CONVENTO VIEJO DAM: BEHAVIOR AFTER THE FEBRUARY 27TH 2010 CAUQUENES EARTHQUAKE IN CHILE

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ABSTRACT

The Convento Viejo embankment dam is a zoned earth dam, 32 m high, founded in alluvial deposits, whose construction was completed in 2006–2008. The site is located near the town of Chimbarongo, 160 km south of Santiago, Chile's capital city, about 85 km from the coast and 220 km from the epicenter of the earthquake that took place on February 27th, 2010. This article examines the seismic behavior of the dam, compares the accelerations measured during the earthquake at three different levels in the dam with those estimated from dynamic analysis performed during the design phase. The paper also comments on piezometric levels and settlements induced by the earthquake.

Keywords: Zoned earth dam, geotechnical behavior, Cauquenes earthquake, seismic response.

INTRODUCTION

The Convento Viejo embankment dam, located 160 km south of Santiago nearby the town of Chimbarongo, is situated above the creek of the same name. This embankment dam, which is 32 m high, is a zoned earth dam with an impervious clay core and it is separated by a small hill of a concrete dam where spillway and gates are located. The two dams allowed to form a reservoir with a storage capacity of 237 Mm^3, a flooded area of 4,500 ha and a network of channels more than 90 km long.

The construction of the project started in 1973, but work was halted in 1975 due to financial problems. The construction was resumed in the period 1978-1979, but once again was stopped due to changes in financing policies implemented by the Chilean government. Regarding the earth zoned dam the works that were completed in the above-stated periods included a slurry wall, the cofferdam of the earth dam, water diversion and water delivery tunnels, emergency spillway, and the excavation of the abutments of the earth dam. Figure 1 shows the state of the works by the mid 1990's and Figure 2 shows the construction process for the slurry wall, which reached a depth of 55 m, and a length of 500 m using panels 7.2 m wide. The cofferdam and the slurry wall allowed the formation of a smaller reservoir that was in operation until the dam completion in 2008. Between years 2000 and 2002, the engineering studies were updated and the designs modified in order to accommodate the existing conditions at that time. The construction of the earth and concrete dams was initiated in 2006 and finalized in 2008. Figure 3 shows the present condition of the embankment dam.

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DESCRIPTION OF THE EARTH DAM

The earth dam, which has an approximate volume of 2 Mm^3, has a central impervious clay core, upstream and downstream compacted gravel shoulders with upstream and downstream intermediate filters of selected granular materials. The typical section of the earth dam is presented in Figure 4, which also illustrates the distribution of the different types of materials that were used in its construction.

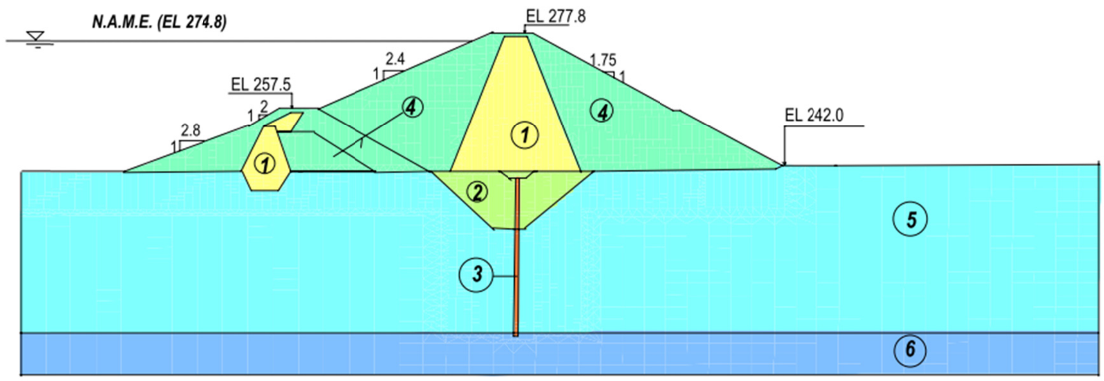
Figure 1: Panoramic view of the dam operating only with the cofferdam (ICOLD, 1996).

Figure 2: Construction of the slurry wall (Bachy, around 1974).

Figure 3: Panoramic view of the embankment dam (ARCADIS Geotecnica files, 2007).

The earth dam is founded on fluvial material of the Chimbarongo creek, formed mainly by gravels and sands. The fluvial deposit has a maximum thickness of 55 m beneath the central part of the dam. Near the surface this deposit presents a stratum of sandy and clayey gravels (GP-GC), with a maximum thickness of about 10 m in the central part of the dam, with a relatively high degree of compactness. Beneath this stratum, in turn, it was possible to identify silty sands with few fines, which typically classify as SP-SM or SW-SM, showing also a relatively high compactness. The thickness of this layer is 15 m in the central part of the dam. Finally, beneath these materials and resting upon the basal rock, there are sandy gravels that classify as GW or GP, and which present relatively high densities too.

A cut-off trench in fluvial material 7m to 16m deep was excavated along the top of the slurry wall and then filled with compacted clay. According to the available information regarding the history of the project, due to the onset of winter and the consequent flood risk, an emergency action was then taken, filling the trench partially beneath water and most probably without compaction control, in the right portion of the future dam. The thickness of this fill was about 10 m for a length of approximately 180 m. The material for this emergency fill was extracted from the excavation of the trench itself. A geotechnical profile along the axis of the dam is presented in Figure 5, where it is possible to appreciate the thickness of the fluvial deposits, the soil-rock contact, the cut-off trench fill and the emergency fill, mostly a sandy gravel fill.



1 Compacted clay core (CL)
2 Compacted clay (CH/CL). Fill trench
3 Slurry wall
4 Shoulders (Sandy gravel type GW / GP)
5 Fluvial deposits, Chimbarongo creek (GP, GM and SP/SM lens)
6 Bedrock

Figure 4: Cross section of the earth dam (ARCADIS Geotecnica, 2002)

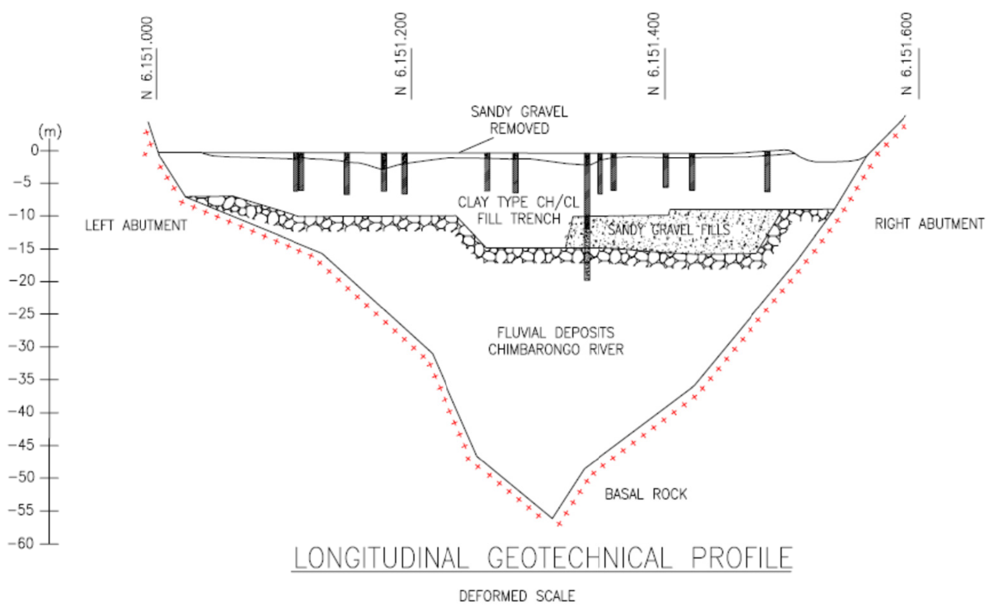


Figure 5: Longitudinal section along the earth dam axis (ARCADIS Geotecnica, 2002)

DESIGN OF THE CONVENTO VIEJO EARTH DAM

Geotechnical Parameters

The geotechnical characterization of the materials of the dam and those of the alluvial deposits were determined through various field investigation campaigns and in-situ as well as laboratory tests. Based on this information a determination of representative geotechnical parameters was achieved be used in static, pseudo static and dynamic stability analyses, as well as in water seepage models. The most relevant parameters adopted for the designs are presented in Table 1.

Table 1: Representative geotechnical parameters used in the design of the earth dam

Materials	γ_d (t/m^3)	$E_{drained}$ (t/m^2)	ν	c (t/m^2)	ϕ (°)	k (m/s)	n
1 - Core earth dam	1.73	$2{,}600(\sigma'_3)^{0.16}$	0.3	0.5	30	1E-7	0.347
1 - Core cofferdam	1.67	$2{,}400(\sigma'_3)^{0.16}$	0.3	0.3	28	1E-7	0.370
2 - Clay trench	1.45	$425(\sigma'_3)^{0.51}$	0.3	2.0	15	1E-8	0.453
3 - Slurry wall	-	40,000	-	50	36	1E-8	-
4 - Shoulders	2.27	$6{,}000(\sigma'_3)^{0.5}$	0.3	0	40	1E-4	0.160
5 - Fluvial deposits foundation	2.20	$6{,}000(\sigma'_3)^{0.508}$	0.25	2.5	45	1E-4	0.185
5a - Sandy gravel fill (trench)	1.90	$3{,}000(\sigma'_3)^{0.508}$	0.35	0	45	1E-4	0.296

γ_d : Dry density
$E_{drained}$: Drained deformation modulus
σ'_3: Confining pressure
ν: Poisson coefficient
c: Cohesion
ϕ: Angle of internal friction
k: Permeability
n: Porosity

Dynamic analyses

The tension-deformation behavior and the dynamic response of the earth dam were analyzed using the finite differences method. The analytic tool used was the FLAC 2D program in its version 4.0. The construction of the dam was simulated considering three construction stages. The deformations generated in the dam by its own weight were evaluated in two sections of the earth dam as it was their interaction with the slurry wall and clay filled trench. The first section considered only the presence of clayey fills next to the slurry wall, whereas the second analysis incorporated the existence of relatively loose granular materials in the trench next to the slurry wall, as well as medium consistency CH/CL clayey fills. In this way, the impact of the loose granular fills on the deformations of the dam and upon the slurry wall were evaluated. In both sections, the incorporation of compressible material above the top head of the slurry wall was also considered.

For the seismic analysis, the same sections adopted for the static case were considered. The behavior of the dam was analyzed for two types of earthquakes: "operation" and "maximum credible" earthquakes, the first with a 10% probability of being exceeded in 50 years, and the second one associated to a 10% probability of being exceeded in 100 years. This last one was comparable to the one obtained using deterministic methods. The determination of these earthquakes was done using probabilistic methods, adopting the attenuation law proposed by Martin (1990) and quoted by Ridell et al. (2002). The elastic spectrum was defined using the Newmark and Hall (1982) methodology quoted also by Ridell et al. (2002), specifically adapted to Chilean conditions based on a detailed statistical study of the ground response to registered national earthquakes, especially those obtained during the March 3rd, 1985 earthquake, that impacted Chile's central regions (Ridell, 1995). In this last study, the effect of local geotechnical conditions on the amplification properties of the ground response were specifically considered, by classifying the location sites of the accelerograph stations (Ridell et al, 1992) in accordance with the geotechnical classification defined in the Chilean Code. For the case of Convento Viejo, a design spectrum in hard soil or Type I soil (Chilean norm NCh 433) was generated. The response of elastic structures, in turn, was considered with a degree of damping of 5%. The seismic risk study (Ridell et al., 2001) generated two types of artificial records: (i) records compatible with the design spectrum and (ii) records compatible with the power spectral density. From these record sets, two were selected for the design, one associated to the "operation" earthquake and the other to the "maximum credible" earthquake. Their main characteristics are indicated in Table 2.

Table 2: Design earthquakes main characteristics

Characteristics	Unit	Earthquake	
		Operation	Maximum credible
Duration	Seconds	65	75
Probability of being exceeded	-	10% in 50 years	10% in 100 years
Peak Ground Acceleration (PGA)	cm/s^2	392.4 (0.4g)	431.8 (0.44g)
Frequency (main range)	Hz	2.5 – 7.5	2.5 – 7.5

Both, the "operation" earthquake and the "maximum credible" earthquake analysis were carried out for longest periods of time than the duration of the considered earthquakes in order to ensure a "stable" situation for the residual deformations. The most relevant results of each analysis are presented in Table 3, in which profile 1 represents a cross-section of the left part of the dam and profile 2, a cross-section of the right part of the dam, where the non-compacted granular materials that partially fill the cut-off trench are located. Figure 6 presents some examples of the results obtained from these analyses.

Table 3: Summary of estimated seismic deformations.

VARIABLE	OPERATION EARTHQUAKE		MAXIMUM CREDIBLE EARTHQUAKE	
	Profile 1	Profile 2	Profile 1	Profile 2
DHM/DHMR at center of crest (m)	0.19/0.09	0.17/0.06	0.23/0.16	0.20/0.13
DHM/DHMR at edges of crest (m)	1.30/1.30	1.40/1.40	1.20/1.20	1.30/1.30
DVM/DVMR at center of crest (m)	0.23/0.21	0.21/0.19	0.24/0.22	0.22/0.20
DVM/DVMR at edges of crest (m)	1.28/1.28	1.27/1.27	1.20/1.20	1.20/1.20
AMH at crest (g)	0.40	0.50	0.60	0.46
DHM: Maximum Horizontal Deformation, DHMR: Maximum Residual Horizontal Deformation				
DVM: Maximum Vertical Deformation, DVMR: Maximum Residual Vertical Deformation				
AMH: Maximum Horizontal Acceleration				

a) b) c) d)

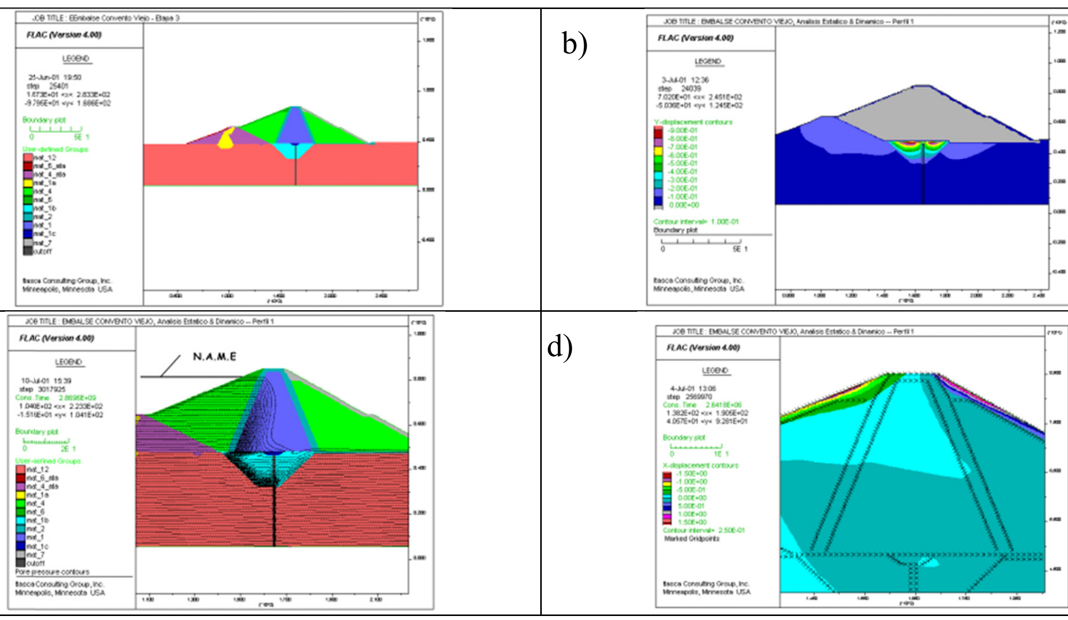


Figure 6: a) analysis model, b) settlements in trench at end of construction, c) estimation of static water-table levels over time, d) residual horizontal deformations (ARCADIS Geotecnica, 2002)

THE FEBRUARY 27TH EARTHQUAKE

The epicenter of this earthquake (EQ) was located in the ocean in front of the town of Cobquecura at a depth of 47.4 km below the earth crust. Figure 7 shows the locations of the epicenter and of the Convento Viejo Dam. Figure 8 shows the estimated Mercalli intensities in the central and south regions of the country (Astroza et al., 2010). The long duration of the earthquake, the high content of long period waves and maximum vertical accelerations similar to the maximum horizontal accelerations (Table 4) are the main earthquake's characteristics that explain many of the observed damages. Another important characteristic of this earthquake was the high magnitude of the numerous aftershocks that occurred in the days that followed the main earthquake. In fact, until May 3rd some 17 shocks with magnitudes over 6.0 M_w occurred. Special mention has to be given to the particularly strong earthquake of March 11th, initially considered as an aftershock but afterwards recognized as a new earthquake with an epicenter 9 km inland and some 150 km North of the February 27th epicenter. This earthquake had a magnitude of M_w=6.9 and after a few minutes was followed by two aftershocks of M_w=6.7 and M_w=6.0.

The earth dam of Convento Viejo is instrumented with three triaxial accelerographs identified as A1 (located at the crest), A2 (located at the foot of the dam, in alluvial soil) and A3 (located inside the tunnel, founded in rock). The earthquake of magnitude M_w=8.8 was recorded by all three instruments, and some values are presented in Table 5, which also includes the main aftershocks. The duration of strong movements was 90 seconds (over 0.05 g) in the main earthquake. As a way to exemplify the event, Figure 9 presents the accelerograms recorded for the main earthquake in its different components (accelerograph A2). Figure 10 indicates the frequency contents together with the spectrums, and a comparison with the proposal made by the project's seismic risk study.

Table 4: Registered peak ground accelerations (PGA) during the February 27 earthquake

Site	Barrientos, S. (2010)		
	PGA (N-S)	PGA (E-W)	Vert. PGA
Colegio San Pedro, Concepción	0.65g	0.61g	0.58g
Cerro Calán, Santiago	0.20g	0.23g	0.11g
Campus Antumapu, Santiago	0.23g	0.27g	0.17g
Cerro El Roble	0.19g	0.13g	0.11g
Melipilla, RM	0.57g	0.78g	0.39g
Olmué , V Región	0.35g	0.25g	0.15g
Casablanca, V Región	0.29g	0.33g	0.23g
San José de Maipo, RM	0.47g	0.48g	0.24g
Colegio Las Américas	0.31g	0.23g	0.16g
Cerro Santa Lucia, Santiago	0.24g	0.34g	0.24g

Site	U. de Chile (2010)	
	PGA	Vert. PGA
Depto. Ing. Civil, RM	0,17g	0,14g
Est. Metro Mirador, RM	0,24g	0,13g
CRS Maipú, RM	0,56g	0,24g
Hospital Tisné, RM.	0,30g	0,28g
Hospital Sotero del Río, RM.	0,27g	0,13g
Hospital de Curicó, VII región	0,47g	0,20g
Hospital de Valdivia, XV Región	0,14g	0,05g
Viña del Mar (M.Marga), V Región	0,35g	0,26g
Viña del Mar (Centro), V Región	0,33g	0,19g
RM: Región Metropolitana		

Table 5: Recordings February 27th earthquake and main aftershocks.

Date	Hour (local time)	Mw	Lat.	Lon.	H (km)	Loc.	PGA E-W	PGA N-S	Vert. PGA
27 Feb.	3:34:08	8.8	-36.29	-73.24	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	11:39:41	6.9	-34.30	-72.13	33.1	A1	0.14	0.18	0.13
						A2	0.12	0.15	0.07
						A3	0.06	0.08	0.06

Hour = Time that the earthquake began
Mw= Magnitude according to U. de Chile
Lat, Lon, H = Coordinates of the earthquake's epicenter
Loc = Location site of the accelerograph

PGA E-W = Max. acceleration of the E-W component
PGA N-S = Max. acceleration of the N-S component
Vert. PGA = Max. vertical acceleration in g's

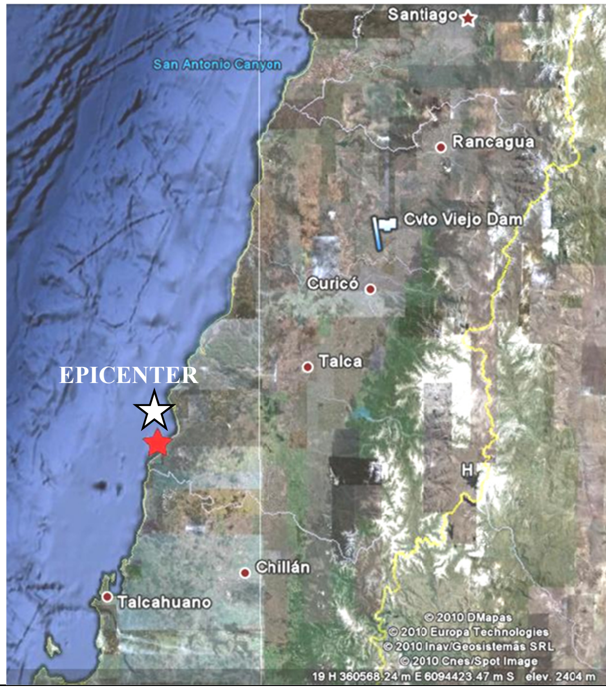


Figure 7: Location of the epicenter and Convento Viejo Dam.

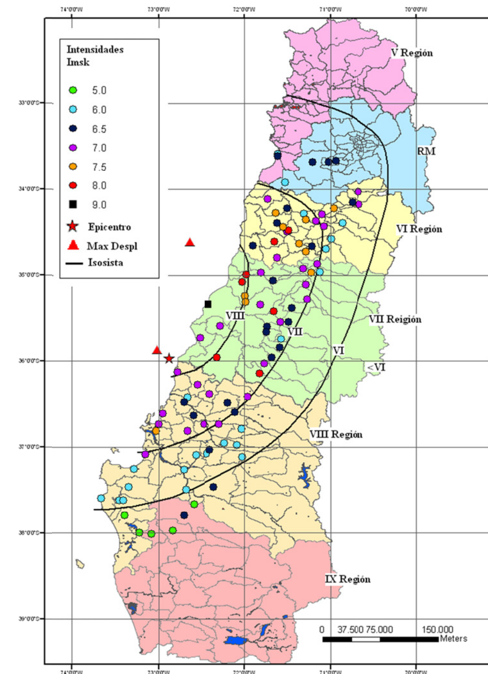


Figure 8: Intensities in the areas affected by the February 2010 earthquake (Astroza et al., 2010).

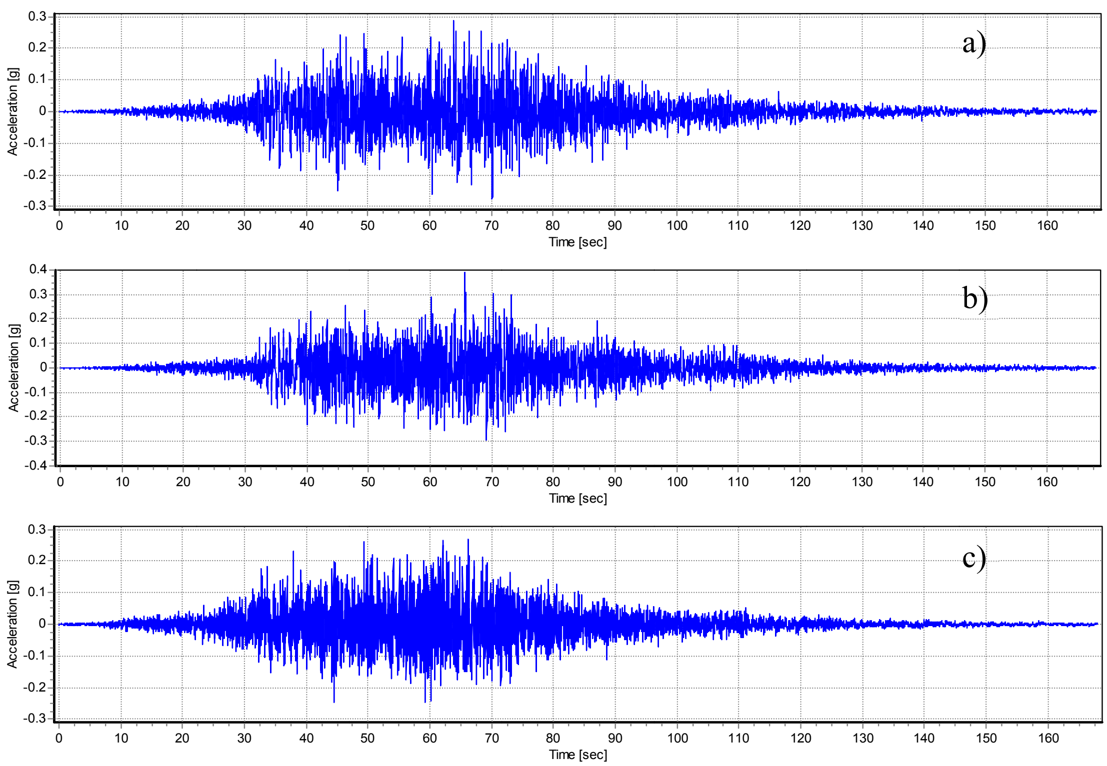


Figure 9: Accelerograms of the main earthquake, recorded by accelerograph A2 (ARCADIS Geotecnica, 2010): a) E-W, b) N-S, c) Vertical.

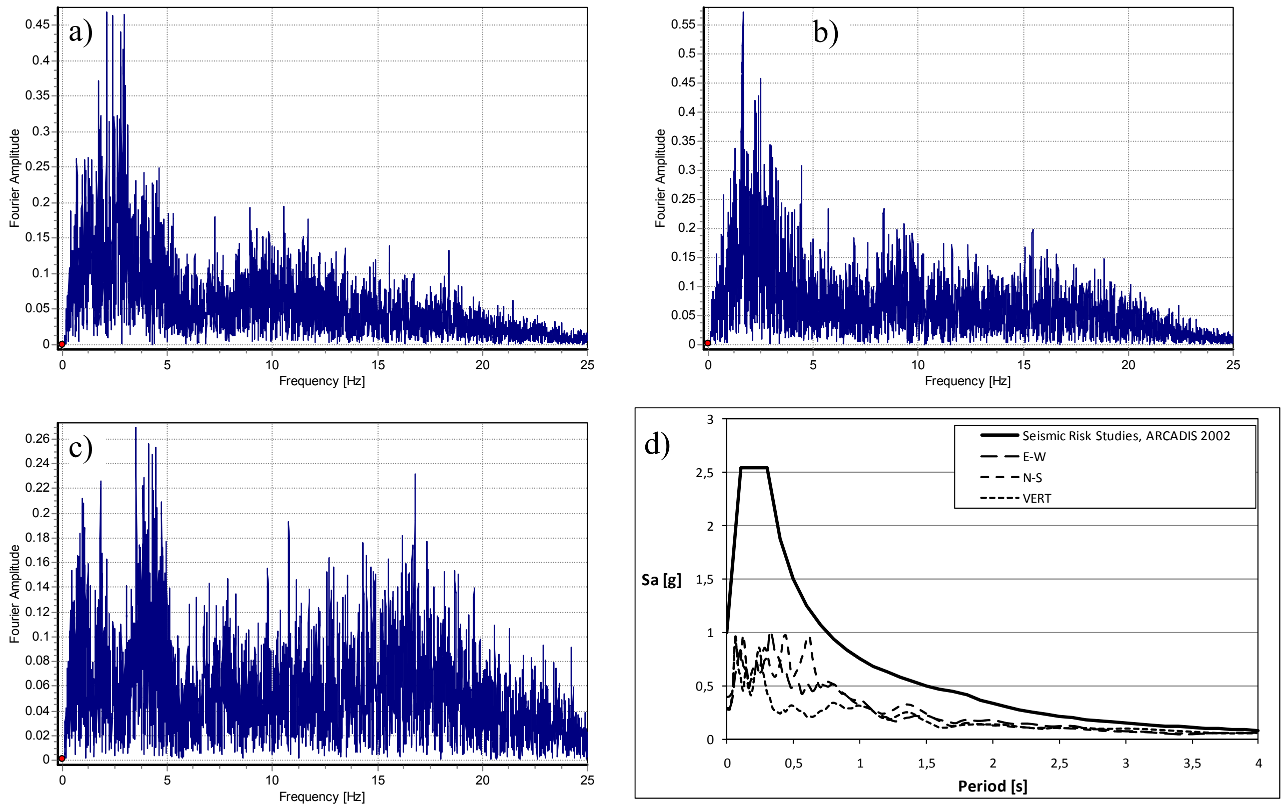


Figure 10: 27 February Fourier spectrums recorded in accelerograph A2 (ARCADIS Geotecnica, 2010): a) E-W, b) N-S, c) Vertical, d) elastic spectrums.

BEHAVIOR OF THE EARTH DAM DURING FEBRUARY 27TH EARTHQUAKE

General behavior of the dam

In general, the behavior of the earth dam was quite satisfactory; there were no external signs that accused significant problems. Superficial cracks were detected at the dam crest in the area where greater settlements were to be expected, according to the dynamic analysis carried out during the design stage. Nevertheless, two of the Casagrande-type piezometers presented a behavior that required a detailed analysis as well as a continuous monitoring of their evolution for at least a month.

After the earthquake, practically all the Casagrande-type piezometers showed an increase in their piezometric levels, something that is to be expected following an event of the magnitude and intensity as the one that took place on February 27th. Two of them kept the increased levels for a longer period and this situation is discussed later on. One positive aspect to be mentioned is that 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. Also no seepage flow or filtrations were detected in the channel destinated to collect water from the dam's drainage system.

The behavior of the concrete dam was also very satisfactory. No external signs were identified of any unsatisfactory response of the dam to the strong earthquake.

Registered deformations

The frequent inspection of 12 concrete monoliths located at the crest of the dam and at the downstream slope allowed to monitor the deformations produced at the dam under normal and seismic conditions. In addition, it was also possible to monitor the deformations of piezometers and other fixed points located at the crest. The dynamic analyses (ARCADIS Geotecnica, 2002) predicted maximum vertical deformations at the crest of 230 mm and of 1,280 mm at the edge of the slope. The maximum vertical deformations recorded at the crest, after the main earthquake, were lower than 350 mm.


Figure 11: Cracks observed at the crest of the earth dam: a) downstream slope, b) upstream slope.

Behavior of piezometric levels

The dam is equipped with piezometers of the Casagrande type and also electric ones. In total there are 14 Casagrande-type piezometers, 7 installed in the impervious clay core (elevation of the bottom 247m a.s.l.) and 7 in the foundation of the dam, beneath the impervious trench and downstream of the slurry wall (elevation of the bottom 220m a.s.l.). Complementing the Casagrande-type piezometers, there are 15 electric piezometers, distributed between the impervious clay core, the downstream filter and the foundation soil, beneath the downstream shoulder.

As it was previously mentioned, after the earthquake, practically all of the Casagrande- type piezometers presented an increase in their piezometric levels, but three days after the main event the piezometric level of all the piezometers descended approximately to the levels recorded before the earthquake. The only exceptions were piezometers P-7 and P-9, where the water level continued to rise. The area where piezometers P-7 and P-9 are located corresponds to the central part of the dam, the same area where the most important superficial cracks were detected, both in terms of their length and width, and also concordant with the most significant depression of the crest. This area coincides approximately with the limit of the emergency fill of the cut off trench with sandy gravels deposited under water, before 1981 when the construction of the dam was suspended.

Also, the electric piezometers showed an important increase in the readings after the earthquake, but around 12:00 am of the same day, these instruments began to record a decrease in piezometric pressure levels. It should be noted that none of the electric piezometers, located at the filter and at the base of the downstream shoulder (drains), recorded any increase in their readings as a result of the earthquake or after the several aftershocks.

In the case of Casagrande-type piezometer P-7, where the increase of the ground-water level was of about 2 m, it is noteworthy that the rate of increase was not affected by the aftershocks or by the decrease in the level of the reservoir.

Actually after examining the situation of these two piezometers P-7 and P-9, a few days after the earthquake, it was decided to start lowering the water level in the reservoir anticipating in a few weeks the lowering of the reservoir that had to be done anyway. Indeed, the commitment acquired in the environmental permits was to empty the reservoir after one year of operation to check their general conditions.

On March 11th, an important increase in the piezometric level was once again recorded, which continued until March 15th. It is important to point out that during this same period, more than 50 aftershocks took place in the nearby Pichilemu area, with magnitudes Mw>4 and a maximum of Mw=6.9. Between March 15th and 19th, the piezometric level at P-9 was stabilized at a constant level, and then began, for a few days, to decrease the level at a rate greater than the descent of the dam's water level. Afterwards, the level of this piezometer decreased at a rate equal to that of the level water of dam. The latter indicates that piezometer P-9 was working correctly given that its oscillations were associated to changes in the border conditions, such as cyclic loads (earthquakes) and the water level in the reservoir. Piezometer P-7, on the other hand, did not present variations during this period, which makes it possible to presume that its operation was anomalous (probably plugged). Figure 12 presents the variations in levels recorded by piezometers P-7 and P-9, up to the end of March 2010.

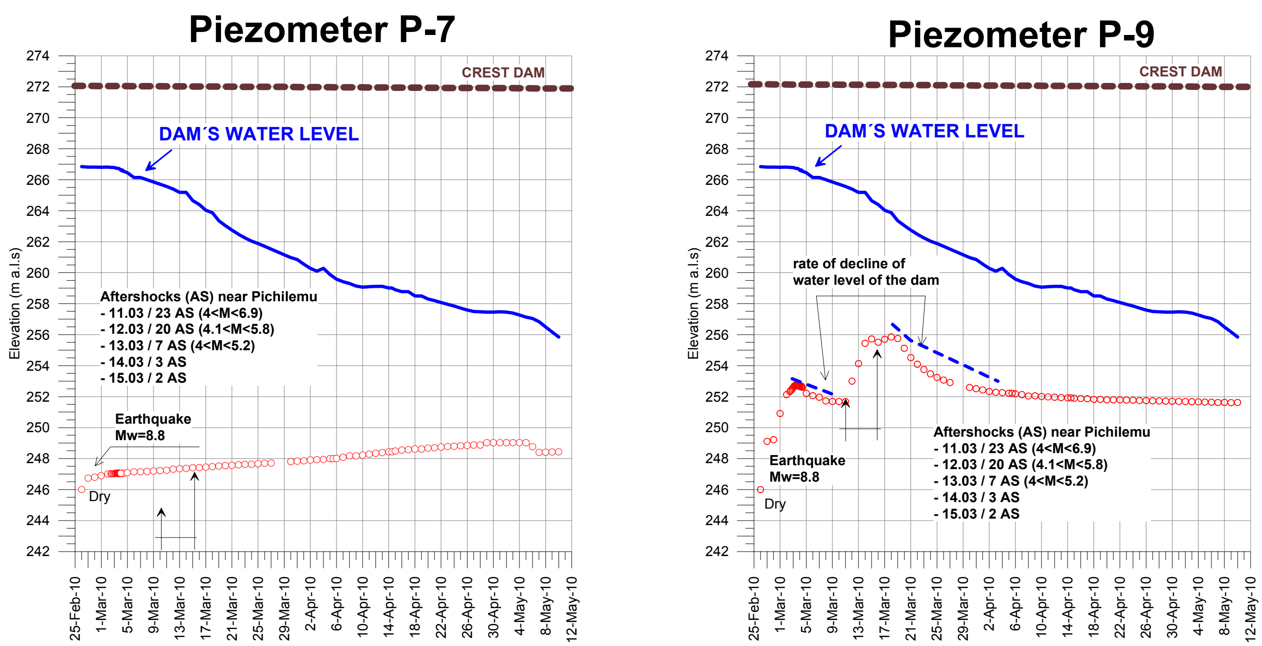


Figure 12: Variations in piezometric levels recorded after the main earthquake (Valenzuela, 2010)

Recorded accelerations

The main parameters of the design earthquake and the ones recorded at the foot of the dam (accelerograph A2) during the earthquake are compared in Table 6. The seismic register in accelerograph A2 could be comparable to the earthquake identified in the seismic risk study like an earthquake in "hard soil" and with "maximum free field acceleration". It is important to note that the accelerometer A2 is located 75 m downstream of the footprint of the earth dam and the register of maximum free field acceleration could be influenced by the earth dam, though unlikely.

During the February 27th earthquake, at the foot of the dam the free field acceleration recorded was 0.49 g (resultant from the E-W and N-S components). This value is 11% higher than the maximum credible earthquake adopted in the seismic risk study. On the other hand, the maximum destructive potential, which is defined as the Arias intensity (I_A) divided by the square of the number of crossings by zero, gave a result equal to $47x10^{-4}$ ($g{\cdot}s^3$) for the design earthquakes. The February 27th earthquake presented a destructive potential equal to $63x10^{-4}$ ($g{\cdot}s^3$), which means a value 34% higher. In relation to the seismic amplification between the foot and the crest of the dam, the dynamic analyses estimated a value equal to 25%, whereas for the actual earthquake, the corresponding amplification was 46%.

Table 6: Comparison of characteristic parameters between the design earthquake and data recorded on February 27th.

Parameters		Seismic Risk Study		Dam Foot l – Accelerograph A2		
		Design EQ	Maximum Credible EQ	E-W	N-S	Vertical
Maximum acceleration	$a_{máx}$ [g]	0,40	0,44	0,29	0,39	0,28
Maximum velocity	$v_{máx}$ [cm/s]	27,2	22,6	24,6	38,4	22,6
Maximum displacement	$D_{máx}$ [cm]	4,5	4,7	9,0	10,0	7,9
Arias intensity	I_A [m/s]	8,999	10,5	5,053	6,466	3,680
Predominant period 1	Tp1 [s]	0,27	0,27	0,34	0,61	0,28
Destructive Potential	PD [10^{-4} g s3]	45,9	46,7	63		10,2

where: $I_A = \dfrac{\pi}{2g}\int_0^{t_0}[a(t)]^2\,dt$ $\qquad P_D = \dfrac{I_A}{v_0^2}$

I_A: Arias Intensity $\qquad$ t_0: total duration of ground motion
g: gravity acceleration $\qquad$ a(t): ground acceleration
v_0^2: intensity of zero-crossings per second, obtained from the acceleration record

CONCLUSIONS

The overall behavior of the Convento Viejo earth dam during the February 27th earthquake and the subsequent aftershocks can be defined as follows, based on the response of control instrumentation, seismic records and field inspection:

- The February 27th earthquake was somewhat greater than the design earthquake adopted for the analysis of the dynamic behavior along the length of the dam, both in terms of the maximum acceleration, duration of the strong movement period and destructive potential,
- Notwithstanding these facts, the earth dam presented very satisfactory structural behavior in the face of this strong earthquake and there is no evidence of water filtrations, significant cracks or settlements, and the operation of the works and the facilities located downstream of the dam suffered no risk whatsoever,
- The dynamic analyses carried out in the design stage, by means of finite difference methods and the adoption of the Mohr-Coulomb failure criterion for soils behavior, made possible to produce a model able to adequately predict the overall behavior of the dam under of the impact of a large magnitude earthquake,
- It has become clear the importance to opportunely implement and afterwards to maintain in operation a geotechnical instrumentation system, so that in the face of important seismic events it is possible to carry out inspections, verifications, updates and corrections that are necessary to guarantee the safety of the works.
- Another aspect that is interesting to point out is that during the design stage, complementary stability analyses were carried out using limit equilibrium methods. In these analysis a horizontal seismic coefficient equal to k_H=0.14 was adopted, which gave safety factors of FS > 1.2.

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