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SAFETY PERFORMANCE OF DAMS IN CHILE HIGHLY SEISMIC ENVIRONMENT




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ABSTRACT

Chile is known as one of the most seismic countries in the world, being responsible for more than 40% of the seismic energy liberated globally. The Valdivia earthquake (EQ) of 1960 with a magnitude Mw=9.5 is the greater EQ ever registered. Recently in February 2010 occurred the Maule EQ of Mw = 8.8 which is among the 6 world bigger earthquakes. The paper first presents a general description of the tectonic and seismic environment of Chile and a summary of the most important EQs (M≥ 7.5) occurred in the country in the last 200 years and more detailed characteristics of three strong EQs in the last 35 years in the Central Northern region of Chile: Valparaiso (1985), Maule (2010) and Illapel (2015).

The paper presents a summary of the performance of large dams in Chile (heights ≥ 15 m) from the end of XIX century to the present, including moderate high dams for irrigation and water supply built since 1850 up to 1930 and larger dams for the same purposes as well as for energy generation from 1930 to the present days. Being Chile an important mining country there are numerous tailings dams of different types which seismic performance is also discussed. The variety of dam's types for different purposes and different construction materials constitutes an interesting scenery to the analysis of dam behaviour under high seismic loading. Although the number of concrete dams in Chile is rather limited, comments on the seismic behaviour of two arch dam and a few rolled concrete dams of medium height are also presented, as well as the case of the only rockfill dam with asphalt core existing in Chile.

Special attention is given to compacted gravel dams with concrete faces and to sand tailings dams, both types of dam representing good examples of local development in dam engineering. In the case of sand tailings dams constructed with the downstream method, there is already a 200 m high dam that ended its operation and another one designed for a height of 240 m. The geotechnical characteristics and seismic behaviour of some specific dams of different types are presented as representative examples of Chilean dams for different purposes built on different periods. The paper analyses design criteria, local dam safety legislation and dam design and construction practices in the different industries. The paper finalizes with a discussion on the validity of different seismic stability and deformation analysis, both pseudo static and dynamic.

Keywords: Dams, Safety, Seismicity, Performance, Resilience.

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1 INTRODUCTION

Chile is characterized as a narrow strip of land between the high Andes Mountains in the East and the Pacific Coast in the West, with average width of about 200 km and around 4,000 km long. The Northern regions of the country are desertic with quite low precipitation while the Southern regions have relatively high precipitation. The Central regions show mild climate and moderate annual precipitations. Most of the mining activity is in the North and Central regions; agriculture and hydroelectricity are mainly located in the Central and Southern regions. The highly seismic environment and the special geography represent quite extreme conditions for dam safety and the Chilean engineering has had a major challenge with a long learning process in order to improve design, construction, operation and maintenance of dams, a challenge that is always present with relatively frequent strong earthquakes and dams with increasing heights as the ones built in the last three decades.

A register of the Chilean dams built until 1996 and the characteristics of the most representative dams are presented in a publication by ICOLD Chile [1]. This publication which includes different types of dams also presents a condensed history of Chilean dams [2].

Already in the Inca empire in the XVI century the agriculture required intensive irrigation in most parts of the country and many canals were built mainly after the Spanish conquer in the XVI century, but no register of dams are available for the period before the middle of XIX century. The rest of many old non-registered tailing deposits are still visible in the desertic Northern region of the country.

Since the middle of XIX century and up to approximate 1928 most of the dams were the result of private initiatives with little participation and supervision from authorities or government institutions. After 1928 started a significant role of government organizations in the design and construction of water retaining dams both for irrigation and water supply and for hydroelectric generation but tailings dams continued to be built under private initiatives.

Approximately in the 1950s and 1960s, because of the growth of world economy the country started the construction of larger water dams for irrigation and hydroelectricity as well as larger tailings dams. Also, in those years an important development in seismology and earthquake engineering was initiated, allowing for much better seismic design of dams. Fig. 1 show the development of water reservoir capacity during the last 100 years as reported by the Ministry of Public Works [3] and updated in 2020. The total water reservoir capacity in 2020, is on the order of 4,500 million of m^3.

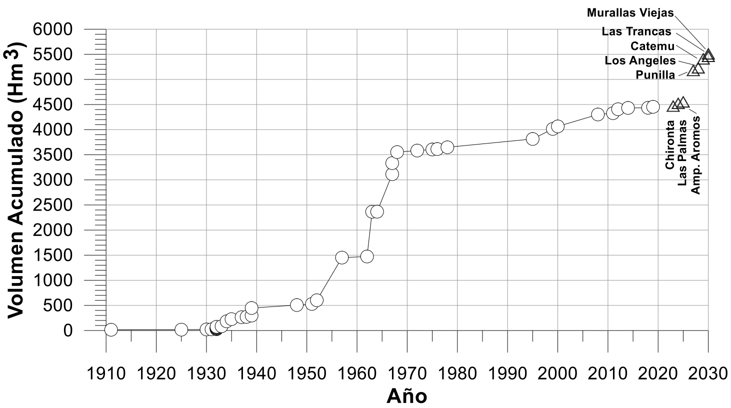


Fig. 1 - Reservoir capacity in millions of m^3. MOP [3]

During all these years the development of local guidelines and legislation dedicated to dam design have been relatively minor and most of the designs relied on international experience and international standards although in the last 50 years Chilean dam engineering has gradually taken a significant and distinctive international role in the seismic design of dams, supported by the experience obtained on important dams of different types that have been subjected to strong EQs.

2 TECTONIC AND SEISMIC CHILEAN ENVIRONMENT

2.1 Subduction Seismic Environment

The seismic activity of Chile is mainly the result of the subductive seismic environment generated by the collision between the Nazca and South American tectonic plates, which are converging at an estimated rate of 65 to 80 mm per year. The Nazca plate is subducting under the South American plate, moving down and landward. Accordingly, four types of seismic mechanisms in the Chilean subductive seismic environment can be identified: outer rise (outside trench, in the bending zone of the Nazca Plate), interplate or thrust-faulting type that occurred on the interface between the plates, intraplate that take place inside the Nazca Plate, and cortical (faults on the South American Plate).

From an engineering point of view, the most important earthquakes (EQs) are the interplates of large magnitudes. Then, the intraplates and corticals are important too, but their severe effect is restricted to a rather limited zone close to the epicenter. Typical acceleration records of both intraplate and interplate Chilean EQs are shown in Fig. 2 and Fig. 3, respectively. The records obtained in Pica are associated with an intraplate EQ (Tarapacá EQ, Mw=7.8, June 13, 2005) and the recorded accelerations at Talca are associated with an interplate EQ (Maule EQ, Mw=8.8, February 27, 2010). The main features of intraplate EQs are the high value of the peak vertical acceleration, the high frequencies (low period), and the rather short duration. On the other hand, the interplate earthquakes have high peak accelerations (in general, horizontal components greater than the vertical), a broad frequency band, and a long duration.

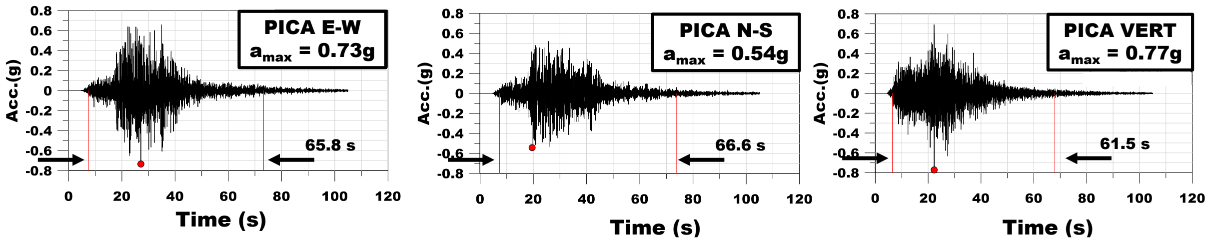


Fig. 2 - Typical acceleration records of intraplate EQ (Tarapacá, 2005)

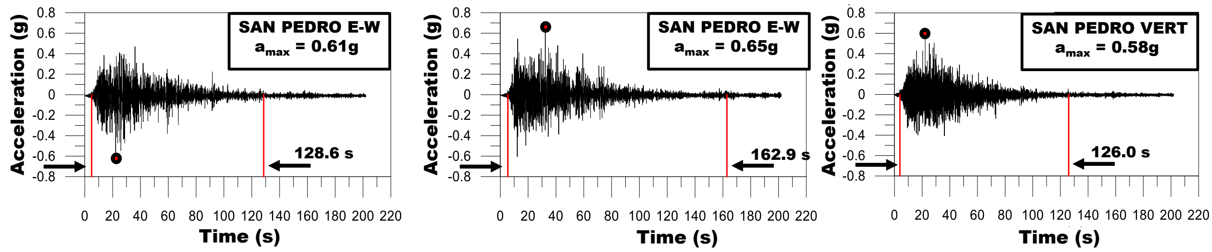


Fig. 3 - Typical acceleration records of interplate EQ (Maule, 2010)

Since 1850, when the first large dams (height over 15 m) began to be constructed in Chile, an important number of destructive EQs have struck the country and its infrastructure. A list of the EQs of magnitude M≥7.5 occurred in Chile since 1850 are summarized in Table 1 and their respective epicenters are plotted in Fig. 4. In the case of EQs occurred before 1979, the Richter Magnitude is informed, whereas for those occurred after, the Moment Magnitude Mw is reported. An exception is the Valdivia Mega-Earthquake occurred in 1960 which moment magnitud is reported (Mw=9.5). The magnitude of the old earthquakes that occurred at the beginning of the past century has been estimated basically based on the extension of the affected area and the level of the damages. It is observed that the Chilean territory has been continuously struck by strong seismic events, which imply that all the Chilean dams have also been subjected to strong seismic demands. It is interesting to observed that the high seismic activity of Chile is reflected by the presence of two large EQs in the list of the top 10 world EQs: the top 1 (Valdivia EQ) and the top 6 (Maule EQ).

Fig. 4 - EQs M ≥ 7.5 since 1850

Table 1 - EQs of M ≥ 7.5 since 1870

Date	Lat.	Long.	Depth (km)	M
16-09-2015	-31,572	-71,674	22.4	8.3
03-04-2014	-20,570	-70,493	22.4	7.7
01-04-2014	-19,609	-70,769	25	8.2
27-02-2010	-36.122	-72.898	22.9	8.8
14-11-2007	-22.247	-69.890	40	7.7
13-06-2005	-19.987	-69.197	115,	7.8
30-07-1995	-23.340	-70.294	45.6	8.0
05-03-1987	-24.388	-70.161	62.3	7.6
03-03-1985	-33.135	-71.871	33	8.O
10-05-1975	-38.183	-73.232	6	7.7
09-07-1971	-32.601	-71.076	60.3	7.8
28-12-1966	-25.494	-70.550	25	7.7
22-05-1960	-38.143	-73.407	25	9.5
22-05-1960	-38.061	-73.039	25	7.8
21-05-1960	-37.824	-73.353	25	8.1
06-05-1953	-37.093	-72.866	66	7.5
06-04-1943	-31.432	-71.475	35	8.1
25-01-1939	-36.305	-72.315	35	7.8
13-07-1936	-24.720	-70.110	35	7.5
01-12-1928	-35.155	-72.105	35	7.7
11-11-1922	-28.293	-69.852	70	8.5
04-12-1918	-26.538	-70.608	40	7.8
17-08-1906	-32.400	-71.400	35	8.2

2.2 Recent Large Earthquakes

2.2.1 Maule EQ 2010

On February 27, 2010, at 3:34 a.m. local time, a mega-EQ of Mw= 8.8 hit the Central-South region of Chile. A series of many aftershocks followed the initial EQ; the most important of Mw=6.2 occurred 20 minutes after the main shock. The Maule EQ corresponds to a thrust-faulting type, or interplate, occurred at an average depth of 30 km. The rupture zone responsible of this EQ covered a rectangular area of approximately 550 km by 170 km. The southern end of the rupture zone overlapped the rupture zone of the 1960 Valdivia EQ of Mw=9.5 and stopped near the southern end of the 1985 Valparaíso EQ of Mw=8.0 [4].

Considering the large extension of the rupture zone, the concept of hypocenter associated with a single point has to be replaced by the actual physic of the phenomenon consisting of an approximately planar area from where the seismic energy is emanated according to the evolution of the rupture itself. In the context of EQ of large magnitude, the hypocenter has to be identified as the place of initiation of the rupture, or the starting point of the EQ.

The horizontal peak ground accelerations (PGA) recorded on rock outcrop and soil deposits are presented in Fig. 5. It is interesting to observe that horizontal PGAs recorded on rock outcrops are surprisingly moderate; in fact, according to the available data the maximum recorded value is 0.32g in Santa Lucia Hill, Santiago. However, it is important to recognize that there was a lack of accelerometers in the most affected area toward the south of the rupture zone.

Another important characteristic of this mega-earthquake is its duration. The available records show two general patterns; to the south of the rupture zone, the duration of the ground motion tends to exceeds 2 minutes (i.e. San Pedro, Fig. 6a), but to the north of the rupture, the duration is systematically shorter (i.e. Melipilla, Fig. 6b). In any case, the long duration of the ground motions is a characteristic of earthquakes of large Magnitude, which certainly increases their destructiveness.

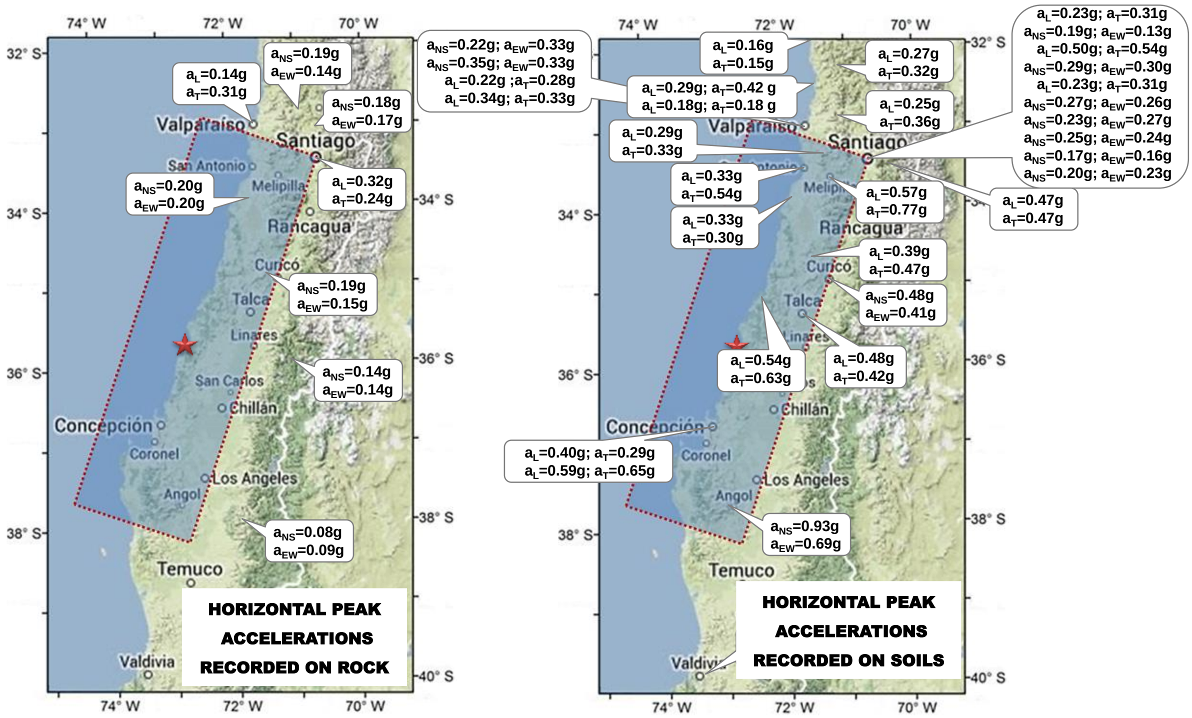


Fig. 5 - Recorded PGA on rock outcrop and soil deposits Maule EQ (2010) [5]

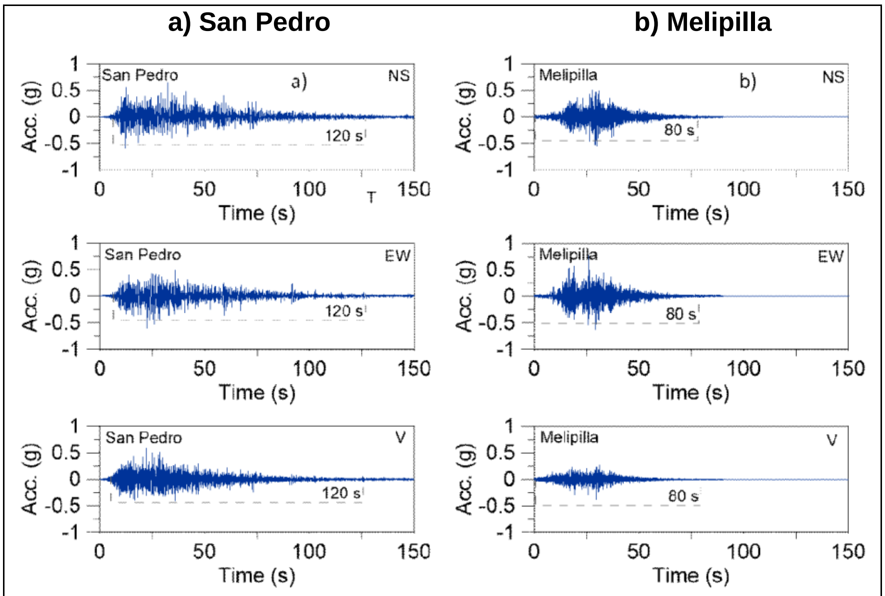


Fig. 6 - Acceleration time histories

2.2.2 Illapel EQ 2015

The Illapel EQ, Mw=8.3, that hit the Central-North region of Chile, occurred on September 16, 2015. It is the result of a movement along the interface between Nazca and South American tectonic plates, the rupture zone had a length of approximately 250 km and a width of about 100 km. The EQ generated a tsunami with a wave of a few meters affecting the nearest coast. PGA recorded are presented in Fig. 7, where a maximum value of 0.83g is observed in station C110. The horizontal acceleration histories of this station are presented in Fig. 8, where also the Arias Intensity (I_A) is plotted at the right axis of each graph. It is interesting to observe that peak acceleration takes place at the very beginning of the motion, and thereafter, several important high values occur with potential to generate damage.

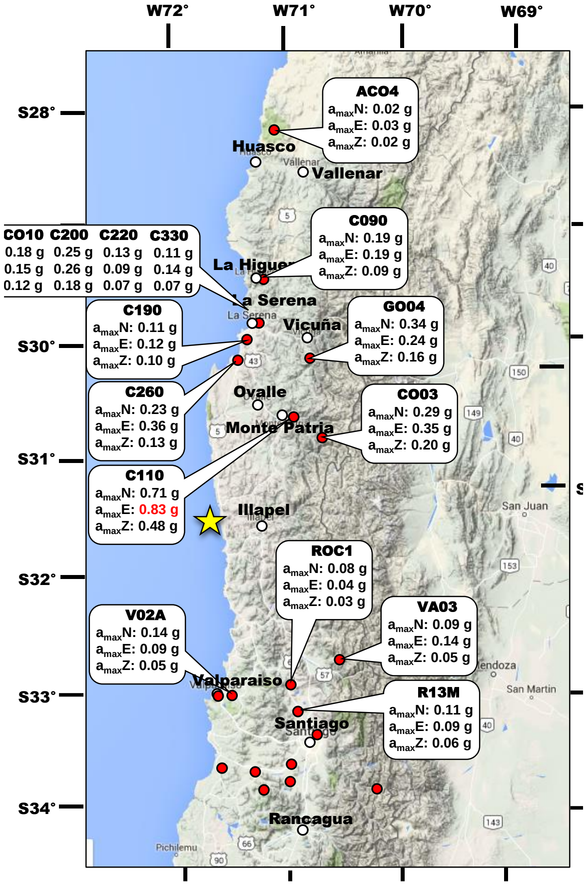
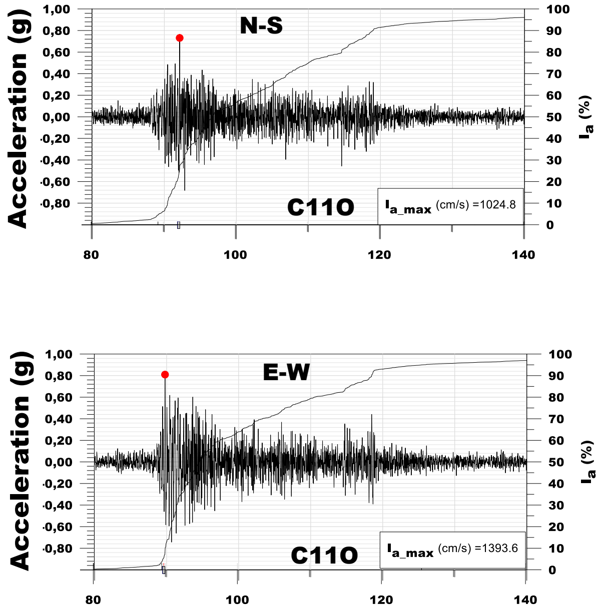


Fig. 7 - PGA Illapel EQ 2015



The **Arias Intensity** (**I_A**) is a measure of the strength of a ground motion. It was proposed by Chilean engineer Arturo Arias in 1970. It is defined as the time-integral of the square of the ground acceleration:

$$I_A = \frac{\pi}{2g} \int_0^{T_d} a(t)^2 \, dt \text{ (m/s)}$$

g is the acceleration due to gravity
T_d is the duration of signal above threshold

Fig. 8 - Acceleration time histories, horizontal component. C110 seismic station

2.2.3 Valparaíso EQ 1985

On March 3, 1985, in the central part of Chile occurred an EQ of Mw=8.0. In Fig. 9 are presented selected acceleration records from the 1985 and 2010 seismic events, where it is possible observe the differences in terms of amplitudes and durations, for the same stations.

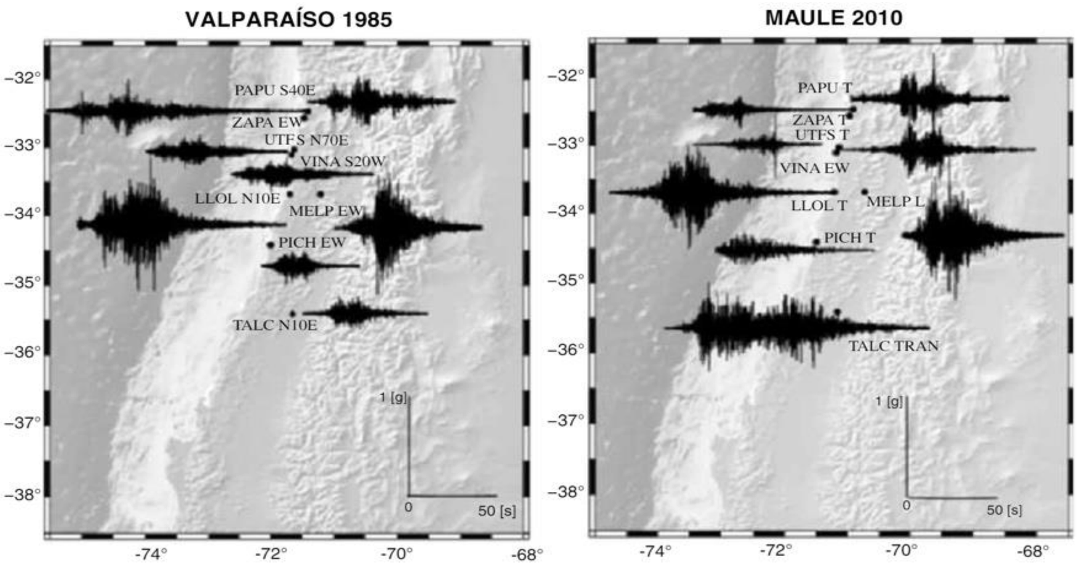


Fig. 9 - Acceleration records of Valparaíso 1985 and Maule 2010 EQs [4]

The seismic event of 1985 confirmed a quite systematic sequence of EQs that have occurred in this part of Chile. This sequence has been inferred from the large EQs occurred in this area in 1575, 1647, 1730, 1822, 1906 and 1985. This sequence can be characterized by an average period of 82 years ± 6 years [6]. The aftershocks of the 1985 EQ defined a rupture area of 170 km by 110 km. The main shock was the result of two shocks; the first motion began at 22:46:56 and then a larger event took place at 22:47:07 (Greenwich Mean Time). The maximum peak accelerations were recorded in the Llolleo seismic station, reaching the vertical component a value of 0.85g, whereas the maximum peak horizontal component reached a value of 0.67g (N10E).

This situation of a larger vertical acceleration has deserved important studies regarding its effect on the stability of civil structures. However, in other stations as Melipilla, the peak horizontal accelerations were larger than the vertical component: 0.67g in the north-south component and 0.59g in the vertical component [7].

Besides the enormous damage to buildings, industrial structures, ports, roads, and infrastructure in general, there was a significant impact to some small earth dams and tailings dams but only minor damages on large earth dams as discussed in chapters 3 to 6.

3 EARTH DAMS IN CHILE

Although there are references of some water dams built between 1838 and 1853, the first dam registered is one commissioned in 1853. The history associated with some older dams built before 1953 is confused since after the destruction of some dams (by a mudflow in 1877 and the 1928 EQ for example), quite frequently the new replacing dams were given the same name.

The comments on earth dams will be done separately for 3 periods: a first period from 1853 to 1928 with dams built mainly by private agriculture associations; a second period from 1930 to 1950 with dams built mainly by the Chilean State and some agriculture associations and finally a third period from 1950 up to the present days considering that after 1950 geotechnical and seismic engineering are already developed and in constant evolution and improvement. In the last decades dams starting to be higher and consequently becoming more relevant in terms of the importance of their safety.

3.1 Earth dams built between 1853 and 1928

Between 1853 and 1928 many earth dams were built and owned by privates and usually operated by irrigation associations. These facts have made it difficult for many of these dams to obtain details of their operation, seismic performance, and present conditions. Many of these dams are of modest height, less than 15 m. In this paper only dams with a height of about 15 m or more are considered, in accordance with the dam register published by ICOLD Chile in 1996 [1]. The dams built during the period 1853–1928 in the Central region of Chile have been subjected to the impact of several strong EQs with M ≥ 7.5 at relatively short distance of some of the epicentres, typically in the order of 100 km to 150 km, so even if there is no detailed report on EQ damage on these dams most probably many of them needed some repair works along their life period.

This register mentions 11 earth dams built in this period with heights ([1]) varying from 14 to 28m, including Catapilco, a 15 m high earth dam commissioned in 1853 being the first dam registered in Chile from which information is available. With the creation in 1887 of the Ministry of Industry and Public Works, nowadays Ministry of Public Works (MOP), the first dam commissioned by the Chilean State is the 22m high Peñuelas dam with construction completed in 1900 and in operation until now as part of the water supply system of the city of Valparaiso. It is not a typical earth dam but a one with a complex zoned cross section formed by bricks

[1] Heights in the ICOLD register are measured between the crest elevation and the lower point of the downstream slope. In most of the other references it is measured as the difference between crest elevation and foundation elevation in the vertical at the center of the crest. Heights not registered by ICOLD will be noted as m*.

and clay covered by rockfill shoulders in both slopes (1V:2,5H upstream and 1V:2,0H downstream). There are no reports informing on damages of some significance during EQs of these two dams [8].

It has been reported that Llui Lliu (20 m high) and Las Palmas de Quilpué (16 m high) dams suffered significative damage in the EQs of March 1965 and July 1971 [9]. The 1985 EQ of M=8.0 caused important damage in Lliu Lliu dam and provoked the failure of Las Palmas de Quilpue dam and La Marquesa (10 m high) dam. The failure of these two last dams was induced by liquefaction of foundation sandy soils [10]. It should be noticed that the Marquesa dam that failed in the 1985 EQ seems to be a different one from the one indicated in the ICOLD Chile, the Marquesa de Los Quillayes dam (21m high). There are also mentions to the case of La Viñilla dam (15 m high) with similar damage as Lliu LLiu dam, with longitudinal and transversal cracks The Lliu Lliu dam also suffered significant damage in the Maule EQ of 2010 as it is commented later. Besides Catapilco, Lliu Lliu and Peñuelas dams, the San Alfonso, La Dehesa, Portal de Alcones and Carrizal dams are still identifiable on relatively recent Google images (2017 to 2019) indicating that those dams are still existent although no information is available on the eventual damage they could have suffered on the numerous strong EQs occurred since their construction. Another earth dam built in that period El Sauce dam (15 m high) near Llolleo coastal town was not identified and probably also failed in some of the big EQs that have hit the area of the San Antonio port.

It should be noticed that before 1928 there were no construction equipment able to guarantee adequate soil compaction and that soil mechanics was not yet a developed engineering discipline. At the same time most of these dams have been operated by agriculture associations, without external supervision or inspection and not all of them have been provided with the adequate level of maintenance as well as the necessary repair works after strong EQs. Also, during drought periods that have been more frequent in the last decades, some of these private water irrigation reservoirs have been abandoned for long periods of time. Consequently, it should not surprise that only a part of them have been able to be operative almost over a century or more.

Catapilco dam is an earth dam with central impervious core, originally 13 m high and raised to 15 m in the decade of 70´s, crest length 1,800 m and 8,0 million m^3 original reservoir capacity [11]. The construction of this dam started in 1853 and completed in the period 1856-1859. This 171 year-old dam, the oldest one registered in the country, was still in operation by a local irrigation association up to some years ago when the operation of this reservoir was interrupted due extreme sedimentation of the reservoir. Removal of 250.000 m^3 of sediments allowed to recover part of the original reservoir capacity. Works started in October 2017 and completed in June 2018 and included some repair work in the upstream slope and placement of rip rap in the dam slopes. The strong EQs of March 1965 (M = 7.4) and July 1971 (M=7.8) could have produced some minor damage, probably the reason for the repair work done in 2017.

Fig. 10 - New spillway Catapilco dam [11]

Fig. 11 - U/S slope Catapilco dam [11]

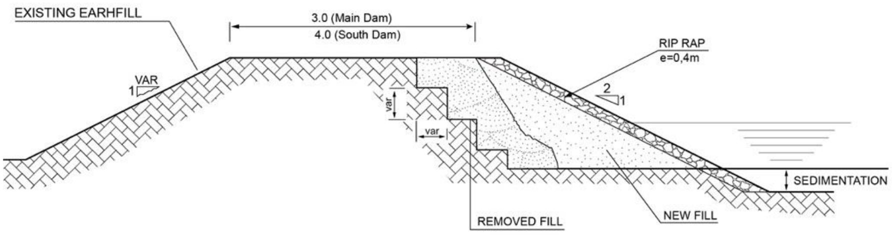


Fig. 12 - Catapilco dam, upstream slope repair. [11]

As already mentioned, Lliu Lliu earth dam was significantly damaged during the EQs of March 1965, July 1971 and March 1985 [9][12][13]. A visit to Lliu-Lliu dam after the 2010 Maule EQ (M=8,8) [14] identified two localized seismic slope failures in the u/s slope of the dam as shown in Fig. 13. The length along the crest of these failures were 36 m and 21 m. Additionally, a series of longitudinal cracks along the crest with a maximum lateral opening of 10 cm were observed.

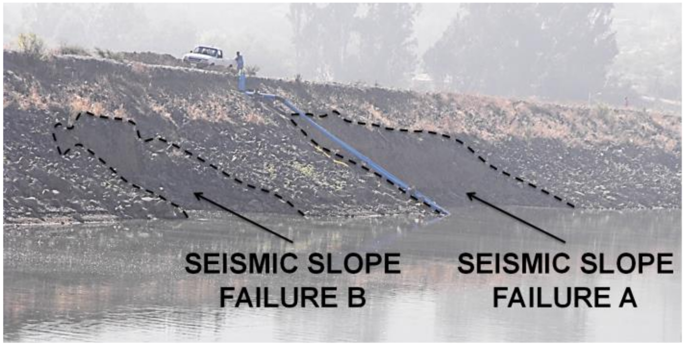


Fig. 13 - Local slopes failures u/s slope Lliu – Lliu dam after 2010 EQ [14]

Lliu-Lliu dam was apparently constructed between 1912 and 1920. It has a crest length of about 500 m and a maximum height of 20 m (18 m*). The upstream and downstream slopes had inclinations of about 1V:1.5 H, and a crest width of 8 m. The limited available information indicated that both shells consisted of gravelly material with an important fines content of clayey soils. The core would have been constructed with a mixture of fines soils and apparently also a high plasticity clay. All the aftershocks, including one of M= 6.9 (March 11, 2010), with an epicenter located approximately 160 km south of the dam, did not induce any additional movements of the upstream slope. Fig. 13 shows a low water level in the reservoir, a situation that has become common in several irrigation reservoirs because of the severe drought that has been present in the last decades in North and Central regions of Chile.

Of the 11 large earth dams registered by ICOLD as built in this period, 4 of them are still operating in relatively good conditions, 2 of them were destroyed by EQs, one has not been found and 4 are still existent although probably with some limitations and lack of proper maintenance. This result is not surprising since all these dams, except Peñuelas dam designed and built by MOP, were built by privates with lack of proper construction equipment at that time and probably also lack of technical supervision.

3.2 Earth dams built between 1928 and 1950

In this period is included earth dams designed mostly by the Irrigation Department of the MOP. The 15 earth dams registered by ICOLD Chile [1] as built in this period have heights varying between 15 and 47 m except for Bullileo dam (70 m high). Of the 15 dams registered, 14 of them are operated by irrigation associations and one (Sauzal dam) was built and operated by a hydroelectric company.

It has been reported that at least two of these earth dams have been seriously affected by the 1985 EQ of M=8,0 [13]. Cerrillos (19 m) and Perales de Tapihue (15 m) dams presented longitudinal and transversal cracks, with Perales de Tapihue dam also affected by localized upstream slope failure. It has also been mentioned [9][12][13] a Huechún dam that suffered longitudinal cracks during this EQ, but apparently it is referred to Huechún Bajo, a 10 m* high dam, which is different from the Huechún 15 m high dam of the ICOLD Chile register, that it has not been reported as affected by EQs, at least significantly.

Bullileo dam is still in operation but it has been repaired about 1983 after a serious piping erosion developed within the central impervious core, probably caused by cracks in its core due differential deformations near one of the abutments. No information was found of damages caused by EQ, although this dam is within one of the more seismic zones of the country.

The Illapel EQs of 1943 (M=8.3) and 2015 (M=8.4) impacted the Central – North region of Chile where three important earth dams are located: Recoleta (47 m), La Laguna (41 m) and Culimo (36 m). None of these dams suffer any significant damage as it has been reported by MOP [15]. In the Culimo dam, built between 1929 and 1933, a diaphragm wall through the central clay core and down to the rock foundation was implemented in 2007 to diminish excessive seepage through foundation.

Fig. 14 - Recoleta dam general view

Recoleta dam (47 m high) is an earth dam in the Central–North region of Chile (Coquimbo) was built in 1934 and it is a good example of the dams designed by MOP for irrigation purposes. At that time was the highest dam built in Chile and in its design can be recognized some features that were starting to be adopted in other countries, especially the USA. Rock foundation was another aspect that contributed to the good seismic performance over its 85 years of operation. This dam has been subjected to the Illapel EQs of 1943 (M=8,2) and of 2015 (M=8,4) and it was recently inspected after the 2015 EQ, identifying only minor slides of the downstream rip-rap and deformation at the crest [15]. The dam is about 125 km from the epicentre of those EQs. In 2017 a lateral wall, built by the operators over the lateral spillway, allowed the water level in the reservoir to go over the spillway crest, and the additional wall was overtopped, bringing the dam to the risk of a much serious major failure, fortunately not realized. The additional wall was later removed and restored the design conditions.

3.3 Earth dams built after 1950

After 1950 earth dams have been designed and built following in an increasing way the international practice and recommendations coming from different international organizations like US Bureau of Reclamation, ICOLD and others. The dam design practice was gradually improving based on the continuous development of soil mechanics, engineering geology, rock mechanics, hydrogeology, seismology, earthquake engineering and numerical analysis as well as the development of construction equipment and geotechnical investigation tools.

Most of the earth dams (24) after 1950 have been built either by irrigation private associations (6), by the MOP (10) or the State-owned electric company Endesa (established in 1943) and other companies (8).

The 6 earth dams designed and built by private irrigation associations have heights varying from 15 to 20 m which is a typical range for this type of dams. These dams are apparently in operation, but no information was available in relation to eventual damages during EQs.

The 18 earth dams built by MOP and hydroelectric companies have heights varying between 20 and 116 m, with only Tucapel dam with 15 m. The higher dam is Colbún dam (116 m) which is part of a hydroelectric complex which comprises other earth dams (Centinela 19 m; Machicura, 32 m and Secundaria 17 m). The oldest dams of this type and period are Tutuvén dams (32 and 17 m) with a positive seismic performance during the Maule EQ of 2010 (M=8.8) despite being the nearest dams to the EQ epicenter area. On the contrary a much modern dam Coihueco (29 m) built in 1971, during the same EQ suffered considerable damage as it is discussed later in this paper. Fig.15 shows the location of some of the dams already mentioned in relation to the fault trace and main or starting epicenter of the strong Maule EQ of 2010 (M=8.8).

El Yeso dam (61 m) built in 1967 is part of the water supply system for Santiago and its main problems have been associated with excessive foundation seepage and extensive cement grouting has been applied after its construction. Laguna del Maule dam (40 m high) built by the MOP in 1957 allowed the use of this natural lagoon as an irrigation/hydroelectricity project. It was inspected in 1970 [16] and recently for the occasion of the connection to the new Los Cóndores hydroelectric plant. None of these two dams have suffered damage during EQs.

Fig.15 - Earth dams impacted by Maule EQ 2010. [14]

The Tutuvén reservoir has two dams, the main one is an earth dam 32 m high and the secondary dam, the nearest to the epicentre, has a maximum height of 17m. These dams were built in 1951. According to the limited available information these dams were repaired in 1979 due to serious damage caused by a flood. The secondary dam is a homogenous earth dam with a much longer crest (1,000 m) that the main dam. This dam did not show any evidence of damage produced by the 2010 major EQ. [14].

The Coihueco dam experienced significant seismic deformations during the Maule EQ of 2010 (M=8.8). The dam, which was built in 1971, is located approximately 28 km east of Chillán. It is a zoned earthfill dam that has a crest length of 1,040 m and a maximum height of 29 m (31m*) as shown in Fig. 16. Following the EQ, a portion of the u/s slope experienced a failure that generated significant deformations and cracks as indicated in Fig. 17 and 18. Maximum vertical displacements of the u/s slope were observed towards the left abutment, reaching values close to 4 m. Uniform clean sand (SP) ejecta was present at the toe of the u/s earth slump.

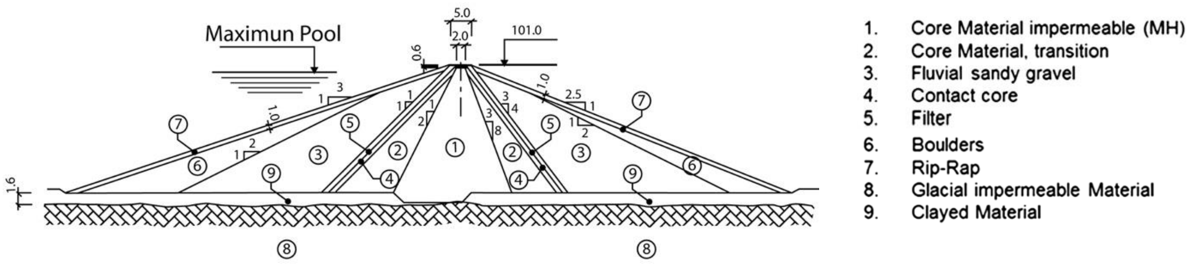


Fig. 16 - Coihueco dam design cross section, significant damage during Maule EQ 2010 [14]

According to the analyses done after the EQ [14], the foundation consisted of layers of sandy gravels with low to medium non-plastic fines content, clayey silts of medium to high plasticity (MH) and silts of low to medium plasticity. A pseudo static slope stability analysis applied to non-circular surfaces was performed with both horizontal (Kh) and vertical (Kv) seismic coefficients, and Kv ≤ 0.5Kh.

The results showed that failure (FoS≤1) occurs for seismic coefficients: Kh≥0.22 and Kv≥0.11. The authors concluded that “These values are consistent with the level of shaking experienced at this site and match well with values adopted in engineering practice for evaluating the impact of large EQs (i.e., MCE) on earthen structures”.

Fig. 17 - U/S slope Coihueco. Maule EQ 2010.

Fig. 18 – Crest Coihueco dam.

Aromos dam was completed in 1979, with a reservoir of 60.3 million of cubic meter in capacity. It is at approximately 90 km to the north-west of Santiago. The dam is placed in a narrow zone of Limache Creek, about 5 km d/s of the confluence with Aconcagua River.

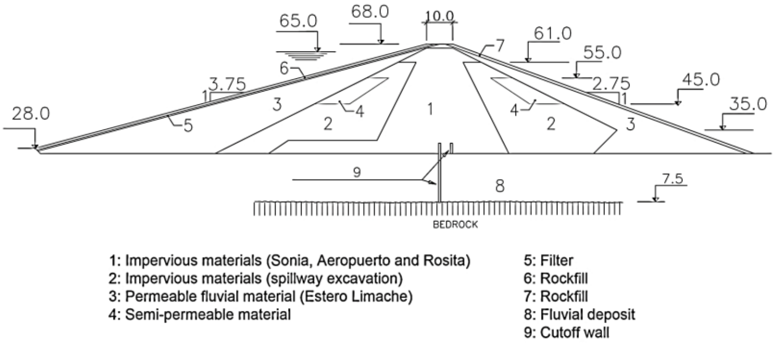


Fig. 19 - Aromos dam design cross section

It is a zoned dam, with a maximum height of 42 m and a length of 220 m. The u/s and d/s slopes are 3.75:1 (H:V) and 2.75:1 (H:V), respectively. The embankment consists of a core of fine soils with supporting shoulders made of gravelly sand. It is resting on fluvial soil deposits constituted by sandy soil materials. Both abutments are constituted by weathered granite that improves with depth. A plastic concrete wall 80 cm thick was built through the fluvial foundation down to a maximum depth of 22.5 m, but without reaching bedrock.

It has been described that near the end of construction some equipment got sunk in loose sand that suffered localized liquefaction due vibrations produced by the equipment. This fact implied the development of a comprehensive geotechnical investigation that concluded that the foundation ground was liquefiable, so the dam requested a significant re-design. Due to economic reasons, no changes in the dam were performed, and only wells were constructed near the toe of the dam with the intention of providing a dissipation system to any excess of pore pressure in case of dynamic loadings [17].

The dam did not suffer any serious damage during the 1985 EQ, although the acceleration level around the dam is estimated in the range of 0.3g to 0.4g. The data showed that the 1985 EQ induced in the embankment vertical settlements smaller than 10 cm, which implied that the foundation ground did not suffer any major liquefaction. Therefore, new studies were carried out, which concluded that the areas of low N-SPT (liquefiable sandy soils) are surrounded by areas with high N-SPT (no-liquefiable soils). It was interpreted that the areas of high N-SPT, that are stiffer, probably took most of the dynamic shear stresses, reducing in this way the disturbances of the loose confined zones [17].

Convento Viejo dam is located 160 km south of Santiago near the town of Chimbarongo. This embankment dam, 32 m high, it is separated by a small hill of a rolled concrete dam where spillway and gates are located. The two dams allowed the formation of a reservoir with a storage capacity of 237 Mm^3. The dam, has a central impervious clay core, u/s and d/s compacted gravel shoulders with u/s and d/s intermediate filters of granular materials founded on fluvial deposits of gravels and sands with a total maximum thickness of 55 m beneath the central part of the dam. The typical section of the dam is presented in Fig. 25.

The construction of the project started in 1973, but work was halted in 1975 due to financial problems. The construction was resumed in 1978-1979, but once again was detained. The cofferdam and the slurry wall allowed the formation of a smaller reservoir that was in operation until the full dam completion in 2008. The construction of the earth dam under a new design but incorporating the works realized until 1979 and an adjacent concrete dam for the spillway were initiated in 2006 and finalized in 2008 [18].

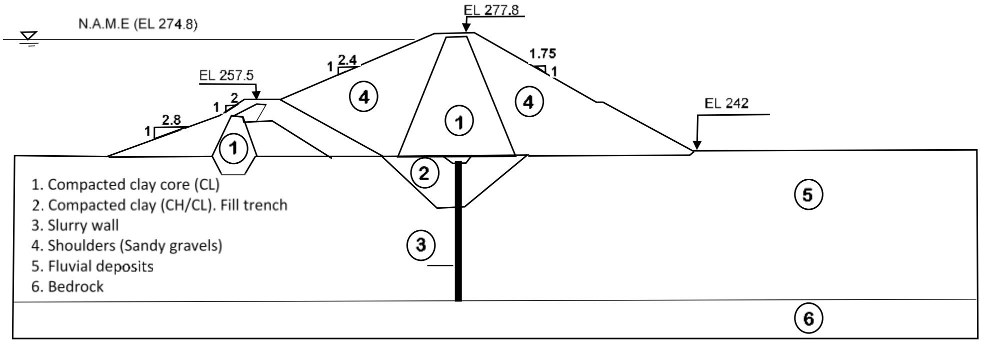


Fig. 20 - Cross-Section Design of the Convento Viejo Dam [18].

The Maule EQ 2010 (M =8.8) was somewhat greater than the MCE considering in the design (MCE of M=8.5). The results of the registers for the Maule EQ of 3 triaxial accelerographs installed in the dam and in an adjacent tunnel are indicated in Table 3. Superficial cracks were detected at the dam crest. It was not possible to identify, during repeated inspections of the earth dam, any seepage flow, or filtrations in the downstream slope of the dam. One of the piezometers located at the crest of the dam presented a sudden rise in water pressure interpreted as a possible crack in the clay core, but no increase in seepage or infiltrations confirmed this hypothesis. The maximum vertical deformations recorded at the crest, after the main earthquake, were lower than 35 cm [18] A study through a numerical analysis carried after the EQ suggested that the February 27 EQ modified the fundamental period of the wall, analysis that should be confirmed with the registers of new strong EQ [19].

Table 3 - Recordings February 27, 2020 earthquake and main aftershocks. [18]

Date	Mw	Dist. (km)	D (km)	Loc.	PGA E-W	PGA N-S	Vert. PGA
27 Feb.	8.8	250	30.1	A1	0.49	0.50	0.44
				A2	0.30	0.38	0.27
				A3	0.19	0.15	0.20
11 March	6.9	105	33.1	A1	0.14	0.18	0.13
				A2	0.12	0.15	0.07
				A3	0.06	0.08	0.06

Mw= Magnitude according to U. de Chile
Dist.= Distance from epicenter to the dam
D= Depth of earthquake
PGA E-W = Peak Ground Acceleration of the E-W component
PGA N-S= Peak Ground Acceleration of the N-S component
Vert. PGA = Vertical acceleration in g's

The Melado dam is a zoned earth dam, 90 m high built in 1991 as part of a hydroelectric scheme. It is 310 m long with a central core and gravel shoulders. The shoulders were compacted at 85% relative density and the core to 95% of Standard Proctor. The dam is founded in a coarse gravel fluvial deposit 50 to 75 m thick and it has a cut off wall through the fluvial deposit. The canyon walls are steep, and the embankment is contiguous to the reinforced concrete spillway on the right abutment [20] see Fig. 21. A dynamic seismic analysis of the dam was done in 1992 [21].

Because of the Maule EQ of 2010 (M=8.8) the dam, which is 200 km from epicentre, received a strong impact. The peak ground acceleration at the dam was estimated to be 0,15 g using an attenuation relationship [22]. Actual accelerograph measurements in the crest of the dam indicated maximum accelerations of 0,47g (transversal), 0,33g (longitudinal) and 0,45g (vertical) with estimated amplification with respect the base of the order of 3,6 to 3,8. [23].

Visual inspection indicated a longitudinal crack along the crest, 10 to 20 mm width and 100 cm long. Also, a transversal crack near the left abutment and another one in the contact with the spillway. None of the cracks were too deep. No variation on Casagrande piezometers were detected as well as no changes in infiltration flows.

Surface monitors indicated a settlement of 7,1 cm at the centre of the dam diminishing to a 20% of that value near the abutments.

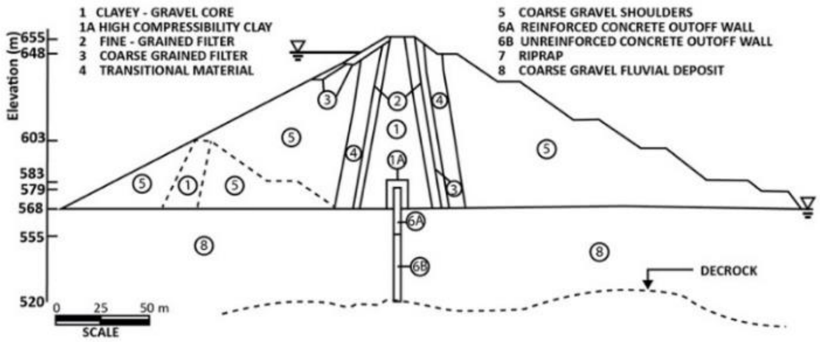


Fig. 21 –Typical cross section Melado dam [20]

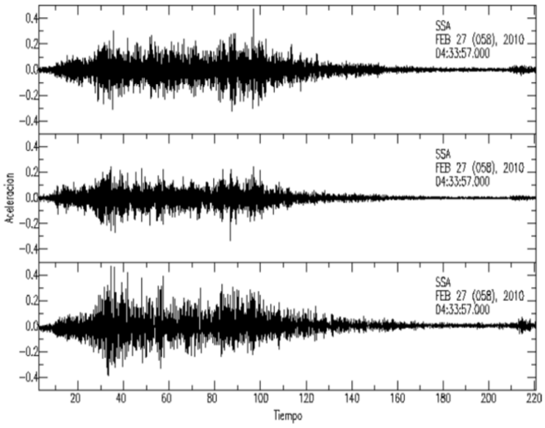


Fig. 22 – Registers of accelerographs at the crest of Melado dam. February 27, 2010 [23].

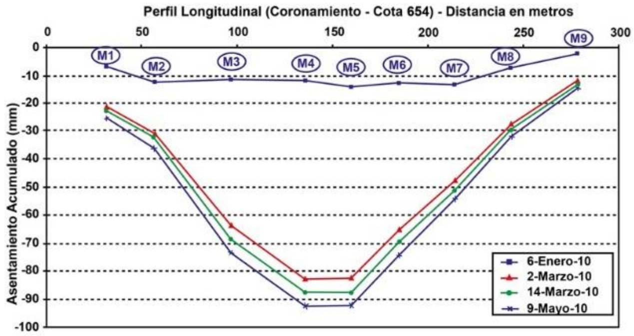


Fig. 23 – Melado dam crest settlements, Maule EQ 2010 [23]

La Paloma Dam is located approximately 25 km to the South-East of Ovalle City, with a reservoir of 740 million m^3 capacity. This dam was constructed between 1959 and 1967, it has a maximum height of 82 m and a length of 1000 m (including a spillway of 100 m of width in the left abutment). Paloma dam is a zoned dam with a clay core and shoulders constituted by borrow materials as observed in the cross-section of Fig. 24. It is founded in an old fluvial gravelly material of variable thickness, however, part of the right abutment is directly supported by the bedrock (see Fig. 24). The irregular foundation ground has been the cause of non-homogeneous settlements; however, Paloma Dam has shown a good seismic behavior, with minor damages after the Punitaqui and Illapel EQs.

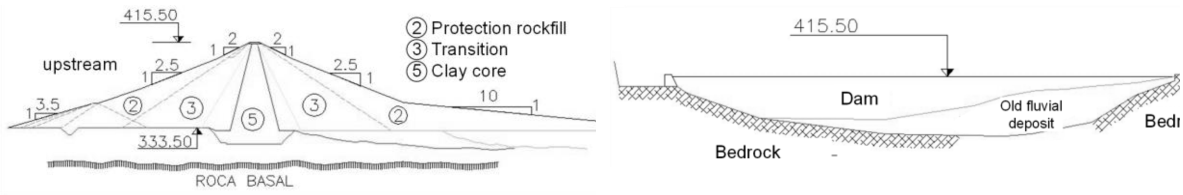


Fig. 24 - Cross section and longitudinal section of Paloma Dam

Colbún dam is a zoned earth dam 116 m high and 550 m long. It has an inclined sandy clay core and compacted gravel shells. It is founded in a fluvial deposit 68 m thick. A concrete cut off wall was built through the fluvial deposit to control seepage. During the Maule EQ of 2010, an accelerograph in a rock tunnel next to the dam registered a peak horizontal acceleration of 0,37 g. The registered peak vertical acceleration was 50% over the horizontal acceleration.

It has been informed [20][24] that this dam, the highest earth dam for water retaining purposes in Chile, suffered only minor damage during the 2010 EQ and relatively low settlements and displacements, of no more than 10 cm. Transverse cracks across the crest next to both abutments had a thickness of 1 cm, and when excavated disappeared at a depth of 3 m.

An electrical conduit next to downstream edge of the crest was displaced a maximum of 2 m horizontally and almost 1 m vertically, but inclinometers along the downstream shoulder did not show deformations after the EQ.

Fig. 25 – Colbún dam general view (https://mapio.net/pic/p-9289343/)

4 ROCKFILL AND GRAVEL DAMS

Rock fill dams as water retaining structures started relatively late with the construction of two dams designed by MOP and operated by irrigation associations and two dams built by privates, all of them built between 1935 and 1946. Only in the 1970´s new rockfill dams were built and a new revival appeared later with natural gravel dams with concrete faces following the general concepts of rock fill dams. Location of some of the dams that will be mentioned later is indicated in Fig. 26 together with the accelerations registered in various stations during the Illapel EQ of 2015. Of the 5 dams indicated in the figure only La Paloma (a zoned earth dam (96 m high) suffered some damage in its right abutment [15]. The other 4 dams of the CFRD type did not suffer damage during this EQ.

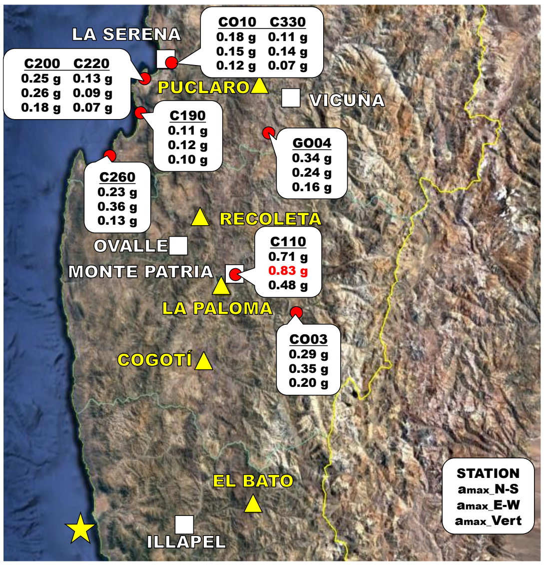


Fig. 26 - Dams impacted by Illapel EQ 2015

4.1 Rockfill dams with central impervious core

The construction of the first rockfill dams followed the conventional design with a central impervious core, rockfill shoulders and transition material between core and shoulder. These dams were Caritaya (32 m) built in 1935 and two dams built by privates La Marquesa (23 m) and El Sauce (21 m) built in 1940 and 1946, respectively. From the last two there is no information on their performance but a recent review done in 2011 of the dams built by MOP [3] showed the good conditions of the Caritaya dam located in the extreme region of North Chile.

An evolution of the original concept of the rockfill dams with central clay core is represented by Los Cristales dam (31 m high) built in 1977 by MOP including an asphalt core. The cross section of the dam and details of the top and bottom of the asphalt core are shown in Fig. 27. The dam was visited after the Maule EQ of 2010 in view of its relatively short distance to the fault trace and epicenter of the EQ. The asphaltic vertical core has a constant thickness of 60 cm. The post seismic survey carried out on the dam indicated settlements of the shells of only a few millimeters. Even though the seismic response of this dam was satisfactory the core settled about 14 cm (Fig. 27) next to the left abutment of the dam shown in Fig. 26. [14].

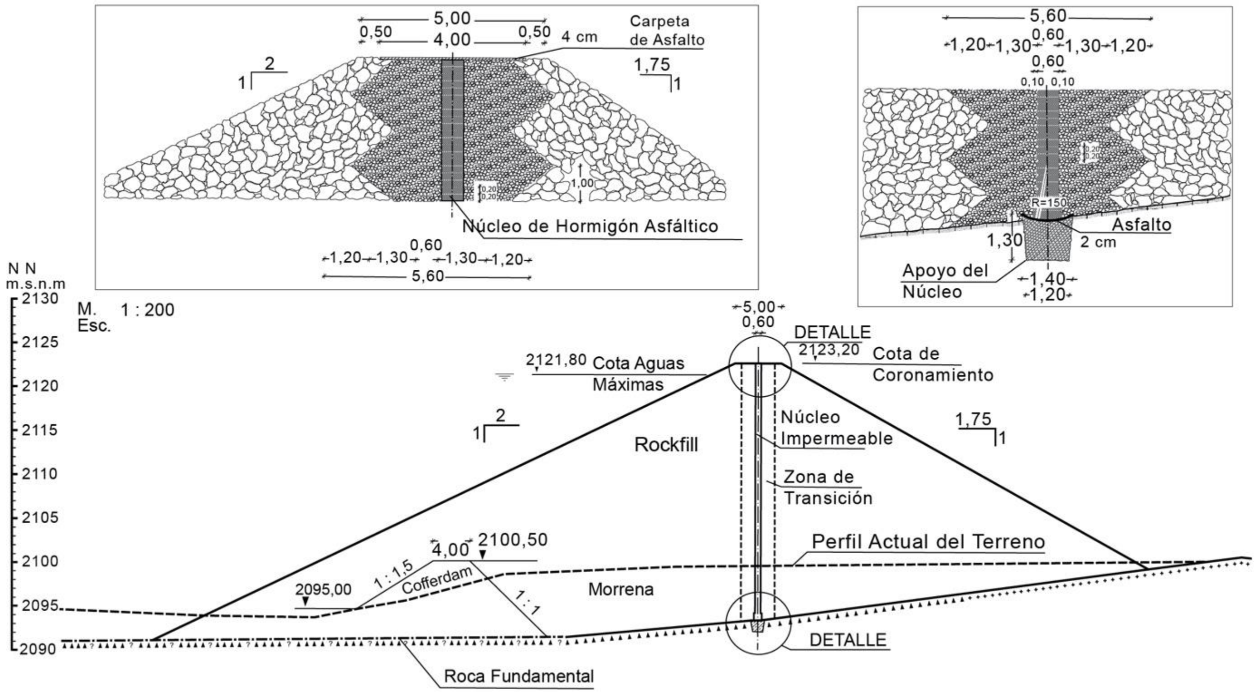


Fig. 27 - Cross-Section Design of Los Cristales Dam.

Fig. 28 - Crest of Los Cristales from left abutment

Fig. 29- Settlement asphaltic core

4.2 Rockfill dams with concrete face (CFRD)

Only two rockfill dams with concrete faces have been built in the country: Cogotí dam (83 m high) built in 1939 and Conchi dam (66 m high) built in 1976. The first one is a well-known case and it will be commented in more detail. The Conchi dam is also located in the North of the country and was also visited by MOP in 2011 and found it in good conditions [3], although no more details were available.

Cogotí dam is in the confluence of Pama and Cogotí rivers about 65 km south of Ovalle city. It has a reservoir with a maximum capacity of 150 million m^3. In the left abutment a lateral spillway excavated in rock and without gate allows a water flow of 5 thousand m^3/s. The dam is a concrete face rock fill dam (CFRD), completed in 1940, although the main body was finished earlier in 1938. The embankment, 160 m long, with a maximum height of 83 m and a crest width of 8 m and it was built with rock from an adjacent quarry placed in the dam without compaction. The u/s slope is 1.42-1.47:1 (H:V), while the d/s slope is 1.47-1.50:1 (H:V). A cross-section of the embankment is shown in Fig. 30. In the first 15 m, rock blocks with a maximum size of 1.5 m were used, which were just dumped by gravity in the dam site. Afterwards, the same material limited to a maximum size of 1.3 m was placed by mechanical means, which induced a slight compaction generated by the passage of trucks during the construction. The technical specifications requested the application of abundant sluicing with water (water pressure of 60 m) using a quantity of water equivalent to three times the volume of the sluiced layer. However, the available technical records of the construction indicate that most of the time the requested amount of water was not fully satisfied. Due to construction procedure, it is possible to conclude that Cogotí Dam is composed of a rockfill with poor densification.[17]

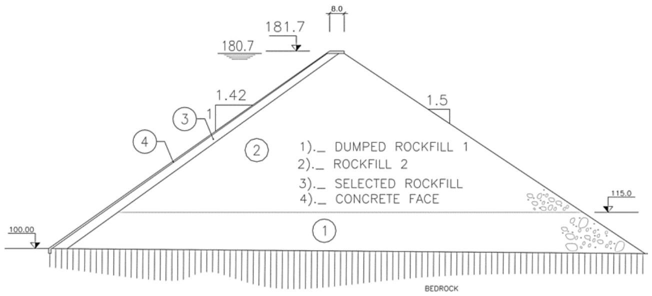


Fig. 30 - Cross section Cogotí Dam [17]

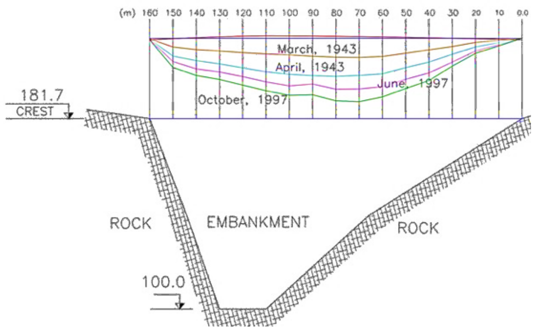


Fig. 31 - History of settlements measured in Cogotí Dam [17]

The shape of the valley gorge is shown in Fig. 31, where it can be seen a non-symmetrical profile, with a right abutment less steep than the left one and with a change in its inclination. This narrow gorge is mainly constituted by andesitic rock. Three important structural alignments were recognized in the site of the dam: N450–75E0, E-W and N-S, which have been related to the leakages observed throughout the abutments and foundation of the dam.

The face slab is made of a series of square concrete plates of 10mx10m, that have a thickness varying from 0.8 m at the base to 0.2 m at the top, where it continues as a short vertical retaining wall of 1 m high. The concrete face is resting on a layer with an average thickness of 3 m, constituted by hand-placed small rock particles. The joints between the concrete plates were sealed with water-stop copper pieces of 1.5 mm thick and 60 cm wide. The plinth at the foundation consists of a reinforced concrete cut-off wall that was initially designed with a depth of 3 m, but it was finally built with a maximum depth of 20 m and an average depth of 7 m. On the other hand, the plinth along the abutments is a concrete wall with a variable depth in the range of 1.4 to 2 m.

Several EQs have hit Cogotí dam, causing damages in the concrete face due to the settlements experienced by the poorly compacted rock fill. The registered settlements are shown in Fig. 31. It is interesting to mention that the maximum vertical settlement does not take place in the section associated with the maximum high of the rockfill. Indeed, the maximum vertical settlement occurs above the point where the bedrock presents a change in slope. The maximum settlement corresponds to control point M7. At the end of 2001, the total maximum settlement reached 138.7 cm, resulting in 1.7% of the high of the dam after 63 years of its construction [17].

4.3 Gravel dams with concrete face (CFGD)

Gravel dams with concrete faces and founded in fluvial deposits constitutes one of the most important developments of Chilean dam engineering. These dams are taking advantage of the natural conditions of many valleys in Central and Northern regions of Chile. These dams are represented by the following dams: Santa Juana built in 1995 that will be commented in some more detail and Puclaro, Corrales and El Bato that are briefly described as it follows:

Puclaro dam is 83 m high and it was built in 1999 and, as the other CFGD dams, is formed by a compacted gravel embankment with an u/s concrete face which is connected to a plinth and a diaphragm wall of plastic concrete (58 m) built through the fluvial foundation consisting mainly of sandy gravel that has an estimated maximum depth of 100m. The crest width is 8 m and u/s slope is 1,5:1 (H:V) and d/s 1,6:1 (H:V).

Corrales dam is 69 m high built in 2000–2004 and it is also a CFGD type of dam with compacted gravel and similar geometry as Puclaro and El Bato dams. It was not necessary a diaphragm wall.

El Bato dam is 55 m high and it was built in 2011, with similar geometric characteristics as Puclaro and Corrales and a diaphragm wall 40 m deep.

These 3 CFGD which location is shown in Fig. 24 were inspected by the MOP [15] a few days after the Illapel EQ of September 16, 2015 (M=8.4) and other dams also located in the III and IV Regions of Chile. Puclaro dam is 167 km distant from the EQ epicenter, Corrales 98,4 km and El Bato only 94,3 km from the epicenter. None of these dams presented damage of some importance, showing only typically local surficial rip-rap slides, movement in some auxiliary structures such as ladders, lighting poles and surficial cracks at the crest. In Corrales and El Bato dams that are closer to the EQ epicenter were also observed openings of the vertical joints of the concrete faces that required sealing maintenance and also cracks at some point of the union between the concrete faces and the concrete parapet.

The CFRD Santa Juana dam is located at approximately 17 km east of Vallenar City, in the north of Chile. It was completed in 1995, having a capacity of 166 million of cubic meters. The maximum height of the dam is 113.4 m, with a crest length of 390 m. It was constructed with rock particles with a maximum size of 1 and 0.65 m in the upstream and downstream supporting shoulders, respectively.

All the material was adequately compacted. A cross section showing the distribution of these two materials is presented in Fig. 32. It can be seen that the u/s and d/s slopes are 1.5:1 (H:V) and 1.6:1 (H:V), respectively. The concrete face has a variable thickness from 45 cm at the base to 30 cm at the top, where it is transformed in a parapet wall.[17]. The shape of the gorge of the rather narrow valley eroded by Huasco River is shown in Fig. 31. The profile is rather regular, probably due to the homogeneity of the rock properties at both sides of the river. A cutoff wall made of plastic concrete of 30 m depth was built in the fluvial material located in the riverbed.

In Fig. 33 are also presented the measured settlements at the control points located along the crest of the dam. It is observed that the vertical deformation is in some way shifted to the right, confirming the results observed in Cogotí Dam that suggest that the shape of the bedrock has an important effect on the distribution of the vertical settlements, static and seismic.

The maximum settlement after 4 years of operation was quite small, close to 1 cm. Several seismic events have hit the dam, which have not changed the general trend of the settlements. The Punitaqui EQ of Mw=7.1, hit the dam in October 14, 1997. The maximum horizontal accelerations recorded at the crest of the dam were 0.226g and 0.124g, in the E-W and N-S direction, respectively [17].

Taking advantages of the high seismic activity of the region and the accelerometers installed on the crest and toe of Santa Juana Dam, the natural period of the dam was evaluated through the transfer functions between the ratio of Fourier transforms of the motions recorded at crest and toe.

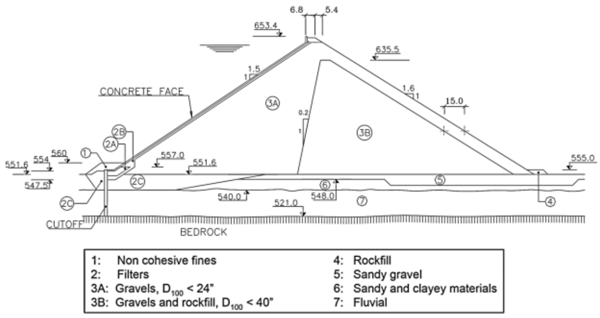


Fig. 32 - Cross section Santa Juana Dam [17]

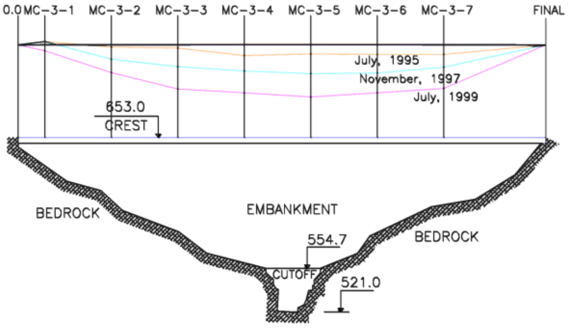


Fig. 33 - Profile of the valley and settlement distributions [17]

The result of the Spectral Ratio is shown in Fig. 34, where a peak is clearly identified at a period of 0.43 s, which would be associated with the fundamental period of the dam. It is important to mention that this value is coincident with the classical relationship for a dam (simplified as a triangular shape above a rigid half-space) if an average shear wave velocity of 700 m/s is considered for the compacted rockfill material [17].

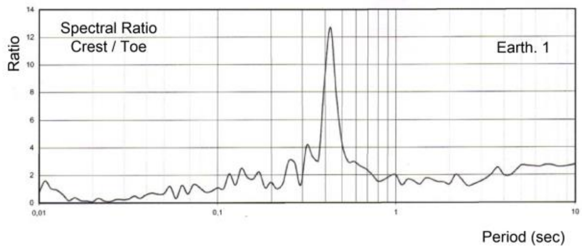


Fig. 34 - Spectral ratio Crest – Toe. Santa Juana Dam

5 CONCRETE DAMS

5.1 Concrete gravity dams

There only 7 concrete gravity dams in Chile with two of them built as early as 1911 in the Northern regions of the country: Laguna del Huasco dam (15 m high) built by MOP and operated for many years by an irrigation association in the III Region and still in use but not in optimum conditions and Sloman dam (40 m high) a private initiative that operated for many years under different hands for different purposes and today abandoned and full of sediments although with the structure apparently intact according to recent aerial photographs. There are no registers that these dams have suffer some significant damage from EQs but there is evidence that Laguna del Huasco did not suffer damage for the Punitaqui EQ of 1997 (Mw=7.1)

There are 2 concrete gravity dams, that were built around 1950: Marga Marga dam (21 m high) built in 1949 in the Valparaiso region by privates for irrigation purposes and Arroyo Salado dam (34 m) built in 1951 by a mining industry for water supply purposes. La Ventana dam (25m high) is another concrete gravity dam built by a mining industry in 1975 in the Central Region for water supply purposes. No damages because of EQs have been reported to the last two dams. No information of present situation of Marga Marga dams was found.

Hydroelectric company Endesa built two concrete gravity dams: Calabocillo dam (20 m high) built in 1975 and Polcura (26 m high) built in 1980. Both dams located in the Central Southern VIII region of Chile were not affected by the Maule EQ of 2010 (M=8.8) although they are located quite near the fault trace and initial epicenter of the EQ.

5.2 Concrete arch dams

Two arch dams have been built in Chile: one in 1910 (El Sauce dam) and other almost 60 years later (Rapel dam), both in operation.

El Sauce dam (26 m high) was built in 1910 in the Valparaiso region by private investors as part of a hydroelectric project that operated until 20 years ago and nowadays the reservoir is used for water supply purposes [25]. No damaged has been reported in this dam although is in a zone of high seismicity and not far from the Valparaiso EQ of 1985 (M=8.0) epicentre.

Rapel dam (112 m high) is a concrete gravity arch with double curvature. It was built in 1968 by Endesa and it was an important milestone in the development of Chilean engineering of dams, because of the extensive and intensive studies done in terms of rock mechanics, concrete technology, stress–strain and seismic analysis [20]. Numerical analyses were incipient in dam design in the sixties nevertheless a complex physical model was developed under the supervision of Prof. Lombardi from Switzerland.

This dam has been subjected to two majors strong EQs, the Valparaíso EQ of 1985 and the Maule EQ of 2010. The Valparaíso EQ of 1985 (M = 8.0) affected mainly the region of San Antonio port and the town of Melipilla, precisely the region where Rapel dam is situated, the dam is only 100 km from the EQ epicenter. The accelerograph installed in a tunnel in the left abutment of the dam registered the following PGA: 0,31g (E-W) and 0,14g (N-S) in horizontal direction and 0,11 g in vertical. An inspection carried out after the EQ concluded that globally the seismic performance of the dam was satisfactory, although with several damages that were considered no significant from the safety and operational point of view. The arch dam did not suffer any significant damage, but the appurtenant structures did have damage. The spillway walls were cracked and there was leakage at the wall of the right spillway. The upper part of one intake tower cracked and separated from the dam.

In one report it is mentioned that “the satisfactory performance of the dam was consistent with the excellent seismic capacity exhibited by many concrete arch dams (to that date)”. It is reported that the main damage occurred at the location where the intake structure become physically separated from the dam, indicating a possible structural drift and possible out-of-phase movements between adjacent structures [20]. There was a rise in the infiltrations coming from the u/s zone of both spillways, from 28 l/s in November 1984 to 78 l/s after the EQ. At that time, it was estimated that this was due to a damage in the water tightness plaques caused by relative movements between u/s spillways blocks during the EQ and a gradually diminishing of the flow could be expected.

The dam was designed in 1960 for a horizontal seismic coefficient of 0,12g. The critical load case was found at that time to be a transverse (u/s-d/s) EQ with the reservoir empty. Although no dynamic analyses were performed before or after the EQ, comparisons were made with the results of finite elements analyses of another similar dam (Sir dam, Turkey). That comparison indicated that Rapel should have performed satisfactorily under seismic loads like those induced by the 1985 EQ [20].

The Maule EQ of 2010 (M=8.8) also affected Rapel dam but less intensively that in 1985, although was relatively near to the fault trace of the EQ. The maximum PGA registered in an accelerograph located in the dam was 0,21g (horizontal). The joint between two adjacent concrete blocks next to the left abutment of the dam showed a rise of 0,5 mm. Seepage increased, as it did in 1985, along the right abutment, in this case from a normal 11 l/s to 40 l/s. Some concrete pavements at the dam crest cracked. It is understood that with time seepage returned to more normal values [20].

5.3 Rolled compacted concrete (RCC) dams

Pangue dam (121 m high) built in 1996 by Endesa–Pangue company is the first rolled compacted concrete dam constructed in the country. The dam is in the VIII region and relatively near the fault trace and epicenter of Maule EQ of 2010, but no damaged was reported as a consequence of this strong EQ (the world 6th in terms of magnitude).

Ralco dam (155 m high) built by Endesa in 2002 is also an RCC dam located 20 km approximately to the South of Pangue dam and consequently also relatively near to the epicenter of Maule EQ of 2010. Also, no damage has been reported for this high dam because of this major EQ.

Convento Viejo RCC dam (23 m high) was built in 2006 in parallel with the Convento Viejo earth dam forming the new Convento Viejo reservoir. This RCC dam has the spillway and water discharge of the reservoir and is also associated with a hydroelectric plant. This RCC dam and the neighbour earth dam are located almost over the fault trace associated to the Maule EQ as it is shown in Fig. 15. Although the earth dam suffered some damage as it was already commented, not damage was found in the RCC dam.

Mauro RCC dam (29 m high) was built about 2006 by a mining company to storage and deviate water from entering in a tailings deposit. This dam was subjected to the Illapel EQ of 2015 (M=8.4) and although is relatively near the EQ epicenter, around 80 km, no damage caused by this EQ has been reported.

6 TAILINGS DAMS

Mining started in North of Chile in the Inca´s time in the XVI century and continued during the Spanish until present time with Chile being the bigger world producer of copper and an important producer of molybdenum and other metals as well as non-metallic products. It was with the start of metal concentration process probably during the beginning of XIX century that mine tailings started to be produced and deposited in dumps or discharged in creeks and even rivers. With production increasing along time the need of controlled tailings deposits required the construction of dams to retain those tailings. Fig. 35 shows the development of the accumulated volume of large tailings deposits according to the register of the government mining institution Sernageomin (Servicio Nacional de Geología y Minería). It should be noted the first important increase in mining activity around 1950 and then in 1980 a sharp increase in deposited tailings that continues up to nowadays. In the same Fig. 35 is shown the register of the main individual authorized tailings deposit.

The characteristics and the evolution on the design, construction and operation of tailings dams in Chile have been described in detail elsewhere [27] [28] [29] [30][31].

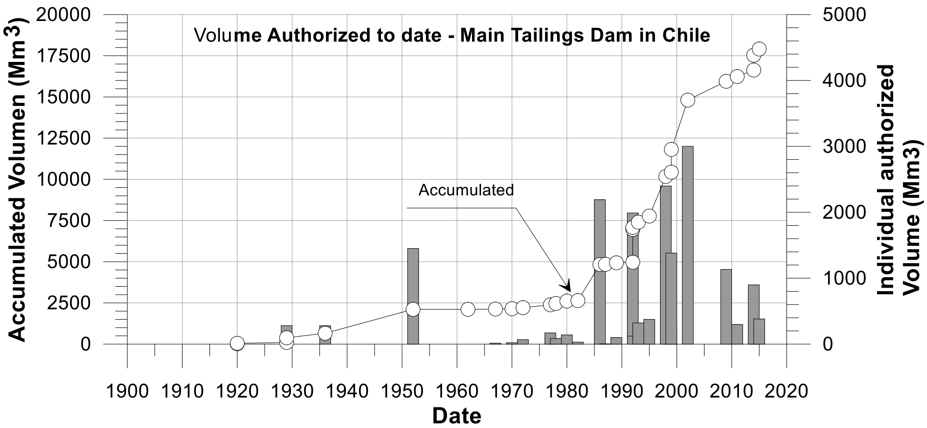


Fig. 35 - Accumulated volume of tailings and authorized deposits [26].

6.1 Tailings dams built before 1965

There are few records of old tailings deposits although several of them or their rests are still visible in the Northern more desertic areas, many of them destroyed after abandonment and as consequences of erosion, mud flows and probably also of some EQs, being in general quite vulnerable because of their nature since most of them have been built with uncompacted tailings. In the present study only those old tailings deposits that have been described in the literature or the press because of their failure affecting human lives are commented.

Most of tailings dams built before 1965 were built with the same tailings, in various cases combined with sand, when some tailings classification process was included. The usual construction system was the called upstream method in which the dam growths in the upstream direction, resulting that the dam formed by tailings or sand tailings (with some compaction generally) is founded on deposited tailings as shown in Fig. 36.

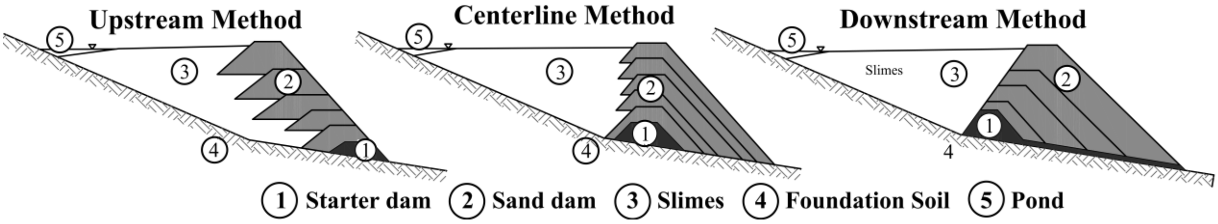


Fig. 36 - Methods of tailings dam construction [28]

The upstream construction relies on the strength capacity of the tailings for the stability of the deposit. Because of that condition this method requires intense ongoing supervision and control during all its construction/operation period (longer tailings beaches /small ponds/ phreatic line control, foundation conditions control and others). This construction method has been successfully used for decades in several countries keeping the adequate control and generally using flat downstream slopes and well specified and controlled compaction as the construction proceeds. This was not the case of most of the upstream tailings dams in Chile where much steeper downstream slopes were used such as 2,0 to 2,5:1 (H:V) with no or little compaction, poor or inefficient cycloning effort if any and many times and too high dam raising velocities. This type of upstream construction without adequate design and poor control is highly vulnerable to failure by liquefaction during strong EQs or even by static liquefaction as in the case of Feijao Dam 1 in Brazil [32]. In Table 4 are shown the best-known tailings dams that have failed during EQs in Chile [28]. All of those in the list correspond to upstream tailings dams. Of the dams shown in Table 4 only Barahona, El Cobre and Cerro Negro could be considered as large dams based on their maximum heights. Beside the failed dams shown on Table 4 during the 2010 EQ other two dams failed that are rather small tailings deposit and not proper dams. Fig. 37 shows the 8 tailings dams that failed during strong EQs, independently of the dam dimensions or condition, according the EQ magnitude and the distance of the dams to EQ epicentre. As it can be observed in the figure, all the failures occurred for EQs with M ≥ 7.0 but it is quite probable that other smaller and older dams could have failed in the past for EQs of lower magnitude and have not been registered because of different circumstances (too distant of population areas, too small and relatively low impact on environment).

Table 4 – List of failures upstream tailings dams

Tailing Dams	Date	Fatalities	EQ Magnitude
Barahona	1928	54	Ms=8,3
El Cobre Nº1, Nº2 y Nº3	1965	>200	Ms=7,4
Veta del Agua Nº2	1981	-	Ms=6,5
Cerro Negro	1985	-	Ms=7,8
Veta del Agua Nº1	1985	-	Ms=7,8
Nº5 Veta del Agua	2010	-	Mw=8,8
Las Palmas	2010	4	Mw=8,8

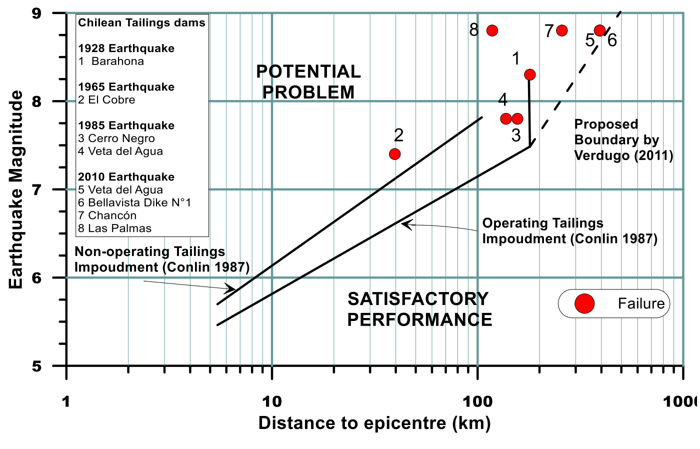


Fig. 37 - Tailings dams failures in 1928, 1965, 1985 and 2010 EQs [33][34]

The date of 28th of March 1965 became a milestone in the practice of design and construction of tailings dam in Chile. Because of the failure of El Cobre tailings dams that caused more than 200 fatalities and significant environmental damage, the construction of tailings dams constructed by the upstream method was legally forbidden (decree N°86 of 1970) [35] and this restriction has been kept until the present time (decree N° 248 of 2007) [36] The failures that occurred after 1965 correspond to dams that were built before 1965 using the upstream method or relatively new dams, as Las Palmas, that had a substandard design and poor operation and abandonment conditions

El Cobre tailing dams is a system of three copper tailings deposits located approximately 10km from the town La Calera, in the Valparaíso Region. The oldest ones started operation in 1930 (Old Dam and Small Dam), while the third dam (New Dam) began to be built in late 1963. As a result of La Ligua EQ (M = 7.4), on March 28, 1965, the failure of two of the dams of the El Cobre deposit system occurred, released approximately 2 Mm3 of tailings, reaching a distance of 12 km in a few minutes, killing more than 200 people, destroying the El Cobre town located downstream [37] [39].

In Fig. 36 three aerial views of the El Cobre deposit are presented (two before and one after the failure), while in Fig. 37 a cross section of the Old Dam is schematically shown.

Fig. 38 - El Cobre tailing dams, before and after March 28, 1965 [37]

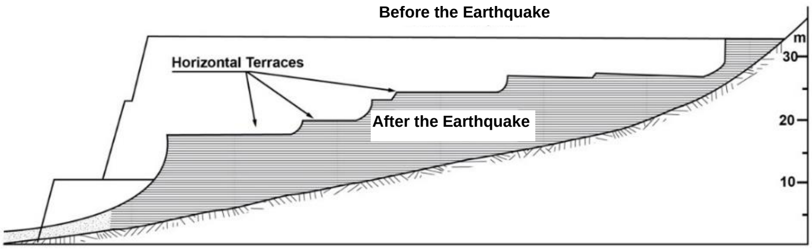


Fig. 39 - Failure of El Cobre tailings dam (deformed scale) [37]

Las Palmas tailings dam experienced the most catastrophic seismic failure triggered by the 2010 Maule EQ. The mine was operating from the early 1980's until 1997, when the mining activity finished, and the tailings pond was partially covered with a 15 cm layer of gravelly material. The tailings deposit consisted of two dams (a lower one and an upper one). Both dams were constructed in stages using the upstream construction method

During the Maule EQ of 2010 (M = 8.8), the tailings deposit retained by the lower dam of Las Palmas liquefied, the eastern portion of the tailings impoundment nearest the mine facilities was breached, and a tailings volume in excess of 100,000 m^3 flowed a distance of about 0.5 km, killing 4 people in a house that was buried under four meters of tailings (Fig. 40).

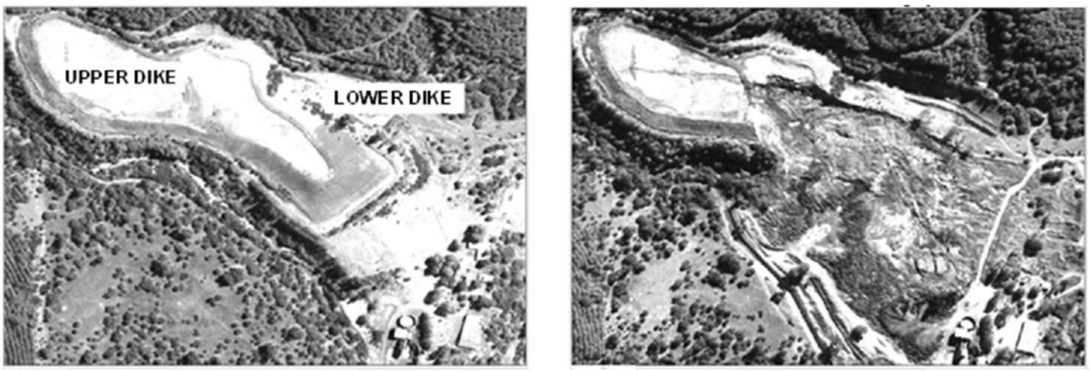


Fig. 40 - Failure of Las Palmas tailings dam during the Maule EQ of 2010. [14]

The tailings liquefied again (less severely) during the aftershocks of M>5.0. During these aftershocks, new evidence of sand boils and ejecta as well as water draining from the subsurface was observed. A number of these liquefaction features were observed in the terraces associated with the upstream construction method at the western end of the tailings impoundment area even though no full breach of the structure occurred at that location. Field surface wave measurements conducted in the non-failed tailings indicated shear wave velocity of the order of 250 m/s, and dynamic cone penetrometer soundings performed in both non failed and failed tailings materials yielded blow counts of about 10 [14].

6.2 Tailings dams built after 1965

After 1965 no upstream tailings dams have been built in Chile, although a few older upstream tailings dams are still existent but not in operation and their gradual removal and in some cases, re- mining of the tailings is underway or next to. Table 5 presents is a list of the main tailings dams that have been built after 1965. All of them use the downstream construction method (explained in Fig. 34) except for El Torito dam that at a certain height changed from downstream to center line construction method. In Table 5 are not included those tailings deposits that do not necessary require a dam or at least an important dam such as deposits with paste tailings or filtrated tailings.

None of these dams has suffer some significant damage during the relatively recent EQs of Valparaiso 1985 (M=8.0), Maule 2010 (M=8.8) and Illapel 2015 (M=8.4) although most of these dams are at a relatively short distance of the epicentres, as it is shown in Fig. 41 for the particular case of the Illapel EQ of 2015.

6.2.1 Earth dams to retain tailings

There are 9 earth tailings dams built after 1965 starting with Los Leones in 1980. These dams have maximum heights (completed or expected to be reached) between 29 m and 198 m. Most of them correspond to conventional zoned earth dams but Talabre and Piuquenes dams are compacted sand dams constructed with the downstream method. Three of these dams are no more in operation: Los Leones, Colihues A and El Indio dams, but only the last one is already closed. None of these dams have suffered significant damage during EQs as already mentioned.

From these earth tailings dams a good example is Los Leones dam with a maximum height of 198 m (160 m*). This dam is one of the largest conventional dams designed to impound tailings. It was built in 4 stages, the first one started to be built in 1980 reaching its maximum height at the end of the 4th stage in 1999 when stop being raised and then been kept up to the present just an emergency deposit. In March 1985, the Valparaiso EQ (M=8.0) shook the dam in its 2nd stage with a height of about 108 m with the water pond 30 m below the dam crest. The dam is a conventional, compacted earth fill embankment. It has an upstream core (inclined because it has been constructed in stages) built of morainic deposits, an inclined chimney drains and filter and transitions zones on the downstream side of the core. The downstream shoulder is built of compacted earth fill, borrowed from alluvial fans. Near the toe, a secondary drainage system collects seepage from the underlying aquifer and paleo stream in the dam foundation. The dam responded elastically to the 1985 EQ, as post -EQ surveys indicated no measurable crest settlements. No cracking nor other disturbances of the dam slopes were reported [20]. A dynamic back analysis of the dam as it was in 1985 and considering the dam loaded with the seismic registers of the accelerometers installed in the dam allowed to calibrate the numerical model used to estimate the dynamic behaviour of the final dam for the design EQ, that provided relatively small deformations for the final conditions. In February 2010, the dam with its full height was shock by the Maule EQ (M=8.8) not showing measurables settlements or deformations [40].

Table 5 – Tailings dams built after 1965 including earth, waste rock and sand tailings dams.

Name	Max height (§) (m)	Dam length (m)	Capacity (Mm³) (+)	Operation start	Type	Operation end
El Cobre 4	68	1,140	31	1969	DS	1992
Piuquenes	58	500	20.5	1970	DS	1980
Pérez C. 2	115-135	500	84	1978	DS	1992
Los Leones	160	500	140	1980	RF	1999
Colihues A	83	1,200	160	1981	EF	1986
Talabre	40	5,300	1451	1985 (*)	EF/DS	OP
Caren	54	950 (+)	300	1986	EF	OP
El Indio	79	290	42	1987	EF	1999
Pampa Austral	29	700 (+)	100	1989	EF	OP
Laguna Seca	100	>2000 (+)	3000	1990	EF	OP
El Chinche	110	470	14.5	1992	DS	1999
Las Tórtolas	150-170	1,700 (-)	690	1992	DS	OP
Torito	78	2,190 (+)	122	1992	DS/CL	OP
Candelaria	163	2,400 (+)	260	1995	RF	2017
P. Pabellón	90	4,500 (+)	2400	1998	RF	OP
Quillayes	175-198	1,600 (+)	360	1999	DS	2008
Ovejería	58-120	3,600 (+)	235	1999	DS	OP
Zaldívar 3/3A	45	2000 (+)	15	2005	EF	OP
El Mauro	237	1,450 (+)	1088	2008	DS	OP
Esperanza	w/i	>3000 (+)	468	2011	RF	OP
Carmen de Andacollo.	150 (-)	w/i	297	2011	RF	OP
La Brea	255	1800 (+)	318	2014	EF/RF	OP
Sierra Gorda	190	w/i	900	2014	RF	OP
Los Diques	156	w/i	337	2017	EF	OP

Note: The data shown in Table 5 are approximated, obtained from different sources. Some of the deposits have been expanded, not all them registered in this table. (-) Only main dam; (§) Approximate final height; (+) Approximately; (*) Deposition began in 1952, just in 1985 was necessary confinement walls. DS: Downstream sand dam; CL: Centerline sand Dam; RF: Rock-fill dam; EF: Earth-fill dam

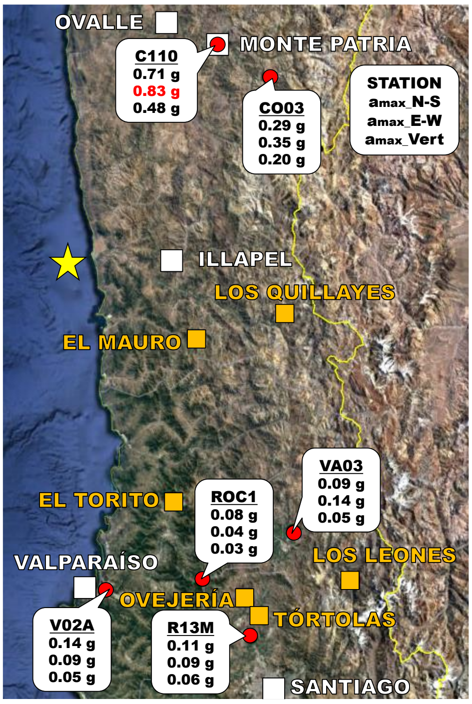


Fig. 41 – Tailings dams, Illapel EQ

6.2.2 Rockfill dams retaining tailings

There are two main cases of rockfill tailings dams in Chile and both using mining waste rock fills: Pampa Pabellón dam and Candelaria dam.

Pampa Pabellón dam initially planned for a maximum height of 90 m and that it is still in operation. The compaction of the rockfill is done by the passes of high tonnage mining trucks and passes by bulldozer that work in the distribution of rock dumped by the trucks. A transition (filter) between the rockfill and the tailings is obtained from borrows near dam (gravelly soil).

Candelaria dam is also a waste rock dam built with waste rock from the nearby mining open pit. The dam started its construction in 1995 and ended its operation in 2017 with a height of 171 m*. It was designed as a dam without a berm supporting it, but it was known that soon a large waste rock dump will be formed immediately downstream of the dam. The rockfill was compacted with the passes of mining trucks and D–8 bulldozers with a zone about 10 m width near upstream slopes in layers of 2 m thickness and then in the downstream direction a zone with layers 4 m thickness and towards downstream zones of 8 m thickness and 16 m thickness with no compaction. The dam was founded partially over a wide alluvial channel with a generous drain built inside and no cut off. About 1 km downstream of the dam the alluvial channel was much narrow, and a deep clay cut off about 27 m deep was built. The seepage water interrupted by the cut off was then deviated to a tunnel towards a shaft where the water (over 200 l/s) was pumped back to the processing plant. For many years Candelaria mine had the record as the mine with the lowest freshwater consumption.

6.2.3 Sand tailings dams

After the collapse of El Cobre dam in 1965 a new dam was built in the mine in 1969. El Cobre N°4 dam (68 m high) was constructed with the downstream method and using compacted sand obtained for the first time from tailings in a centralized cyclone station with strict control of fines content in the sand and hydraulic transport of the sand with positive displacement pumps. This design concept was sometime later in 1978 applied at the Pérez Caldera N°2 tailings dam (135 m High) [28] that was fully instrumented allowing to verify the high positive behavior that this type of dam could represent. Further on this concept was applied and improved in many details at Las Tórtolas dam, built in 1992 (170 m estimated maximum height) that has become internationally known as the representative example of what has been recognized as the Chilean sand tailings dam practice [28] [41]. Other dams of this type follow: El Chinche (110 m), Torito (78 m), Quillayes (198 m), Ovejería (120 m) and El Mauro (237 m). Of these dams Las Tórtolas, Ovejería and El Mauro dams are still in operation. None of these dams that have followed the general concept applied at Las Tórtolas have suffered any significant damage during EQs.

Basically, the contrast in permeability between the tailings or slimes and the compacted sand is the key factor associated to the Chilean tailings dams as shown in Fig. 42.

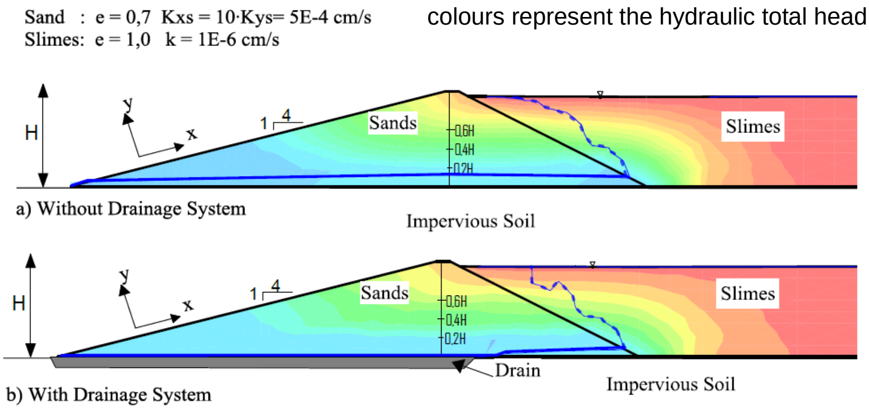


Fig. 42 - Example of flow net in a typical cross section of a downstream sand tailings dam [28]

The following are the characteristics that explain the good performance of these sand tailings dams specially in such highly seismic country as it is Chile:

- Centralized cyclone station to guarantee quality of specified sand, mainly in terms of maximum content of fines (particles below mesh #200 ASTM).
- Specification of maximum fines content between 15% to 20% to guarantee an ample contrast of permeability with that of the tailings or slimes deposited.
- Distribution of sand from the crest of the dam as it grows forming a gently downstream slope to facilitate sand compaction with bulldozers (typically between 3,5 to 4,0:1 (H:V).
- Sand compaction to a minimum of 95% Standard Proctor or equivalent to guarantee a dilatant behavior of sand at least for a certain threshold of confining pressure.
- Generous drains, basal and lateral fingers, capable of instantaneously get rid of water during construction (operation) stage obtaining a low phreatic line at the level of drains. Once operation is finished the drains will have and extraordinary high capacity to guarantee adequate conditions at closure.
- Discharge of tailings and/or slimes over the upstream slope (typically 2:1 (H:V). An impervious geomembrane is placed over the upstream slope in order to avoid the possibility of water to reach the sandy slope although in depth such membrane would be partially destroyed due to the drag downward forces produced by the consolidation of tailings, but at that moment the tailings and no the water will be in contact with the sand body of the dam.

Las Tórtolas Tailings Storage Facility (TSF) is located at an elevation of 700 masl, 45 km north of Santiago, in Chile's central valley. In this dam, shown in cross section in Fig. 43, conservative design criteria included initially a double cyclone station to guarantee 10% maximum fines content (FC) in the sand. The design considered the implementation of a network of instruments, including piezometers and accelerometers. This is the first TSF that was subjected to a dynamic stability and deformational analysis using the finite differences method (DSAG at the time, which later gave birth to FLAC).

Satisfactory performance of this TSF, which began operating in 1992, made it possible to increase FC to 15% a few years later and to define a new maximum height of 170 m. The current height of this dam is near 120 m. Design of this deposit initially set a maximum capacity of 1,000 million tons of tailings, and recent estimates increased that capacity to 2,000 million tons. The deposit has two others smaller cyclone sand dams, all constructed using the downstream method. In the main dam, the starter dam consists of compacted earth 17 m high. Approximately 5 m of loose alluvial soil were excavated under the starter dam and under part of the sand dam, after which dynamic compaction was applied to the foundation. The dam is equipped with generous basal drains that were built in stages. The dam is constructed with cyclone sands compacted to 95% Proctor Standard. The sand was initially deposited forming a slope of 1:4 (V:H). After a few years of operation, and after verification of very satisfactory dam performance confirmed by density controls and piezometric level records, the deposition slope was changed to 1:3.5 (V:H). A final slope of 1:3 (V:H) is considered for the closure stage [28].

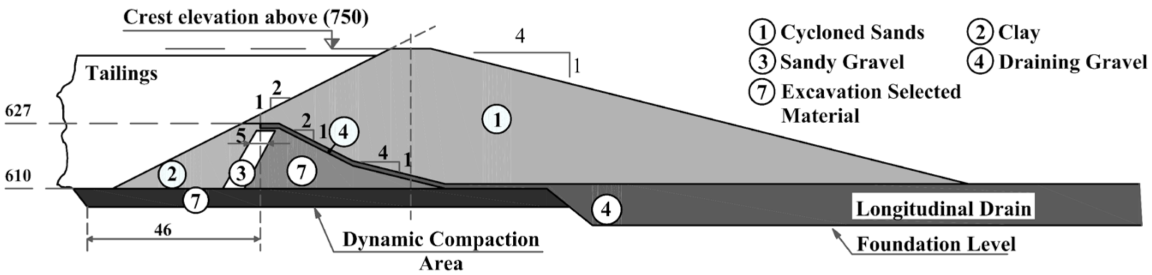


Fig. 43 - Cross section Las Tórtolas dam [28]

Although this dam had not yet been built at the time of the Valparaíso EQ of 1985, there were already accelerometers installed in rock at the dam site and the registers of this strong and relatively close EQ were considered later in the dam dynamic stability analysis. Two large EQs have been registered during the operation of the TSF: the February 27, 2010 EQ Mw=8.8 and the September 16, 2015 EQ Mw=8.4. In both seismic events, no-significant damage or deformation was observed. The instrument located in hard soil, associated with the main dam, recorded the data show in Table 6 [42].

Table 6 - Recordings of large EQs in Las Tórtolas Main Dam (1).

Date	M	Dist. (Km)	D (km)	Loc.	PGA (g)	PGA (g)	Vert. PGA (g)
March 5, 1985 (2)	Ms=7.8	~105	33	Basal soil Downstream	0,172 N64°E	0,144 N26°W	0,106
Feb 27, 2010	Mw=8.8	~420	30	Basal soil Downstream	0,18 N64°E	0,18 N26°W	0,129
Sept 16, 2015	Mw=8.4	~190	23	Basal soil Downstream	0,066 N64°E	0,064 N26°W	0,045

M= Magnitude according to U. de Chile
Dist.= Distance from epicenter to the dam D= Depth of earthquake
Loc = Location site of the accelerograph
PGA= Peak Ground Acceleration (Horizontal)
Vert. PGA= Vert acceleration in g's
Note: (1) Baseline corrected
(2) Epicenter data according to USGS. https://www.usgs.gov/

The Quillayes Tailings Storage Facility (TSF) is of special interest, due to a series of conditions that translated into severe design, construction, and operating restrictions. Los Pelambres mine project is located 300 km north of Santiago with the TSF located near the other mine facilities. The site is in the foothills of the Andes Mountains, at an average elevation of 1,400 masl, distant some 85 km from the Pacific Ocean. The operation of the TSF started with a planned maximum capacity of 257 million tons of dry tailings, but a final capacity of 360 million tons was achieved with the final height of 198 m* of the tailings sand dam [43].

The main dam is a tailings sand dam, constructed by hydraulic deposition of sands obtained from the cycloning of the produced tailings, originally designed to have a maximum height of 175 m after 7 years of operation. The starter dam was a 70 m-high compacted earth embankment. The dam foundation consists of fluvial deposits in the central area of the dam, colluvial deposits, and / or alluvial terraced deposits in the right abutment and intrusive rock (granodiorite type) in the left abutment.

The dam was raised following the downstream construction method, through continuous hydraulic deposition of tailings sands containing no more than 18% fines (material passing ASTM 200 mesh). The inclined deposition surfaces were compacted by tandem bulldozers and smooth vibratory rollers, with 1: 3.5 (V:H) to 1:4 (V:H) downstream slopes. Tailings placement stopped at the end of 2008.

Static and pseudo-static stability analysis using limit equilibrium methods were performed. The deformations under seismic loads were estimated carried out different methods: The pseudo-dynamic analysis proposed by Makdisi and Seed (1978) and formal numerical analysis with FLAC2D and FLAC3D. Both numerical analysis (2D, 3D) did not show evidence of failure by shear resistance or excess of deformation that could affect the global stability. The design earthquake (MCE) was of Ms=8.3, with epicenter 123 km from dam an 23km of depth, with PGA of 0,37g (horizontal) and 0,25g (vertical) is hard soil. The September 16, 2015 EQ of Mw=8.4 was at the same depth and remarkably close to the epicenter of the design earthquake. According to the post-EQ inspection by the Engineer of Record, non-significant deformations or instability was detected [44].

7 DESIGN CRITERIA, PRACTICE AND LEGISLATION

For many years in Chile there was no written design criteria and recommended practices, neither specific legislation applied to the design, construction, and maintenance of dams. Dam engineering in the country in many cases followed the experience of institutions and engineering companies from Europe and North America. The creation in 1887 of the MOP and the establishing of the State owned electric company in 1943 were important milestones in Chilean dam engineering. After 1930 Soil Mechanics was starting to be developed as an engineer discipline and after 1940 a series of local engineers travelled abroad to postgraduate courses on this discipline thus reinforcing the capacity of the local institutions. In the country there were no local norms or legislation that regulated dam design and in general the recommendations and guidelines from international organizations were followed.

As part of the seismic dam design the international recommendations were followed, basically verifying the seismic stability of dams through a pseudo static approach applying seismic coefficients K_h and K_v For many years, a value of K_h=0,10 was applied for most dams. For the design of Rapel concrete arch dam in the 1960´s a K_h=0,12 was used [45] . The discussion on which is the reasonable value to be used has gone for years and no definitive answer has been obtained [46]. Since the 1970´s the more used values of K_h varied from 0,15 to 0,20. In 1975 ICOLD [45] insisted that the seismic stability of dams should consider a dynamic approach as already proposed by some researchers in UK and US [47][48]. ICOLD insisted in the need of proper seismological studies to define the seismic loads to be applied. The first seismic dynamic analysis using artificial EQ and finite elements done in Chile was for Colbún dam (116 m high) in 1974 [49] and then continue been applied for many other dams.

Legislation on dams, mainly earth dams, started in Chile with 1981 review of the Water Code, that gave the responsibility of approving dam projects to the Dirección General de Aguas (DGA). In 2015 it was issued the present decree N°50 of DGA [50] that established the requirement of pseudo – static and dynamic seismic stability analysis for dams with height over 30 m. For dams with heights between 15 and 30 m only pseudo static analysis is required. This decree established the following minimum safety factors (SF): SF_{static}≥1,4; $SF_{pseudostatic}$≥1,2 and $SF_{postEQ}\geq 1{,}0$ and $SF_{postEQ}\geq 1{,}2$ for tailings dams. Currently for large dams the maximum credible EQ is required to be considered. There were no specific requirements for tailings dams until the failure of El Cobre tailings dam in 1965. In 1970 a special decree N°86 [35] was issued jointly by the DGA and Sernageomin. This decree banned the construction of upstream tailings dam, being the first country banning a specific type of dam. The most recent legislation related to tailings dams in Chile is Decree N°248 [36] that replaced Decree N°86 in 2007 and it establishes special conditions for sand tailings dams – dams in. The percentage of fines in the sand cannot be higher than 20% passing n° 200 mesh.

It should be emphasizes that for "upstream tailings dam" it was designated the type of dam that was commonly built in Chile mining before 1965, without compaction, poor or inexistent cycloning of tailings, steep downstream slopes and uncontrolled rate of dam raising. The Chilean legislation has not recognized other possible upstream construction procedures as it is the practice in other countries [51][52].

For all types of dams, except the banned upstream tailings dams, dam design and construction in Chile has considered the international best practices as they are developed, following recommendations of organizations like ICOLD, ICMM, CDA, MAC and others.

8 CONCLUDING REMARKS

The Chilean territory located just above the collision between the Nazca and South American plates is among the most seismic active region of the planet. Indeed, in the last 70 years Chile has been subjected to 23 strong EQs of magnitude M ≥ 7.5 and five with M ≥ 8.0 and one of magnitude 9.5 (the largest ever registered in the world). The epicenters of these EQs are distributed along all the Chilean coastal zone mainly from North of the parallel 40°S to the Northern border of Chile approximately parallel 18°S which coincides with the territory where most of dams have been built. Considering how narrow is the Chilean territory (200 km average width), all the constructed dams have experienced more than one strong EQ. This is probably the most challenging natural scenario to which a large number of dams of different types and during a long period of time have been tested. The study allows to conclude that in general, despite the numerous strong EQs occurred in the last 70 years, the behavior of the Chilean dams have been remarkably good, except in those cases in which evident inadequate design and/or construction was adopted.

In summary the study of dams built from 1853 up to 2015 showed that:

- No concrete dams (built 1910 to 2012) suffered meaningful damage.
- Only two relatively small earth dams, built in the 1920´s by private agriculture association suffered serious damage. Other private irrigation small earth dams had suffered some but reparable damage.
- No major earth dam, rockfill dam or gravel dam has suffered meaningful damage, except Cogotí rockfill dam and Coihueco earth dam that had suffered reparable damage.
- No large tailings dam built after 1965 has failed or had suffered meaningful damage.

The notably successful performance of dams of all types under the extreme conditions imposed by the high seismicity of the country could be explained by the empirical experience developed inside the country, in government institutions, individual professionals and engineering companies always conscious of foreign experience and the recommendations and guidelines of international institutions like ICOLD and others.

It is important to mention that in the particular case of tailings dams, after the collapse, or flow failure, of El Cobre Dam occurred in 1965, the upstream method of construction, as it was applied by Chilean practice, was banned by law in 1970. There are no upstream tailings dams approved and built in Chile after 1965. The tailings dams that have failed after 1965 are mostly relatively small upstream tailings dams built before 1965.The Chilean experience can be used as an empirical evidence showing that all types of dams, including tailings dams are safe structures if designed and constructed according to sound engineering concepts and following the recommendations of the best available practice. It is worth to note that Chilean dam engineering has developed during the last 40 to 50 years a considerable knowledge and experience specially with two types of dams that have performed quite successfully in the seismic conditions of the country: concrete face gravel fill dams founded on alluvial soils including a diaphragm wall connected to the concrete plinth and downstream sand tailings dams reaching heights over 200m.

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