned in the present work, the first filling is a delicate moment in terms of dam stability and security, due to this, the FMAF, has to authorize the filling of a dam, with a preliminary technical examination of the dam and afterward, when enough data has been collected and judged, a final examination is carried out with a final degree of acceptance to the normal operation of the dam. Usually, after a couple of years in operation with impoundment/drew down cycles and the collected data can be considered reliable and safe.

The monitoring and surveillance include stipulated periodic visual inspections, measurements and data collection. After 10 years of operation, the reservoir is drawn down in order to accomplish a comprehensive inspection for an overall safety assessment.

The owner of a dam or hydropower plant has the obligation to keep the public informed about potential risks and their associated consequences in case of failure and in order to ensure the public safety the specific measures are stated into the operating instructions, which cover the possible scenarios and how to proceed in case.

Some of the precautions into the facilities, include the installation of barriers and security measures to sensible facilities like fences, provision of rescue equipment, warning signs, inspection of downstream before an outlet opening or regulation of the discharge and public information and communication. In addition, the dam owner hast the primary responsibility for dam safety and the dam safety engineer must to be informed of all extraordinary events.

The high standard of dam safety in Austria is based on a multilevel principle. Where one level not relies on the other, it should look rather at the security issue from multiple points of view, so redundant, and this prevent the danger of a certain hazard.

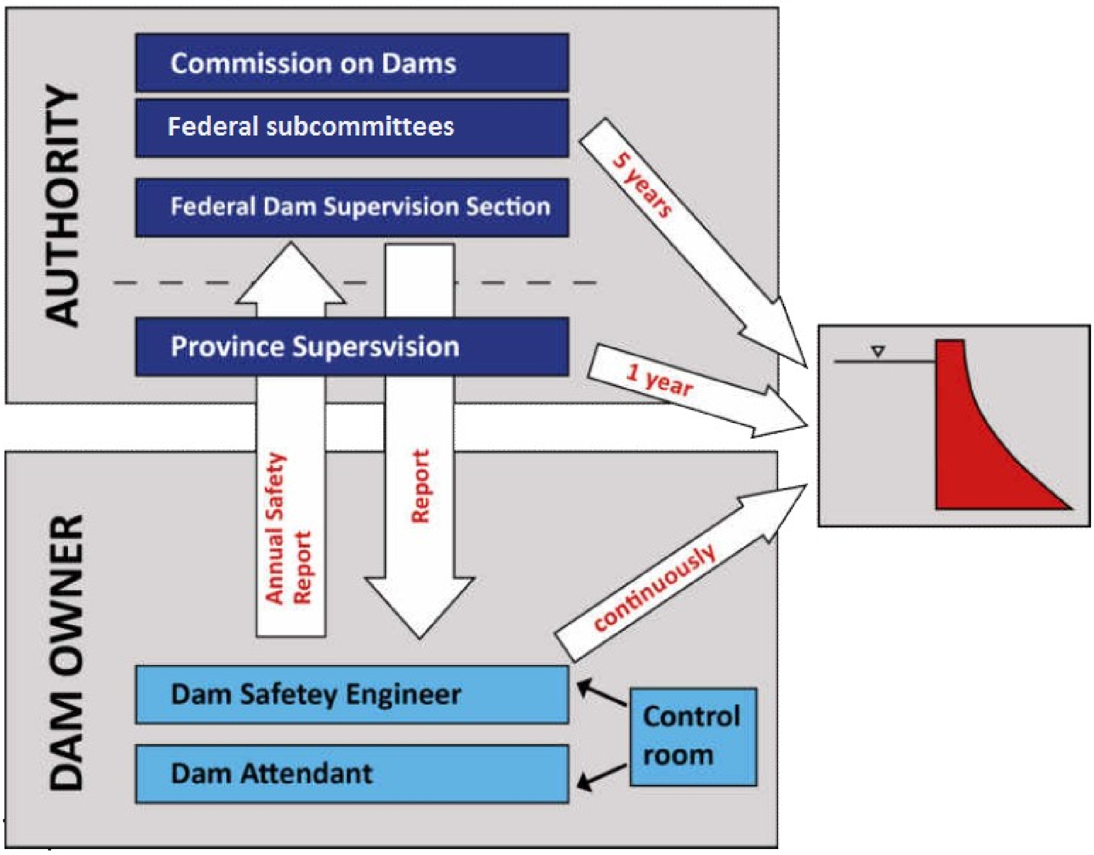

Figure 5. 3: Multi-Level Principle (ATCOLD Dam Safety Expert Seminar R. Melbinger, 2018)

In Austria, there is an important concept of the proper and specialized formation of the professionals involved in dam safety. In order to be prepared to face current and future challenges [11b]. The figure of the dam supervisor has a very relevant role in the Austrian dam safety system.

The dam supervisor is responsible for all surveys and measures for the safety of the dam. In addition, of the monitoring activities, the dam supervisor must to carry out or arrange the activities for the secure operation and maintenance in the long term of the facilities. The dam supervisor also must to have enough power or hierarchy in the organization in order to be able to take appropriate decisions.

The profile of a dam supervisor should fulfill the requirement shown below:

- Belong to the technical management of the company
- Have a specialized education and at least 10 years of experience in the field of dams
- Be familiar with the system
- Have a high degree of reliability
- Own an appropriate level of authority
- Be available and reachable within a reasonable time

The formation of a dam supervisor does not finish when the engineering degree is accomplished. It is a matter of life-long learning during the whole professional performance.

In order to achieve a successful dam safety plan, there are some concepts that have to be kept in mind, which are shown below

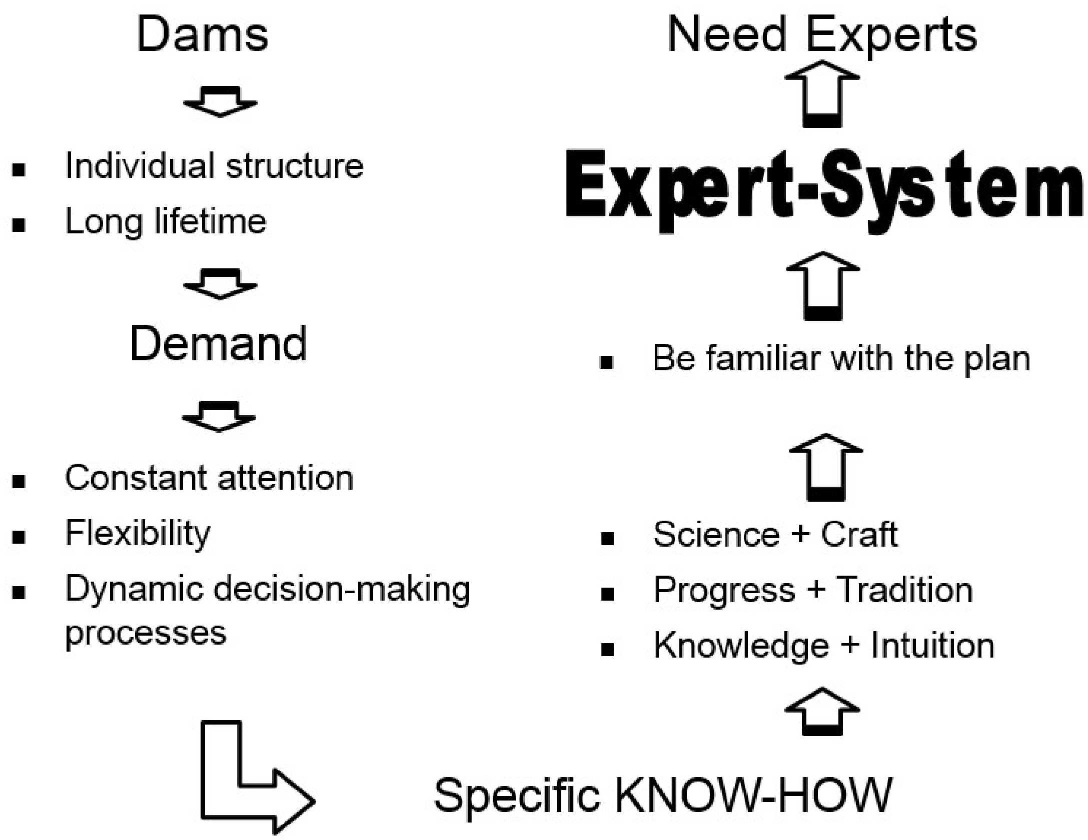

Figure 5. 4: Expert-principle (ATCOLD Dam Safety Expert Seminar R. Melbinger, 2018)

5.2 Safety basic principles

A comprehensive detailed list, with twelve basic principles, with the purpose of an adequate safety of Austrian dams was developed by R. Melbinger [12b]. Each point is briefly presented.

1- The safety must to be high

The consequence of a dam failure can be catastrophic, therefore the design must consider loads that are very unlikely, like the MCE or HQ5000.

A high security requests a comprehensive and intensive monitoring.

2- No safety without monitoring

Deformation and other anomalies in the structure or surrounding must be detected quickly. Therefore, the potential hazards can be managed in time, even with sudden scenarios like earthquakes.

Surveillance activities should discover signs of change or symptoms within an adequate time.

3- Measuring alone is not sufficient

The experience has shown that some anomalies were detected through close observation, even before that they were registered by monitoring devices. The devices and equipment must be checked periodically. The scope and intensity must be included in a measurement program.

The results of the measurement must be analyzed since a critical view. Considering all scenarios, even malfunction of the devices.

4- Any dam without supervision

A dam must to be considered like a life structure, changes can only be detected if the observer is familiar with the facility. The responsibilities must to stay clearly divided into small group.

5- Without dam's responsibilities, doesn't work

The dam supervisor must to be an experienced professional able to take decisions and assess the situations in a short time.

6- Communication must be

The transfer of information and instructions must be reliable and comprehensive specially for the next reasons:

- The dam supervisor can be able to decided. The monitoring data are sent quickly to him/her
- For extraordinary events like extreme weather situations, like phone lines interrupted, networks overloaded. Therefore, it must be redundant to ensure communication
- Communication also means that everyone involved understand each other. Therefore, it is essential that the staff meets regularly to update information and know each other

7- Documentation is troublesome, but necessary

The information about the dam must be easily and quickly available for all the staff involved in the security of the dam. It must be considered:

- The useful lifespan of the dam
- The large number of safety activities and monitoring results related to the dam that have been performed during this time
- The number of persons involved and prevision of retirements

The dam book must be established and continuously updated. With each activity planned.

8- Automation is not all

The automatic detection, transmission and processing essentially relates only on measured data. Exceeding permissible limit values can be reported automatically, but this is only valid when:

- Reliability and availability of the automatic systems are high
- Automatically generated alerts reasonably quickly by a competent professional to be accepted and processed

A robust and reliable component must be installed, with a low rate of failure and considering that in case of failure of these devices, it can be replaced by another system quickly. Redundant system must be available.

Automatic system must be supported by staff. For instance, for the calibration, visual controls, etc.

9- Private plus state

The responsible federal authority periodically verifies that the owner is properly performing its duties of monitoring and maintaining tasks of the structure. They check the yearly reports provided by the owner and every 5 years with inspection on site by the FMAF.

The participation of state organs in the safety of the dams is to achieve an integral security system, and it should remain a political goal and an issue of public interest (see Figure 5.1).

10- Central plus federal

Providing the required specialist know-how at the federal level ensure:

- Continuity and sustainability
- An Austrian federal wide uniform view and procedure
- The most economical solution

The dam supervision at federal states has the advantage of greater proximity to the facilities and thus faster availability than a central one. For example, in case of a special incident, the reaction capacity is faster. Therefore, a meaningful cooperation or division of labor is necessary between federalism and centralism.

11- Individuals make experts

In the design and operation of a dam system must to be involved professional and experts from different fields of knowledge, science and engineering aiming correct diagnosis. It leads to the already explained concept of “expert principle”. This principle allows to have the necessary flexibility and capacity of adapting to individual cases.

12-No specialist falls from the sky

The design and construction of a dam is a synthesis of science and craft, of innovation and tradition. In contrast with the technical knowledge, that could be gathered relatively fast, to get familiar with individual structure with its very special question takes some time. In case of a foreseeable change in staff, the new members of the staff must to be trained with enough time in advance and transfer the knowledge. The ATCOLD and Universities also play an important role in order to achieve it successfully.

5.3 Risk-informed framework

Austria has adopted in order to assess the risk and take decisions concerning dam safety and associated consequences, a comprehensive and worldwide used method to approach risk categories (ICOLD Bulletin 154) [6a].

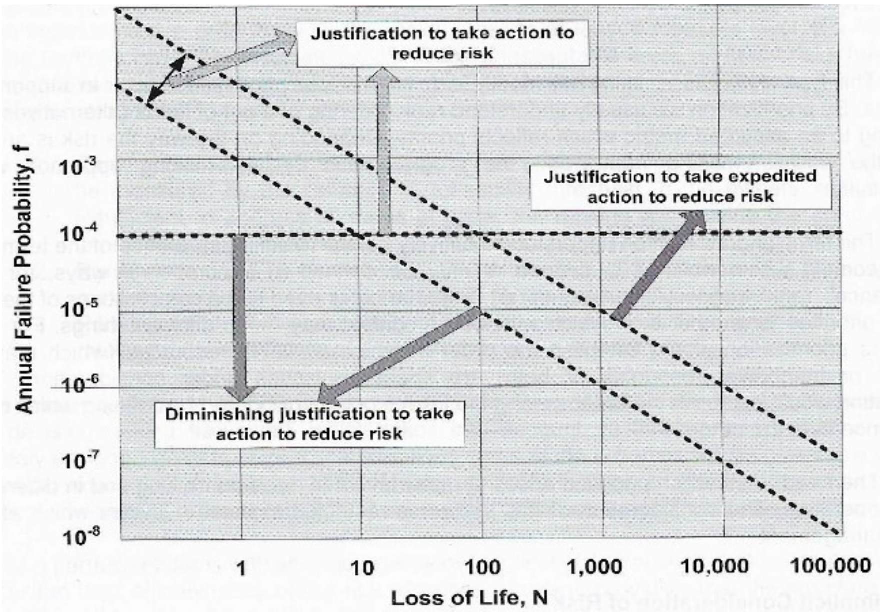

Figure 5. 5: Annual Probability of Failure and Annualized Life Loss (Munger, 2009)

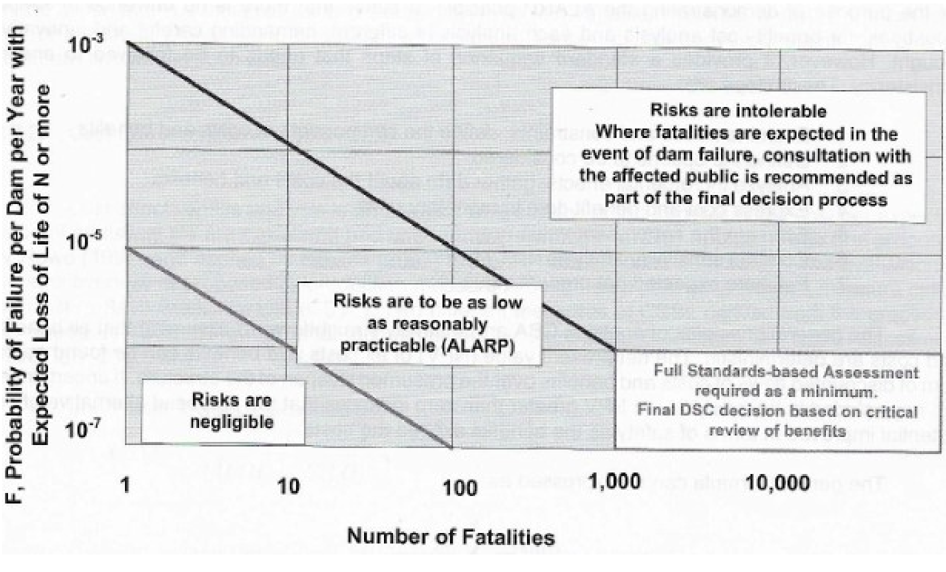

Figure 5. 6: Societal Risk Requirements for Existing Dams (NSW Dam Safety Committee, 2010)

The figures 5.4 and 5.5 show the categories explained below [6a].

- Broadly acceptable risk–Annual risk of casualty significantly lower than 10^{-6} arising from any particular source, generally taken as negligible risk

- Unacceptable risk-An annual risk of casualty in excess of 10^{-4} deemed to be intolerable under normal circumstances. This does not preclude individuals from voluntary participation in recreational activities involving higher levels of risk, often in the range of 10^{-3} to 10^{-2} fatalities per year

- Tolerable risk–An annual risk of casualty between the values 10^{-6} & 10^{-4}

The society risk is the probability of an event that could result in multiple casualties, it should not exceed a value, which is a function of the number of possible casualties and which is declining as the number of casualties increases.

Social risk deals with hazards from engineered installation where the predominant issue is life safety, so it is characterized by frequency-number (F/N) curves. These graphically display the potential for multiple fatalities by relating cumulative frequencies or probabilities (F) against number of casualties (N).

The term ALARP (As Low As Reasonably Practicable) is applied to ensure that the residual risk should be as low as reasonably practicable, because the cost of a residual zero would implies the use of infinite economic and time resources. The use of this approach is a useful tool for the programmatic decision making in terms of dam safety and dam risk evaluation.

6. Dam surveillance & monitoring

6.1 Monitoring

An appropriate maintenance program in the structural, mechanical devices and facilities in general is vital to avoid a malfunction or failure prevention in dams. The dam surveillance consists in one hand in visual inspection and also at the structural monitoring where exist a wide range of old and new methodologies and gauges available.

The reputed Swiss engineer Lombardi stated four questions for the dam evaluation, which are:

1- Does the dam behave as expected or predicted?
2- Does the dam behave as in the past?
3- Does any trend exist which could impair its safety in the future?
4- Was any anomaly in the behaviour of dam detected?

In this chapter different methods and current researches made by several authors are briefly presented and discussed regarding monitoring techniques and concerns about climate change within the dam safety.

6.1.1 Traditional techniques

The importance of a proper monitoring of dam behaviour and performance is vital in order to avoid incidents and accidents of such critical structures during its whole life-cycle.

The present thesis is focused in the state-of-art which are remote sensing techniques. Nevertheless, an overview of the traditional methods and scopes which must to be monitored and controlled are presented and discussed, due to financial, logistic and technical reason in most cases it is not possible to implement remote sensing methods for surveillance.

The main targets to monitoring are [7a]:

- Deformations

- Pore-water pressure and total stress
- Forces, strains and cracks
- Flow and turbidity
- Vibrations and accelerations

➢ **Deformations**

The measurement of deformation with as fibre optic or remote sensing such as GPS, InSAR or LiDAR are the most recent and modern techniques to measure these parameters, but in fact, they are not widely used in most of the existing dams. In contrast, the most common used are:

1- Crest levelling, which are used to measure the displacement on the crest dam using leveling and surveying techniques. When total stations are used, the accuracies can reach 1-2 mm or better with the advantage of the low cost.

2- Extensometers, usually installed in boreholes at different depths and in the foundation. They include vibrating wire displacement transducer with 50 mm range and ± 0.1 % F.S. accuracy to measure the deformations.

3- Sighting rods, used to determine straight lines, reference points and stablish the appropriate geometry in the dams. This technique is old-fashion, so it is used in conjunction with total stations or other survey equipment to figure the deformations out.

4- Precise survey of whole embankment, include the measure of horizontal displacement specially at the toe of the dam. The common technologies for this task are theodolites, total station, optical levels, photogrammetry and electronic distance measurement.

Common problems associated to deformation measurements:

- Movement caused by slope instability

- Movement caused by primary consolidation of the fill and foundation and by collapse compression on inundation
- Movement caused by internal erosion
- Volume changes caused by seasonal moisture content
- Deformation of fill caused by changes in reservoir level

It is very important to identify those processes, because in the case of embankment dams, internal erosion is the most common hazard that could lead to a failure.

- **Pore-water pressure and total stress**

The measurement and control of the pore-water pressure is critical specially in the case of embankment dams. The importance is based on the phreatic surface and to localize the seepage through the dam. In addition, the pore-water pressure can be used to analyse the effective stress with limit equilibrium to compute the factors of safety of the slopes when the parameter.

To analyse the pore-water pressure the following instrument are used:

1- Standpipe piezometers, are the most common piezometers in existing dams, due to the several advantages like easy to install and reading, cheap and stable performance during its life-cycle. Usually the standpipe piezometers are installed in the core and downstream fill. On the other hand, they are better to long term survey due to their low response of pore water pressure variations.

2- Hydraulic piezometers, are installed meanly in new boreholes and they can provide a long term reliable remote reading.

3- Pneumatic & vibrating wire piezometers, the pneumatic ones are less commons than other devices. They are used mainly in the upstream dam shoulders when the slope instability need to be investigated, they are relatively cheap and easy to install. They also provide a rapid time response. Limitation in partially saturated soils. In contrast, the vibrating wire piezometers are more expensive than the pneumatic.

Common problems associated to pore-water pressure measurements:

- Standpipe piezometer must to be read manually
- The lack of response in pore-water pressure changes, partially solved with automatic transducers
- Errors in the readings due to seal failure in the sand cells
- False readings due to failure of preventing water inflows on top of the standpipe where the piezometer tip is in low permeability soil
- Vandalism
- Clogging of piezometer could lead in false results in the long term

➢ **Forces, strains and cracks**

In cases where the core could have hydraulic fracture, there are several ways to investigate it. One of them, consist in surveying the total-earth pressure exceeds. Usually, the surveying methods are more common during the construction, instead for monitoring. However, the knowledge of the internal erosion can provide important information to analyse the dam.

1- Critical pressure test, are done with piezometers to obtain the local stresses. This test is carried out despite, there are others that are more reliable and it is also quite limited to clay soils.

2- Pressuremeter and dilatometer test, these tests are useful to measure the stresses in the cores. They do not provide a continuous measurement profile.

3- Push-in spade cells, are used to measure total horizontal stress and the vulnerability to hydraulic fracture in puddle-clay cores which have low strength. They are useful in long-term monitoring, nevertheless, they are quite complex during installation, maintenance, observation and interpretation.

Common problems associated to earth pressure in the core measurements:
A very high trained and specialized staff must to install and interpret the results from the earth-pressure cells.

- **Flow and turbidity**

It is very important to investigate the seepage and leakage through the different parts of the dam such as the foundations, the body or the abutments to avoid damages of instability.

1- Jug or bucket and stopwatch, are used when there are not previous measuring facilities and provides a guidance of the flow rates filling a jug or bucket. Patterns can be obtained.

2- Chamber with sum or settlement tank, with a flow measurement gauge and a sump to gather sediments to evaluate a potential erosion in the dam.

3- Divining rods, not common and reliable.

4- Tracers and/or chemical analysis, used to locate the source of leakages in a dam with chemicals, dyes and bacteriophages. A chemical analysis is used to determine the degradation of the material in dams.

5- Observation wells, located downstream of the structure and used to monitor changes in the water level which could indicate leakages. Traces are commonly used as well.

6- Underdrain flows, consists in collector trench drains beneath the dam which discharge at the downstream toe of the dam.

7- V-notch weirs, used to measure the flow where the flow is recorded manually or with remotely. Turbidity or suspended particles that could show internal erosion are collected in samples or traps.

8- Controlled source audio frequency domain magnetics, is a new method to locate leakage in embankment dams. It is expensive, but quick, non-intrusive and high accuracy.

9- Resistivity, playing with soil properties such as the mostire content, degree of saturation, permeability and salinity. It is limited to simple stratitfication, otherwise provides inaccuracies.

10- Temperature, based on the different seasonal temperatures within the ground, the leakage can be located with fibre-optic.

Common problems associated to seepage and leakage measurements:

These techniques must to me carried out also by specialist. Identification of the source through chemical analysis is not totally reliable and also the local regulation in the use of tracer must to be considered.

The temperature measurement method is only suitable in dams where access probes can be installed. This method is intrusive and the damage in the structure must to be considered. Other techniques like geophysical are very expensives.

- **Vibrations and accelerations**

To evaluate the structural integrity, some techniques are used such as crack meters and tell-tales, hammer tapping and inspection of gaps in combination with other techniques to obtain a comprehensive view.

1- Hammer tap and chains, the concrete is tapped and listened to locate changes in pitch which provide an idea of the concrete conditions. Chains are similar by dragging across the concrete.

2- Crack meters and tell-tales, are used to measure cracking and gaps in masonry or concrete over the time. The most common types are plate and pin with accuracies of 0.1mm.

3- Pendulum in valve tower, are very reliable and with a low maintenance. They could be either automated or manual monitored.

4- Penetrometer, used to determine the integrity of concrete or rock. Consisting in a calibrated spring dynamometer with a pressure indicating scale on the stem of the handle.

Common problems associated to structural integrity measurements:

It really relies in visual inspections to evaluate the structural integrity, because many of the techniques are for long term assess, which means that the cracks and displacements are identified once they have already occurred, so they are not a real indicator in advance.

It is important to mention that there is not a single instrument or combination of them that are able to cover all the hazard types. In addition, the measures are usually not comprehensive and not continuous in space and time. The precision and accuracy of the different devices used in monitoring must to be known in combination with the structural behaviour in order to avoid a mislead in the interpretation of the collected data.

The most common methods for monitoring according the results of the inquires to dam authorities and owners are [7a]:

- Regular surveillance
- Seepage-flow monitoring
- Visual, crest levelling
- Telemetry

- Piezometers
- Inclinometers
- Water line
- Rainfall monitoring

The table below shows the different monitoring techniques for each threat in hydraulic structures.

Threat	Monitoring and measuring techniques
Ageing	Historical data
Aircraft strike	Emergency response
Animal activity	Natural England
Changes in groundwater flow/ chemistry	Environment Agency
Earthquake	BRE Engineering Guide to Seismic Risk to Dams in the UK + www.bgs.ac.uk
Extreme rainfall/ snow/ flood	Weather forecast/ Environment Agency flood warning system
Failure of nearby infrastructure	Emergency response
Failure of reservoir in cascade upstream	Emergency response
Human activity	Undertaker/ Reservoir Manager
Layout, design and construction	Historical data
Mining/ mineral extraction	Arup Review of Mining Instability in Great Britain + www.coal.gov.uk
Operation	Undertaker/ Reservoir Manager
Snow/ ice/ frost	Weather forecast
Sunlight	Undertaker/ Reservoir Manager
Terrorism/ sabotage/ accident	Police
Wind	Weather forecast

Figure 6. 1: Threats and monitoring techniques (Almog et al, 2011)

6.1.2 Remote sensing. GNSS, InSAR & DInSAR

At the present days, new technologies have been developed especially in the field of displacement monitoring. They suppose and advance in the quality and accuracy of the data acquisition and therefore, in the in the decision making for further measures and short time response due to the life data collection and processing.

Firstly, the sensors should be selected according the time period that want to be monitored and their distribution in time (daily, weekly, years, etc.). It also should be set up between rigid and non-rigid deformations to get the patterns.

Horizontal movements [13b] are commonly triggered by external forces, mainly visible at dam crest and thermal and water level effect in the middle vertical cross section. On the other hand, the vertical movements can be seen in the upper crown, inspection tunnels, foundations and interface between the bedrock on the lateral valley flanks.

- **Global Navigation Satellite Systems (GNSSS)** is a precise geodetic technique very useful to obtain precise point displacements [14b] so it means that is a technique to monitor local displacements. The receivers are positioned in selected locations of the dam and/or slope which are probable to have deformations and are elaborated with respect to reference stations. Therefore, the points where the sensors are installed must be carefully selected in order to obtain good data.

The main contribution of this technique to deformation surveillance in hydraulic structures is the continuous measurement systems for automatic auscultation and early warning. Long term series can be added to the data processing to compensate the errors and to try to link the possible reasons of deformation to factor like water level change or thermal expansions [15b].

To achieve the highest accuracy possible, the monitoring network is very important, the reference or master stations should also be regularly controlled by external links to re check movements or errors.

Some limitation of GNSS in unfavorable environments like dams, that usually are placed in valleys, can be solved with the application of pseudolite-augmented GPS technique [16b].

- **InSAR,** which is synthetic aperture radar interferometry, is able to detect the movement very accurately. However, it also has some limitations like it is better for detecting vertical deformations rather than horizontal ones. Another limitation is detecting hard features such as concrete edges rather than soft features like vegetation.

Currently, there are some devices in the market (metasensing) for Ground Based GBSAR which do not need the installation of sensors and ca provide a scan time of 4 seconds, time between two scans of 10 seconds, spatial resolution up to 0.5 m at 1 km, accuracy till 0.1 mm with a range of 4 km.

With the software the GBSAR are able to generate also Digital Elevation Model (DEM) of the area. The software also allows to an automatic real-time processing and visualization of the SAR and a relative simple and fast data analysis (displacement, velocity, etc.) permitting 2D and 3D map visualizations of time series and an alarm criteria set-up.

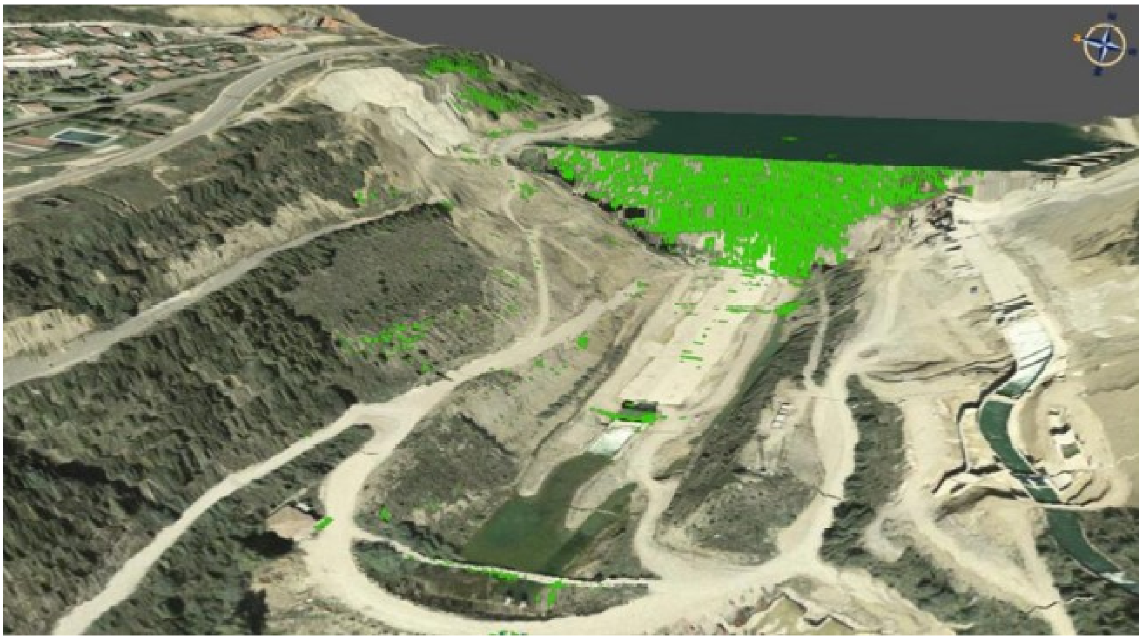

Figure 6. 2: Dam monitoring (M. Crosetto)

The GBSAR also have some limitations with can be lead to a misjudgment [13b]:

- Data coherence
- Ambiguous nature GBSAR deformation measurements because of phase ambiguity phase estimation
- Mono-dimensional nature of the observed deformations given a 3D displacement.
- Atmospheric effects, due to humidity variations in the atmosphere.
- Possible multiple reflections.

- Spaceborne **DInSAR** is still in phase of development due to continuous improvement of sensors and spatial resolution and data processing. It has already been applied in dam surveillance. However, the result obtained from space monitoring must to be validated by in-situ techniques due to the lack of reliability in this new technique, but the correlations between DInSAR and other results shown by GNSS or TLS (terrestrial laser scanner) indicate good matches and encourage further researches about it.

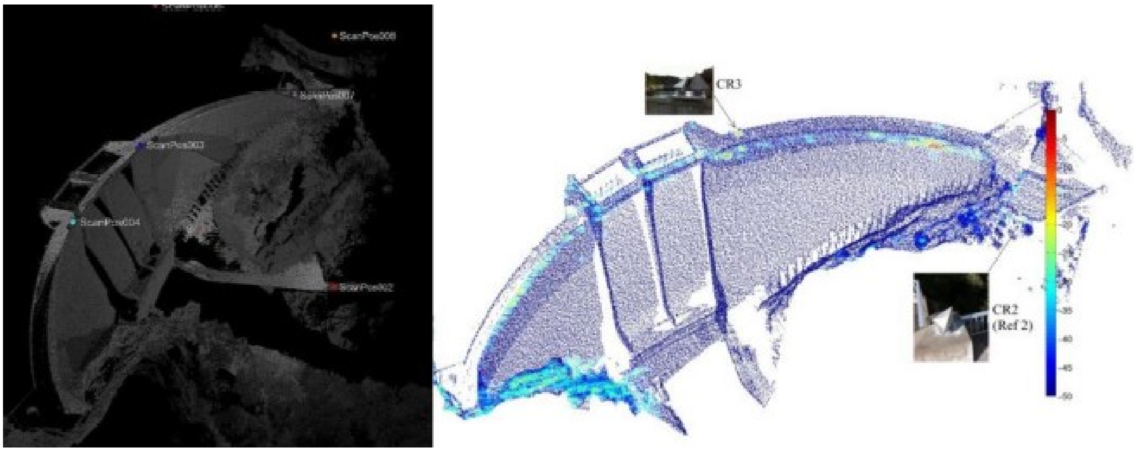

Figure 6. 3: Integration of TLS and DInSAR (Anghel 2016)

6.1.3 Hydrostatic – seasonal – time model (HST)

The Hydrostatic Seasonal Time (HST) model is the most popular statistical method of data monitoring for dams, based on database behaviour models. It is suitable for analysis and modelling of measured data for dam long-term behaviour.

The linear regression model allows to identify the reversible effects by the change of water level, displacements, etc. The model permits to make simultaneous estimation of hydrostatic load, temperature influences and reversible deformations.

The model HST [17b] which is written as $\hat{y} = a_o \cdot H_i + S_i + T_i$ is based on assumption of linear combination of three effects on the dam response which are:

1- The hydrostatic effect or the influence of water level into the reservoir can be expressed as a polynomial regression, such as:

$$H_i = a_1 \cdot h_i + a_2 \cdot h_i^2 + a_3 \cdot h_i^3 + a_4 \cdot h_i^4$$

Where h_i is a relative water level for time point t_i $h_i = \frac{h_{max} - h_{(ti)}}{h_{max} - h_{min}}$

Where h(ti) as water level for a time point tj [m], h_{max}, h_{min} as maximal and minimal water level [m] ant t_i with i = 1, 2…, N is a consecutive time stamp of observations.

2- Temperature effect, which is based on the seasonal variations in time series. The water and air temperature are assumed to seasonal variations which are represented by a linear combination of sinusoidal functions, using harmonic regressions.

$$S_i = b_1 \cdot \sin(\omega_a \cdot t_i) + b_2 \cdot \cos(\omega_a \cdot t_i) + b_3 \cdot \sin^2(\omega_a \cdot t_i) + b_4 \cdot \sin(\omega_a \cdot t_i) \cdot \cos(\omega_a \cdot t_i)$$

Where $\omega_a = \frac{2\pi}{\Delta_{ta}}$

And Δ_{ta} is 12 months for year monthly data or 365.25 days for year daily data.

3- Time effect, Bonelli et [17b] al proposed a positive exponential and a negative exponential based on monotone functions.

$$T_i = c_1 \cdot t_i + c_2 \cdot e^{ti} + c_3 \cdot e^{-ti}$$

Where t_i is reduced time during the analyzed period t_o, t_n. $t_i = \frac{t_i - t_o}{t_n - t_o}$

The HST model has several advantages and it is well known and widely used and easy to interpret since the external variables are assumed as cumulative. It provides a useful estimation of displacement in concrete dams and thermal effect is taken as a periodic function which means that only the variations in the reservoir level are necessary for the model.

The method also present limitations because of the assumptions and other features:

- The real behaviour of the structures is compromised because the functions have to be defined beforehand.

- The governing variables are taken as independent, but some are correlated between each other. There is a link between h and the air temperature as result of the fluctuation of water demand.

- Not appropriate to non-linear interaction models between input variables.

Salazar et al [18b] explained that the HST was implemented. The air temperature is linked with the thermal periodic effects which have been proven successfully in real cases.

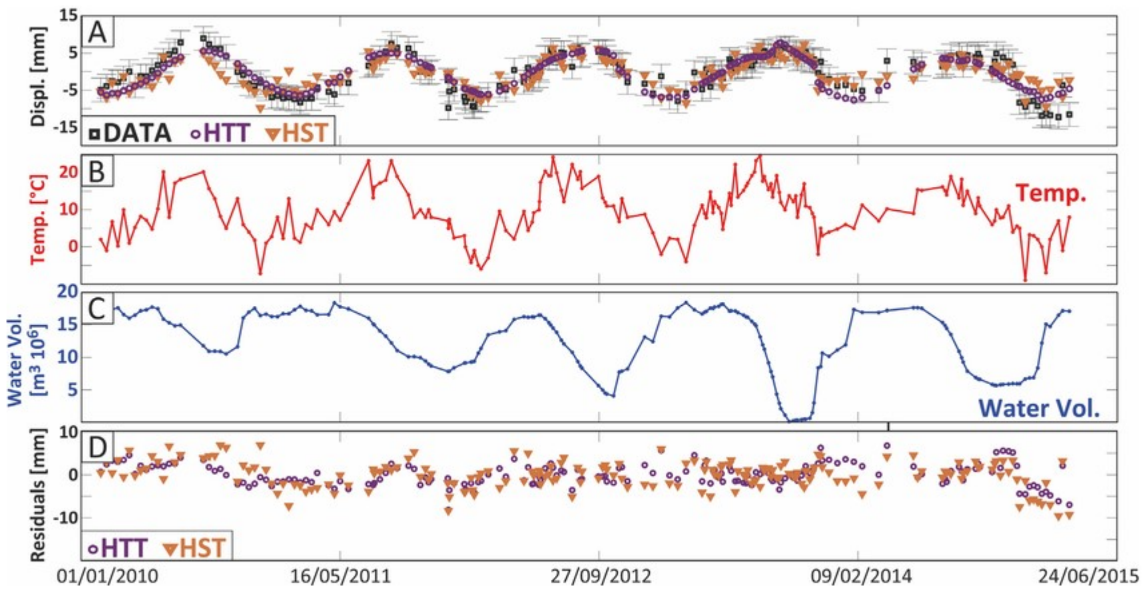

Figure 6. 4: Example of HST/HTT model prediction (Pietro Milillo)

The HST is a limited useful tool to displacement predictions and in order to monitor other effects as leakage or crack opening in real applications around the world, it also have shown that nowadays there are better tools available.

6.1.4 Machine learning models (ML)

Dam´s behaviors can be predicted with the use of data based models. The rotations, horizontal and vertical displacements, leakages, stresses, strains, seepage which are internal variables or external ones like temperature or water level can be monitored and evaluated with the aim of structural behaviour.

The most common and widely applied method is the statistical model. Nevertheless, the Machine Learning techniques provide an advantage for dam behaviour predictions, due to an easy operation, high accuracy and fast training phase compare with older methods.

The existing models for dam monitoring data analysis can be classified [18b]:

- <u>Deterministic</u>: based of finite element method (FEM), the dam response is calculated by physical governing laws

- <u>Statistical</u>: exclusively based on dam monitoring data

- <u>Hybrid:</u> deterministic models which parameters have been adjusted to fit the observed data

- <u>Mixed:</u> comprise by a deterministic model to predict dam response to hydrostatic pressure, and a statistical one o consider deformation due to thermal effects

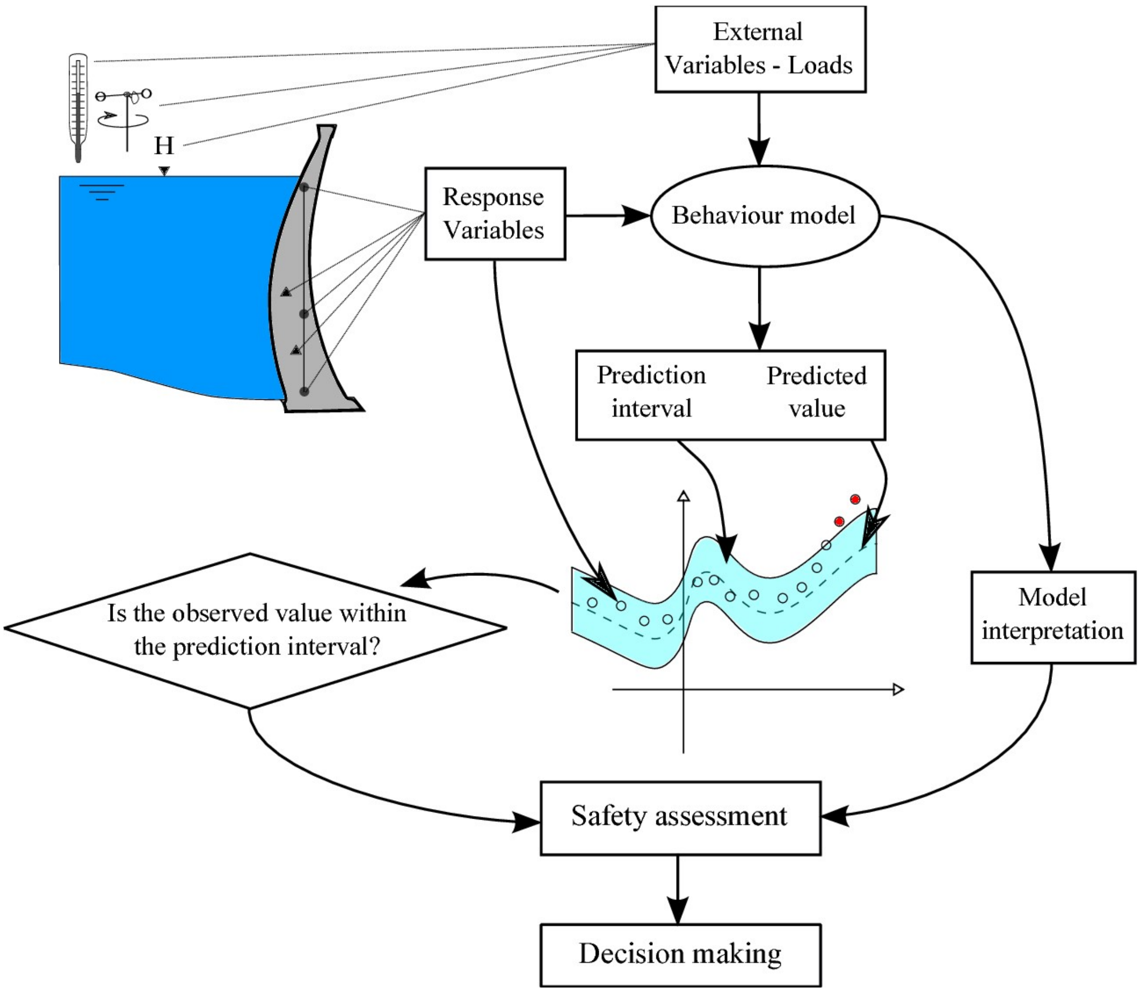

Figure 6. 5: Diagram of dam monitoring data analysis (Salazar 2017)

The HST model is well known by engineers working in the field of dam safety and it has the advantage that it is relatively easy to interpret, but HST is based on some not correct assumptions, therefore, they are not reliable at all. The ML techniques provide a tool, which is more complicated, but improves the quality of the analysis from the dam monitoring and it also gives an efficient early detection of anomalies due to the algorithms used.

As it was mentioned before, statistical tools are widely used in dam monitoring data analysis with favourable factor that are relatively simple. The analysis is based on the graphical exploration of the time series of data in conjunction with statistical models.

The HST uses multiple linear regression (MLR), taking in consideration external variables which are hydrostatic load, air temperature and time. An estimation of the concrete dam displacement for instance, can be provided without the air temperature time series data, because it is assumed to have a constant yearly cycle. Therefore, the resulting model is easily interpretable.

In contrast, the hydrostatic load and temperature are assumed independent. This assumption is not real, due to its relative simplicity, it carries out some other limitations like the lack of flexibility and not appropriate to non-linear interactions between input variables. It may lead to misinterpretation of results.

A great advantages of the ML algorithms is also that with its use in software [18b]:

- Accuracy: in the prediction of the dam response and model which means a narrower prediction interval that allow an earlier anomaly detection.
- Flexibility: high adaptability to the different characteristics and situations of dam typologies as load, strength, most probable failure modes.
- Interpretability: model analysis provides information between input and response.
- Ability to detect anomalies: criterion to assess and distinguish between input-output as normal or potential anomalous.
- Ability to identify extraordinary situations because of load combinations.
- Graphical practical application.

It must also be considered the delayed effects in each dam type, which are, for example, the pore water pressure in earth-fill dams due to the level variation in the reservoir and the influence of the air temperature in the thermal field in a concrete dam body or the structural response comprised elastic and viscous component due to hydrostatic load in arch dams.

To have some figure in mind to understand them, different researches have estimated the delay of those effects, varying from the five days of leakage flow [19b] and the 30 to 60 days [20b] in HST model.

Other authors like Bonelli et al [21b] and Lombardi [22b], have proposed other formulations to compute the thermal response with air temperature changes or with predicting radial displacements. Nevertheless, there are some limitations in the proposals, but the accuracy is being increased considering response variables mentioned before as inputs.

Santillán et al [23b], also proposed a method to select the optimal set of predictor among gradients of air temperature and reservoir level instead of moving averages to ensure the independence among predictors.

The disadvantage of the ML based on algorithms, is a lack of extrapolation, that means, for instance, the load combinations should fit in the input variables for a given situation.

As comparison between HST model and ML, deterministic and statistical models [18b] have proven to be useful tools but with limitations and poor performance at the same time. The development of algorithms in ML is providing accurate solution to practical problems. Going a little bit further, in the analysis of displacement and leakage, models based on random forest (RF), boosted regression trees (BRT) which are the most accurate, neutral networks (NN), support vector machines (SVM) and multivariate adaptive regression splines (MARS) are feasible to predict several variables.

With the growing interest of using the state-of-the-art for analysis. It seems that ML models with algorithms are a more reliable and precise method than HST ones to evaluate dam response. ML models deal with all the involved complexity in a smart way with an increasing volume and quality of data from monitoring. However, it must to keep an understandable level to allow to take final decisions.

6.2 Re-evaluation: calculation and design due to climate change

6.2.1 Spillways resizing

Nowadays there are less precipitations, but more intense. In many cases the spillways are currently obsolete to rainfall events. There are many studies which show that one of the effects product of the climate change regarding precipitations is that currently the rainfall events occur with less frequency, but they have a higher intensity.

Most of the dams that are in operation worldwide are quite old, with more 50 years under performance and when they were designed other parameters were considered in the calculation of the spillways and probably they are under designed for the flow rates that could occur with intense precipitations.

Therefore, it is significant that the spillways and the stilling pools should be re-evaluated being aware of the situation already mentioned in order to avoid cases like the famous Oroville Dam in the USA, where after a very severe drought a very heavy rainfall came and it was necessary to use the bottom outlet for the first time.

As mentioned, many dams are being reevaluated, modified and resized to protect against the failure. One of the topic of survey is the spillways that when they were designed some decades ago. The flood ranging used was 100 years and nowadays for stability reasons they must to be recalculated with the Probable Maximum Flood (PMF). That implies that the weirs and spillways should be updated with the new results.

Not only the flow rate is recalculated, the hydraulic and hydrodynamic loads acting on the spillways change in the new scenarios, consequently, the structural stability must to be evaluated.
The complexity of hydrodynamics flows regarding accelerations and decelerating flows can be carried out with physical models, but they expensive and time consuming. In addition, the stabilizing forces acting on the downstream side of

the spillway were simplified or neglected because they could not be accurately computed. Those forces, when the tailwater is increasing, they become significant and also provide a stabilizing forces, which have an economic impact in the spillway under re-evaluation.

A cheaper, faster and accurate alternative to the physical models, are the numerical models, that provide a good hydrodynamic evaluation (M. Savage et al) [24b]. They have proved in the comparison made between physical model data and Computational Fluid Dynamic to verified the statement.

The table below show the results of the comparison in three model tested.

Table 6. 1: Comparison of Observed Flow Rate Vs Computed Flow Rate (Savage et al)

	Model A			Model B			Model C		
Run	Physical (ft^3/s)	Numeric (ft^3/s)	% Diff	Physical (ft^3/s)	Numeric (ft^3/s)	% Diff	Physical (ft^3/s)	Numeric (ft^3/s)	% Diff
1	1.278	1.272	-0.5	1.218	1.211	0.6	0.805	0.791	1.8
2	3.143	3.244	3.2	2.938	2.949	-0.4	1.646	1.601	2.8
3	3.772	3.878	2.8	3.588	3.574	0.4	3.076	3.024	1.7
4	5.415	5.529 [c]	2.1	4.933[c]	4.955	-0.4	4.428	4.386	1.0
5	-	-	-	-	-	-	4.686	4.630	1.2

[c] Submerged flow due to high tailwater

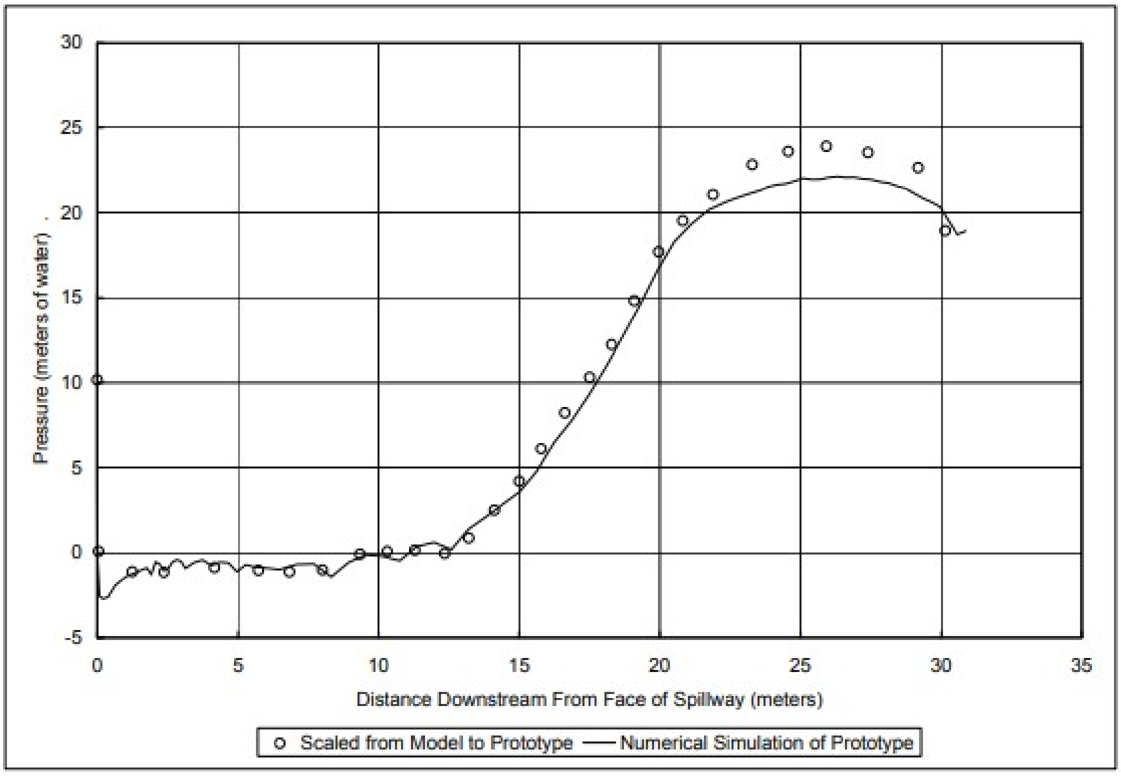

Figure 6. 6: Comparison of physical model data scaled to prototype dimensions and numerical simulation of prototype (Savage et al)

The results of the experiments show, that even with some limitation in the numerical simulations, they provide an accurate, cheap and faster tool to re-evaluate spillways.

In the same line of fulfil new security regulations, some innovating designs are being developing. Considering a feasible both economically and technically resizing of the discharge capacity of the spillways. One example are the labyrinth weirs with polyhedral bottom, which in combination physical and numerical tests are proving to be an interesting solution. One of the center developing researches about it is the CEDEX in Spain (Physical and numerical modeling of labyrinth weirs with polyhedral bottom) [24b].

The labyrinth weirs are an alternative in the discharge capacity, upgrading the traditional solutions, where the geometrical like the limitations of the width available and topographical boundary conditions can be solved with significant economic investments.

A traditional solution to increase the dam capacity, would be the implementation of spillway gates, but this solution entails a higher maintenance works in existing dams. Therefore, the development of the labyrinth weirs is an alternative solution for the geometrical limitation, providing a higher discharge capacity keeping a simple and economic maintenance operations.

The labyrinth weirs present in a plan view a modular al trapezoidal shape which lead to an increase on the discharge length.

The solution also presents some disadvantages. In one hand, in plan view, the labyrinth weirs need more space than a straight weir. The other limitation is that with flow rates higher than 50 m^3/s the hydrodynamic loads acting on the concrete walls increase, therefor,e it is necessary the reinforcement of them.
Formula for labyrinth weirs:

$$Q = L*C*H^{1.5}$$

Where:

Q: is the discharge flow rate [m^3/s]

L: is the length in [m]

H: hydraulic load

C: $2/3 \cdot 2 \cdot g \cdot 0.5 \cdot C_d$

C_d: coeficient of discharge, there are tables with different values according the geometry. For irregular geometries, it must to be modeled.

The laboratory of CEDEX has developed physical and numerical models trying to solve the limitations mentioned.

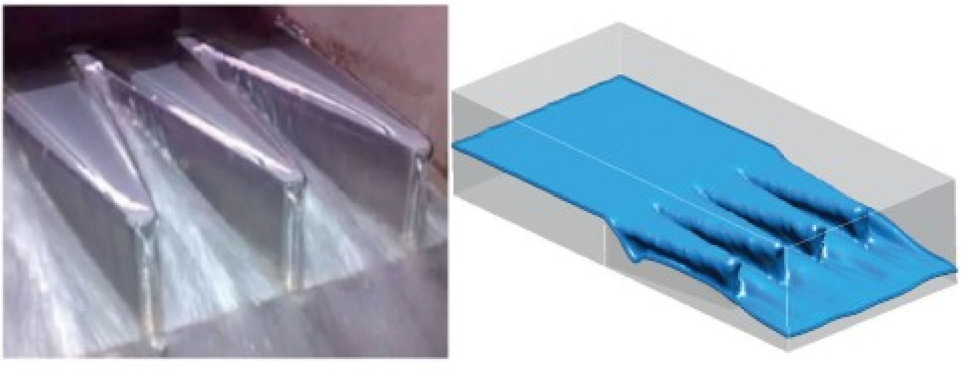

Figure 6. 7: Physical and numerical models of labyrinth weirs (CEDEX 2016)

Four different configuration were tested in order to find the optimal geometry and to analyse the stresses and hydrodynamic distributions.

- labyrinth weir with flat floor
- labyrinth weir with polyhedric floor and ramps upstream
- labyrinth weir with polyhedric floor and ramps downstream
- labyrinth weir with polyhedric floor and ramps upstream and downsteam

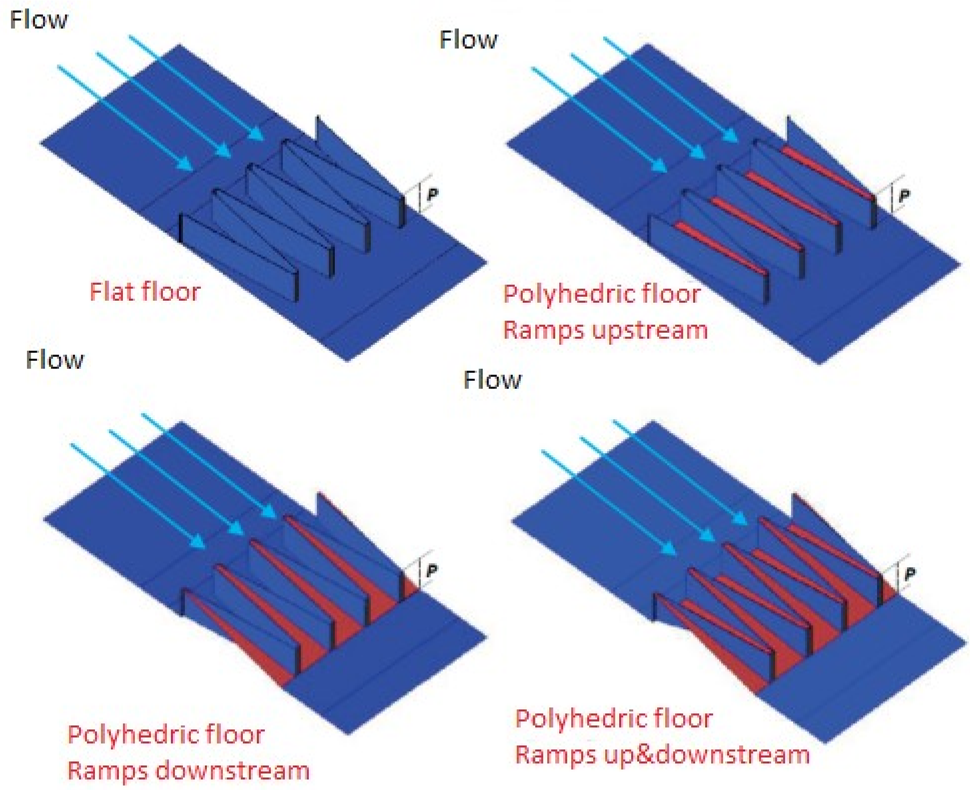

Figure 6. 8: Different configuration tested (CEDEX 2016)

The results of the research partially solved the disadvantages of the labyrinth weirs. The simulations showed that polyhedric floor has an impact in the velocity distributions and on the free surface in comparison with the flat floor. Nevertheless, there is not an improvement in the discharge capacity.

On the other hand, the hydrodynamic loads are decreased, due to the vertices are thicker and the free distance between the abutments. Therefore, it could mean less reinforcement is needed respect the traditional solutions and consequently cheaper.

6.2.2 Overtopping by landslides

Nowadays it is known that one of the consequences of the climate change is that the rainfall events occur with a higher intensity than before. Those heavy rainfall events could trigger landslides as result of the increase of the pore-water pressure, that supposes a hazard of overtopping to existing dams worldwide.

Perhaps the most famous known case of the consequences of dam overtopping is the dam in Vajont, where in October 1963 a massive landslide of approximately 260 millions of cubic meters of trees, soil and rock fell down in the reservoir producing a wave of around 25 m height above the crest dam killing almost 1500 people in the villages nearby.

There is an increasing concern to re-evaluate the surrounding of the dam reservoir. In order to identify possible hazards considering the phenomena's product of the climate change and overpopulation, that probably were not taken in account in former years when the dams were designed and constructed.

The hydrology product of the intense rainfall could be the key to increase the pore water pressure into the skeleton, this implies an increase of the buoyancy forces by reducing the shear strength. To describe the triggering mechanism of landslides, by changing of the hydromorphic responses, is not the aim of the present thesis, rather to raise awareness of the hazard.

There is a recent research carried out [26b], that provides a prediction of outflow hydrograph. It also can be used to evaluate the risks associated with overtopping of a dam due to landslides and also a landslide product of the dam failure.

The outflow hydrograph and its peak discharge should be assessed to reduce the losses and provide a feasible and effective safety measures. Xuan et al

[26b], proved that the landslide dam shape has a continuous decrease of height direction rather of a constant height with a notch. Furthermore, this survey describes the failure process with numerical 3D model and providing an approach of the outflow hydrograph.

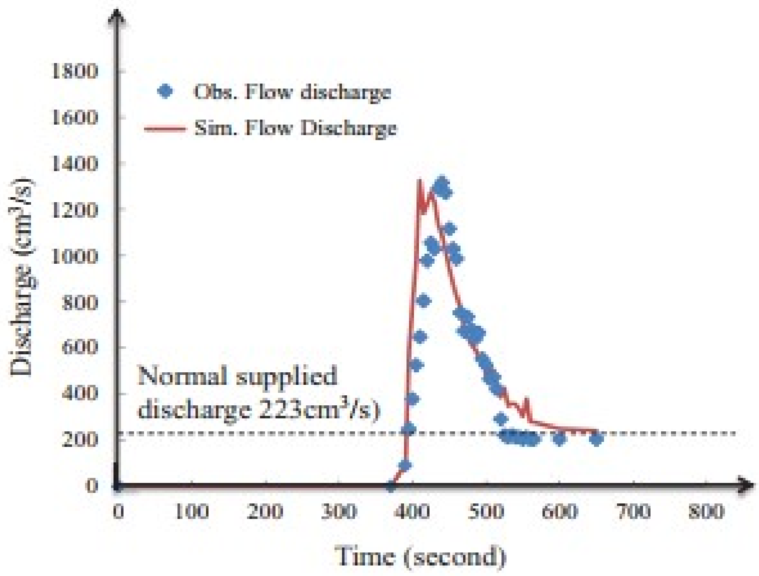

Figure 6. 9: Outflow hydrograph (Xuan Khanh Do et al)

The research also shows that the peak discharge product of a landslide dam failure is many times greater than the normal supplied discharge. So, the outflow hydrograph has proven that it is a useful tool to disaster preparedness.

7. Conclusions

The main results and ideas obtained after the analysis carried out in the different chapters are summed up and presented in general terms.

The conclusion of the failures occurred before the ICOLD Bulletin 99, 1995:

- The percentages large dam's failures decreased. There was a turning point at the 50's
- Between the large dams, those who has a lower height present a higher probability of failure. When the ratio is taken into account, the height is not associated with the failures
- There is a correlation between the age phase of the dam and its probability of failure
- Most of the dam failures correspond to embankment dams of any typology
- For embankment dams, the main mode of failure is associated to hydrological events like overtopping and piping
- For concrete dams, the main reason of failure is foundations problems associated with material deterioration
- Arch dams present proportionally a higher safety than other dam types

Regarding the failures after 1995, it can be observed:

- For all the dams considered (large or not) the year 1999 present the highest number of cases. For only large dams 1996 and 2010
- The highest percentage of failure correspond to dams between 15-30m in embankment dams
- Gravity dams have a slightly greater probably of failure when they are higher

- Most of the failure cases correspond to dams built before 1950

- The tendency of failures (figure 3.8) shows a curve. The probabilities are high in the infant period, then decrease and increase significantly when the dams are older than 10 years.

- The time dependent analysis indicates that for large dams, there is a periodicity of failures every 1.2 years and a change of cycle in the failure patterns after 7.6 years.

- Before 1995, approximately 2/3 of dam failures cases occurred within the first 10 years of dam lifespan. After 1995, for the same period the cases decreased to a 1/3 of the registered incidents.

In relation to the failures, knowledge of hazards is required to understand the dam modes of failures. It is necessary to analyze the combining of hazards, probability of occurrence and its consequences.

The first filling of dams must be carefully planned and supervised. Human factors have to be taken into account as trigger reason of incidents. Maintenance and proper monitoring are the keys to avoid risky situations.

The multi-level principle is the basis of the Austrian dam safety in conjunction with a long-life learning and formation of professional in charge of safety issues. All leads to a high security standard.

Monitoring techniques are constantly being developed. They are crucial to avoid dam failures. Modern and not that modern tools are used worldwide. The most important is the correct installation and interpretation of the data that the monitoring devices provide. Step by step, and as soon as the financial situations make affordable to implement the new methods, they should be done due to the advantages that the state of art present.

Last but not least, many dams should be reevaluated with updated methods in order to establish its level of safety.

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Appendix

Below the created database is presented, with a total number of 191 cases registered. According its height 28 cases correspond to large dams.

The main sources between many for the creation of the database are: internet searches (specific and general); [8a]; [7c].

Failures before 1995

ARCH DAMS					
Country	**Name**	**Height [m]**	**Const.**	**Failure**	**Failure Reasons**
RUSSIA	Sayano–Shushenskaya	242,0	1978	2009	Hydropowerplant explosion as consequence of waterhammer

EMBANKMENT DAMS					
Country	**Name**	**Height [m]**	**Const.**	**Failure**	**Failure Reasons**
USA	Timberlake	33,0	1920	1996	Overtopping
USA	Canyon MT	6,4	1891	1996	OT
USA	Aurora West	2,1		1996	OT
USA	Brookville II	6,1	1912	1996	OT
USA	Hamilton Hill			1996	QP, SEP
USA	Bergeron	11,0		1996	QP; SEP
AUSTRALIA	Dartmouth dam	15,0	1977	1996	Unlined rock steps damaged by flow concentration during low spill
USA	Bruceton Mills	6,1		1996	OT
USA	Mallard Lake	6,4		1996	DT; ABA
USA	Hightland Lake	3,7		1996	
USA	Henry			1996	QP; SEP
USA	Hollenbeck dam			1996	OT
USA	Nine Mile	25,7	1908	1996	QP; QIS
USA	Dillard dam			1996	OT
USA	Nagels Mill Pond	4,9		1996	OT
USA	Boeing Creek	6,1	1998	1996	OT; ISC
USA	Channahon II	7,6		1996	OT
USA	Mendham			1996	QP; SEP
USA	Cranberry			1996	QP; SEP
USA	Puddle Pond	5,2		1996	OT; BOP
USA	Decker dam			1996	QP; QIS
USA	Casa Monte	4,6	1993	1996	OT
USA	Meadow Pond Dam	11,0	1991	1996	Design and const. deficiencies resulted in failure in heavy icing conditions
USA	Apple Valley	6,1		1997	QP, SEP

USA	Henry Kaufman Pond			1997	OT
USA	Hamilton Mill	4,0		1997	OT
USA	Holland dam site A	4,0		1997	QP; SEP
USA	Green Acres			1997	OT
USA	Horn Rapids	1,5	1893	1997	OT
USA	Forsyth	6,1		1997	QP, SEP
USA	Johnson Ck #4	8,8	1959	1997	QP, QIS
USA	Moss Mill Lake	2,1		1997	OT
USA	Middletown dam II			1997	OT; BOS
USA	Patton			1997	OT
USA	Malcolm B. Rawls	6,4		1997	DT; BUA
USA	Lake Venita	9,2		1997	QP; SEP
USA	Anita dam			1997	QP, SEP
USA	Galbreath Sediment			1997	QP, SEP
USA	Port Republic II			1997	OT
USA	Centennial Narrows			1997	QP; SEP
NEW ZEALAND	Opuha Dam	50,0	during const	1997	Heavy rain during construction caused failure, dam was later completed
USA	Bay Meadows			1998	QP, SEP
USA	Little Ocmulgee	2,0		1998	OT
USA	Carl Smith	17,2	1998	1998	QP; SF-SL
USA	Hematite	4,0		1998	QP, SEP
USA	Peace Dale	4,9		1998	OT
USA	Golf Course	3,0	1948	1998	OT
USA	Bookhamer			1998	QP; QIS
USA	Pine Cove Pond	6,1		1998	QP
USA	Peru	1,8		1998	OT; ISC
USA	Boy Scout Camp			1998	OT
USA	Lake Runnemede	4,6		1998	QP; SEP
USA	Johnson Lake I	7,6		1998	OT; ISC
USA	Lacomb Diversion			1998	
USA	Gouldtown	4,6		1998	OT
USA	Jan Land C: Lake	5,8		1998	OT
USA	County Road 15			1998	OT
USA	Big Sandy	3,0		1998	OT
USA	Clay Brook	1,8	1973	1998	OT
USA	Camp Weona	3,4		1998	OT
USA	Archusa	9,1	1971	1998	QP, QIS
USA	Baker	5,5		1999	QP,SEP
USA	Lake Jimmy Carter	4,6		1999	OT
USA	Pittsfield	10,7	1999	1999	QP; SEP
USA	Kelly Pond	4,6		1999	OT
USA	Old Forge Pond	3,7		1999	OT
USA	Deer Creek	6,1		1999	OT
USA	Loden dam			1999	OT

USA	Longo Pond	3,7		1999	OT
USA	Lake Bray dam	4,6	1935	1999	OT
USA	Caloosa Sand			1999	OT
USA	Lookover Lake	3,7		1999	OT; ISC
USA	House Autry Mill			1999	OT
USA	Lake Hyenga	4,3		1999	OT
USA	Lake Lanahan	7,9	1930	1999	OT
USA	Jones Lake	4,0		1999	OT
USA	Fort Ritchie II			1999	QP, QIS
USA	CrownMinning Pond			1999	OT. BOS
USA	Nubble Pond	6,4		1999	QP
USA	Klickitat Mill Pond			1999	PM
USA	High Falls	9,1	1910	1999	OT; ISC
USA	Cold Brook			1999	
USA	Colee Naylor			1999	OT
USA	Cypress Shores	4,6		1999	OT
USA	Bostwick Pond II	4,0		1999	QP; QIS
USA	Kirbys Mill dam	4,3		1999	OT
USA	Covey dam	6,4		1999	OT
USA	Cow Creek	4,3	1935	1999	OT
USA	Beldon Pond			1999	QP, SEP
USA	Hall Lake dam	3,7		1999	OT
USA	J.B. Dunnell	8,8		1999	OT
USA	Foreman Branch	2,1		1999	OT, ISC
USA	Lake Powel	5,8	1964	1999	DT; ABA
USA	Bent Tree			1999	QP, SEP
USA	Essex Mill dam	4,3		1999	OT
USA	Dubose Lake	6,1	1969	1999	OT
USA	Hog Waste L:n Dike			1999	DT, WTA
USA	Frazers dam	5,8		1999	OT
USA	Nagels Mill Pond	4,9	1860	1999	QP; SEP
USA	Lower Robertson	5,5		1999	OT
USA	Bennett Lake II	9,1		1999	OT
USA	Allens Mill	4,3		1999	OT
USA	Camp La Junta	3,7		2000	OT
USA	City of Kake	7,0		2000	OS
USA	Grand Forks	8,8	1989	2000	QP; QIS
USA	Lott dam			2000	PM
USA	Middle Pond			2000	DT; ABA
USA	Fox Trail Lake			2000	OT
USA	Furnace Pond			2000	QP, SEP
USA	Lake Park	3,7		2000	DT; ABA
USA	Moss Creek Lake	20,4	1939	2000	QP; SEP
USA	Mountain Lake	4,0		2000	OT
USA	Murtha Pond	2,1		2000	OT
USA	Edison Pond			2000	QP, SEP

USA	Powell Lake	10,7	1939	2000	OT
USA	Ponca dam			2000	QP, SEP
USA	Chenowith dam			2000	DT; ABA
USA	Ascalmore	10,4	1959	2000	DT, ABA
USA	Eagle lake			2001	QP; SEP
USA	Mill Pond MA			2001	QP; SEP
USA	Marsh Lake	8,5	1942	2001	OT
USA	Pritchard Lake	11,0	1960	2001	QP; SF-SL
USA	Hill #1	7,6		2001	OT
USA	Francis Galloway Lake	4,3		2001	QP; SEP
INDIA	Pratappur	11,0	1891	2001	Brached on account of flood
INDIA	Jamunia	15,0	1921	2002	Piping leading to braching
USA	Clarke Apple			2002	QP; SEP
USA	Dixie Springs Refuge	5,5		2002	OT
USA	Big Sand II	7,6	1972	2002	DT, ABA
SYRIA	Zeyzoun Dam	32,0	1996	2002	Cracks appeared and collapse
NETHERLANDS	Ringdijk Groot-Mijdrecht			2003	Peat dam floated away
USA	East Burke			2003	
USA	Marquette No.3	10,1	1924	2003	DT; BUA
USA	Perry Knitting			2003	OT
USA	Lake Forest	8,2	1974	2003	QP; SF-SO
USA	Brindley			2003	DT; ABA
USA	Hope Mills Dam	10,0	1924	2003	Heavy rains caused earthen dam and bank to wash away
USA	Big Bay Dam	15,6	1992	2004	A small hole in the dam grew and eventually led to failure
USA	Bennet York	8,2		2004	OS
USA	Carter Pond	5,5	1951	2004	
USA	Blue Springs Power	9,4		2004	OT
USA	Johnson Lake II	7,6		2004	OT
USA	Greenview dam	7,3		2004	QP, SEP
USA	Meguire Lake	4,9		2004	OT
USA	Camp Ockinicken	2,1		2004	
USA	Birchwood	2,7	1955	2004	OT
USA	Lake Stockwell	3,7		2004	OT
USA	Lebanon Forest	3,0		2004	OT
USA	Enderlin Park	3,0		2004	OT
USA	Lake Dockery	7,3	1952	2004	DT; ABA
USA	Lower Aetna Lake	3,7		2004	OT
USA	Anderson Powell	7,6		2004	OT
INDIA	Gurilijoremip	12,0	1955	2004	The abutment structure problems with fooundation scouring
USA	New Orleans Dikes	13,0	1965	2005	Levee breach due to hurricane
USA	Lower Robertson	5,5		2005	OT
PAKISTAN	Shakidor Dam		2003	2005	Sudden and extreme flooding caused by abnormally severe rain

USA	East Bank I			2005	QP; SEP
USA	East Bank II			2005	QP; QIC
USA	IHNC East Bank			2005	QP; QIC
USA	Dennery	6,7	1952	2005	QP; SEP
USA	Allen Subdivision	4,9		2005	DT, ABA
USA	Cold dam	5,6		2005	QP, SEP
INDIA	Nandgavan	23,0	1998	2005	Overtopping and weir uncapacity due to heavy rain
BRAZIL	Campos Novos Dam	202,0	2006	2006	Tunnel collapse
USA	Boston Felt	3,7		2006	OT; IF
USA	Kaloko Dam	13,4	1890	2006	Heavy rain & flooding. Poor maintenance; inspection and illegal modifications.
USA	McClure	19,5	1919	2007	Rupture in penstock. Reservoir water level drown down
INDIA	Jaswant Sagar	43,0	1889	2007	Piping leading to breaching
INDIA	Palemvagu dam	13,0		2008	Flash flood resulting in overtopping of the earth dam
NEPAL	Koshi barrage		1960	2008	Heavy rain
INDIA	Chandiya	23,0	1926	2008	Breach
BRAZIL	Algodões Dam	21,6	2005	2009	Heavy rain. Filure spillway and then brake of the wall
INDONESIA	Situ Gintung Dam	10,0	1909	2009	Poor maintenance and heavy rain
KAZAKHSTAN	Kyzyl-Agash Dam			2010	Heavy rain and snowmelt
VENEZUELA	Manuelote	35,0	1975	2010	Overtopping leading to breach
USA	Delhi Dam	18,0	1929	2010	Heavy rain, flooding.
POLAND	Niedow Dam	12,0	1962	2010	Heavy rain, overtopped from flood-ing
INDIA	Gararda	32,0	2010	2010	Breach
JAPAN	Fujinuma Dam	18,5	1949	2011	Failed after 2011 Tōhoku earthqua-ke
BULGARIA	Ivanovo	19,0	1962	2012	Collapse of dam body
ZIMBABWE	Tokwe Mukorsi Dam	90,3	2008	2014	Downstream slope failure. Firs filling
USA	Oroville Dam	235,0	1968	2017	Heavy rain. Damage spillway and
LAOS	Attapeu-saddle dam	16,0	under construc-tion	2018	Failed due to heavy rain

GRAVITY DAMS					
Country	**Name**	**Height [m]**	**Const.**	**Failure**	**Failure Reasons**
USA	Folsom Dam	100,0	1956	1996	Spillway gate failure. Operation
CZECH REP.	Vodní nádrž Soběnov	8,6	1930	2002	Extreme rainfall during the 2002 European floods
BRAZIL	Camará Dam	50,0	2002	2004	Internal erotion in the abutment
USA	Taum Sauk	38,0	1963	2005	Computer/operator error; l. Minor leakages had also weakened byh piping.

USA	Hope Mills Dam	10,0	2008	2010	Sinkhole caused dam failure
USA	Tallulah Falls	42,7	1913	2010	Equipment failure. Uncontrolled realease through the spillway
TURKEY	Köprü	109,0	2012	2012	Gate in diversion tunnel broke after heavy rain during the reservoir's first filing.

Notes:

DT	disasters
OS	others
OT	overtopping
PM	poor management
QP	quality problems

ABA	animal or biological attacks
BOP	blockage of pipes
BOS	blockage of spillways
BUA	breaching of upstreams dams
EQ	earthquakes
IF	insufficient freeboards for settlement or poor design
ISC	insufficient spillway capacities
RLR	reservoir landslides
QIC	quality issues in culverts and other embedded structures
QIS	quality issues in spill-ways
SEP	seepage erosion or piping
SF	structural failures
SF-OV	structural failures of dam body-foundation unit by overturning
SF-SL	structural failures of dam body-foundation unit by sliding
SF-SO	structural failures of dam body-foundation unit by sloughing
WTA	wars or terrorist attacks