

Seismic response of the Convento Viejo earth dam during the February 27TH 2010 Chile earthquake

Respuesta sísmica de la presa de tierra Convento Viejo durante el terremoto de Chile del 27 de febrero 2010

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This article examines the seismic behavior of an earth dam. Accelerations measured during the earthquake at three different dam levels are compared with those estimated from a dynamic analysis performed during the design phase. The article also comments on piezometric levels and settlements induced by the strong earthquake. The Convento Viejo embankment dam is a zoned earth dam, 32 m high, founded in alluvial deposits, whose construction was completed between 2006 and 2008. The site is located near the town of Chimbarongo, 160 km south of Santiago, Chile's capital, about 85 km from the coast and 220 km from the epicenter of the earthquake that took place on February 27th, 2010. A description of the construction and design study as well as observations of the earth dam seismic response is presented. The assessment of the dam performance was based on comparisons of estimated and measured values of maximum acceleration, velocity and displacement in addition to predominant period, Arias Intensity, destructive potential and piezometric monitoring. It is concluded that despite that some of these parameter values were slightly exceeded for this large earthquake, the overall performance of the dam was satisfactory.

Keywords: zoned earth dam, seismic geotechnical behavior, Chilean 27/02/2010 earthquake, monitoring

En este artículo se estudia el comportamiento sísmico de una presa de tierra. Se comparan las aceleraciones medidas durante el sismo en tres posiciones diferentes de la presa con aquellas estimadas de análisis dinámicos efectuadas en la etapa de diseño. El artículo también incluye información sobre niveles piezométricos y asentamientos inducidos por el gran sismo. El cuerpo de la presa Convento Viejo está dividido en varias partes o zonas, tiene 32 m de altura, está fundada sobre depósitos aluviales y su construcción finalizó entre 2006 y 2008. La presa está ubicada próxima a la localidad de Chimbarongo, 160 km al sur de Santiago, capital de Chile, 85 km de la costa y 220 km del epicentro del terremoto que tuvo lugar el 27 de febrero 2010. Se presenta una descripción de la construcción y bases de diseño así como observaciones de la respuesta sísmica de la presa. La evaluación del desempeño se basa en comparaciones entre los valores máximos estimados y medidos de aceleración, velocidad y desplazamiento, además de periodo predominante, Intensidad de Arias, potencial destructivo y monitoreo piezométrico. Se concluye que a pesar de que algunos de los valores de estos parámetros son levemente sobrepasados para este gran terremoto, el desempeño global de la presa fue satisfactorio.

Palabras clave: presa de tierra zonificada, comportamiento geotécnico sísmico, sismo Chileno 27/02/2010, monitoreo

Introduction

The Convento Viejo embankment dam is situated above a stream of the same name. The earth dam is located 160 km south of Santiago nearby the town of Chimbarongo (see Figures 7 and 8). This embankment dam is 32 m high and corresponds to a zoned earth dam with an impervious clay core (Figure 4). It is separated by a small hill of a concrete dam where the spillway and gates are located. The two dams allowed to form a reservoir with a storage capacity of 237 Mm³, a flooded area of 4,500 ha and a network of channels more than 90 km long.

The construction project started in 1973, but work was halted in 1975 due to financial problems. The construction was resumed in the period between 1978 and 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.



Figure 1: Convento Viejo dam operating only with the cofferdam (ICOLD, 1996)



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

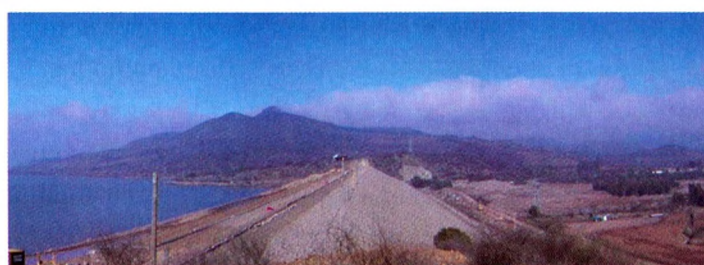


Figure 3: View of the Convento Viejo embankment dam (ARCADIS Geotécnica files, 2007)

Description of the earth dam

The earth dam has an approximate volume of 2 Mm³, 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.

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, 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 7 m to 16 m 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.

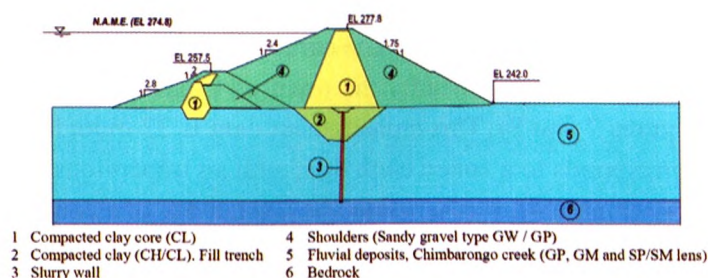


Figure 4: Cross section of the earth dam (ARCADIS Geotécnica, 2002)

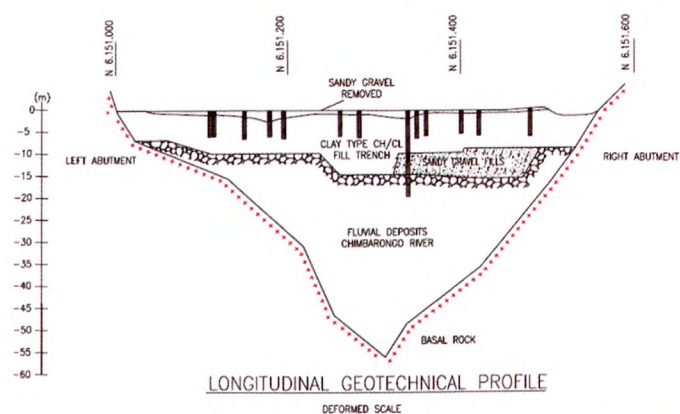


Figure 5: Longitudinal section along the earth dam axis (ARCADIS Geotécnica, 2002)

Design of the 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, which were 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 kN/m ³	$E_{drained}$ t/m ²	ν	c kPa	ϕ (°)	k m/s	n
1 - Core earth dam	17.0	$2600(\sigma'_1)^{0.16}$	0.3	5	30	10^{-7}	0.347
1 - Core cofferdam	16.4	$2400(\sigma'_1)^{0.16}$	0.3	3	28	10^{-7}	0.370
2 - Clay trench	14.2	$425(\sigma'_1)^{0.51}$	0.3	20	15	10^{-8}	0.453
3 - Slurry wall	-	40000	-	500	36	10^{-8}	-
4 - Shoulders	22.2	$6000(\sigma'_1)^{0.5}$	0.3	0	40	10^{-4}	0.160
5 - Fluvial deposits foundation	21.6	$6000(\sigma'_1)^{0.508}$	0.25	25	45	10^{-4}	0.185
5a - Sandy gravel fill (trench)	18.6	$3000(\sigma'_1)^{0.508}$	0.35	0	45	10^{-4}	0.296

γ_d : Dry unit weight
 $E_{drained}$: Drained deformation modulus
 σ'_1 : Confining pressure in t/m²
 ν : Poisson coefficient

c : Cohesion
 ϕ : Angle of internal friction
 k : Coefficient of Permeability
 n : Porosity

Dynamic analyses

The study of 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 dam deformations and upon the slurry wall was 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 performed using probabilistic methods, adopting the attenuation law proposed by Martin (1990). The elastic spectrum was defined using the Newmark and Hall (1982) methodology, 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 433Of96) was generated. The response of elastic structures, in turn, was considered with a degree of damping of 5%. The seismic risk study generated two types of artificial records: (i) records compatible with the design spectrum and (ii) records compatible with the power spectral density (Ridell and de la Llera., 2001). 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: Main characteristics of the design earthquakes

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 longer 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 displacement

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 Displacement DHMR: Maximum Residual Horizontal Displacement				
DVM: Maximum Vertical Displacement DVMR: Maximum Residual Vertical Displacement				
AMH: Maximum Horizontal Acceleration				

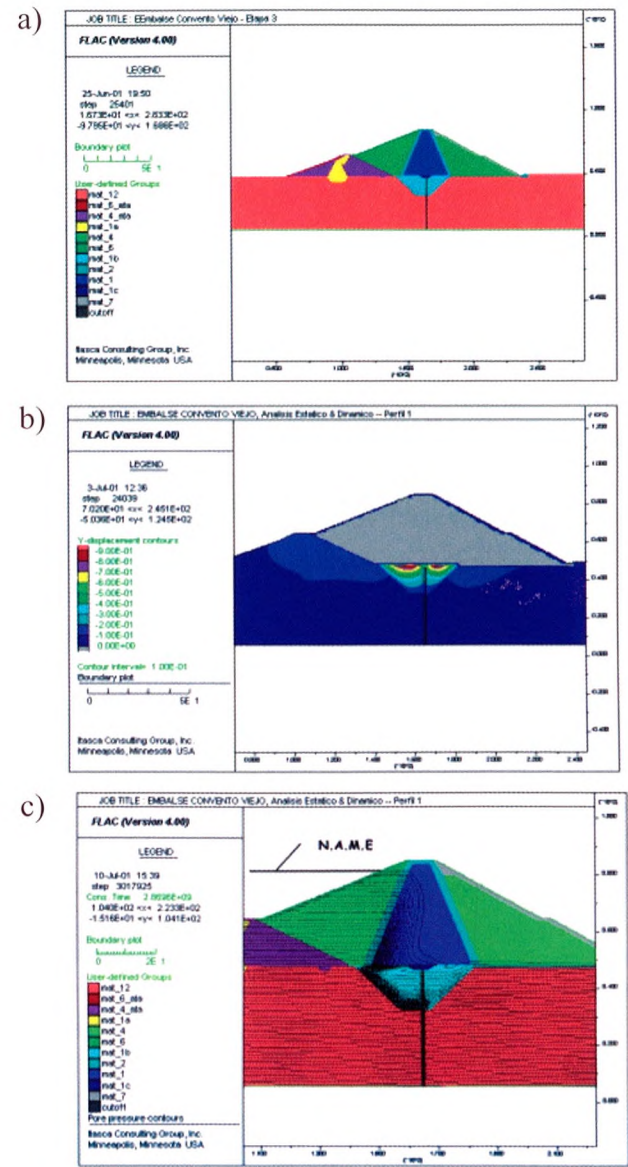


Figure 6: a) Analysis model, b) settlements in trench at end of construction and c) estimation of static watertable levels over time

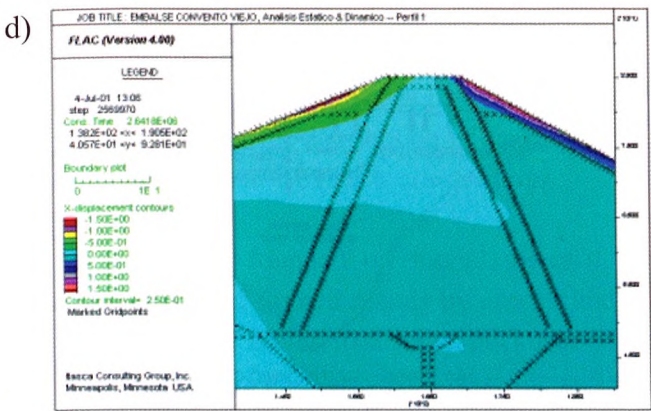


Figure 6: d) residual horizontal deformations (ARCADIS Geotécnica, 2002)

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 may 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 Richter 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 Richter of 6.9 and after a few minutes was followed by two aftershocks of 6.7 and 6.0.

The Convento Viejo earth dam 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 main earthquake of moment 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 spectra, 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 (2010ab)		
	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, PGA in g

Date	Hour (local time)	M _w	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 when the earthquake began

M_w: moment magnitude according to U. de Chile

Lat, Lon, H: coordinates of the earthquake's hypocenter

Loc: location site of the accelerograph

PGA E-W: max. acceleration E-W component

PGA N-S: max. acceleration N-S component

Vert. PGA: max. vertical acceleration



Figure 7: Location of the epicenter and Convento Viejo Dam

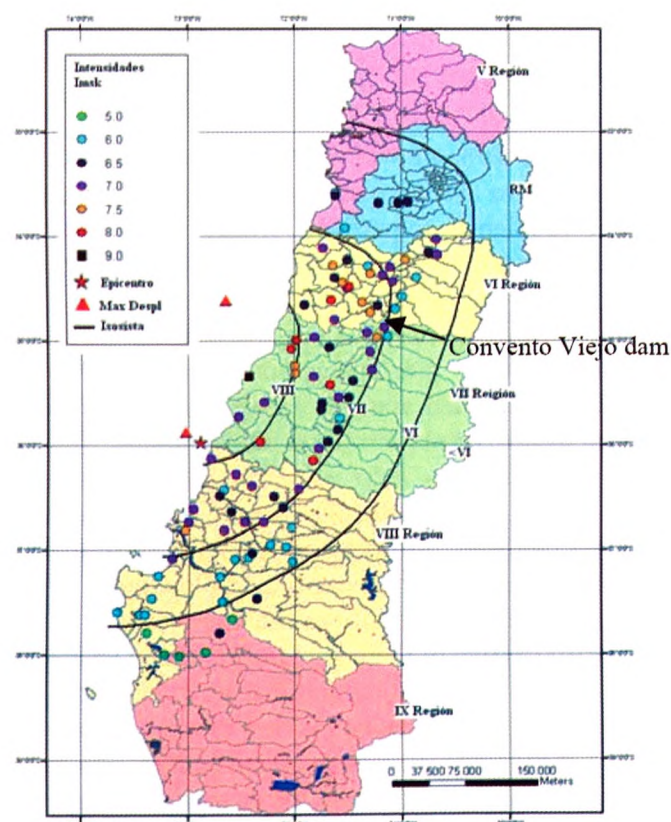


Figure 8: Intensities in the areas affected by the February 2010 earthquake (Astroza et al., 2010)

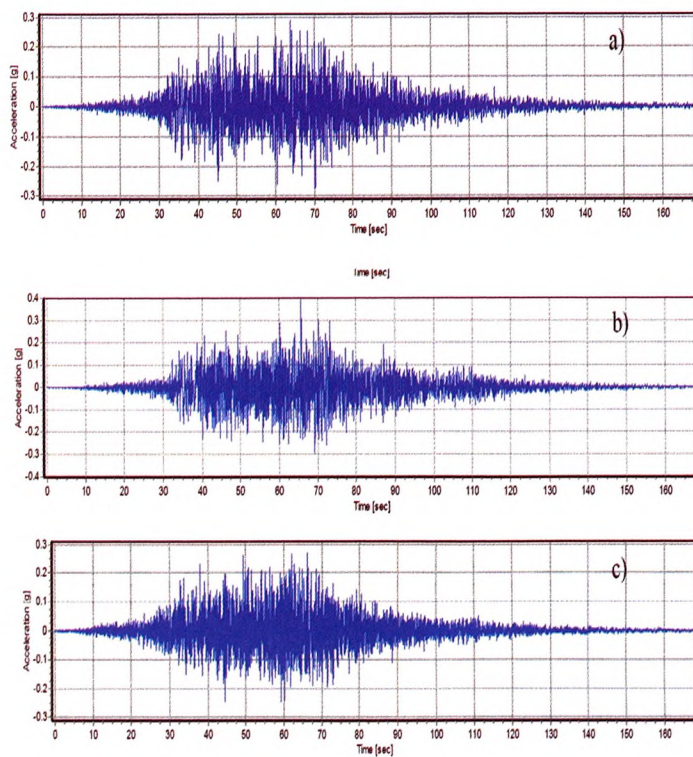


Figure 9: Accelerograms of the main earthquake recorded by accelerograph A2 : a) E-W, b) N-S, c) Vertical (ARCADIS Geotécnica, 2010)

Seismic behavior of the earth 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 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 due to the strong earthquake.

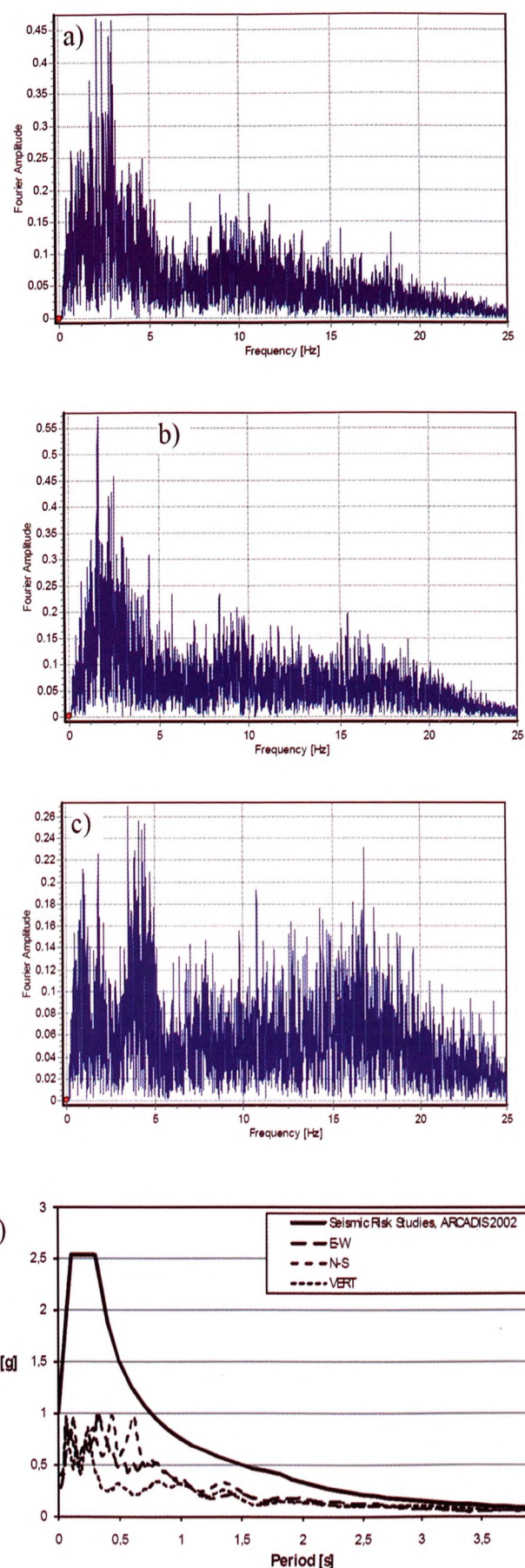


Figure 10: 27 February Fourier spectra obtained from accelerograph A2 : a) EW, b) N-S, c) Vertical and d) elastic spectra (ARCADIS Geotécnica, 2010)

Registered displacements

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

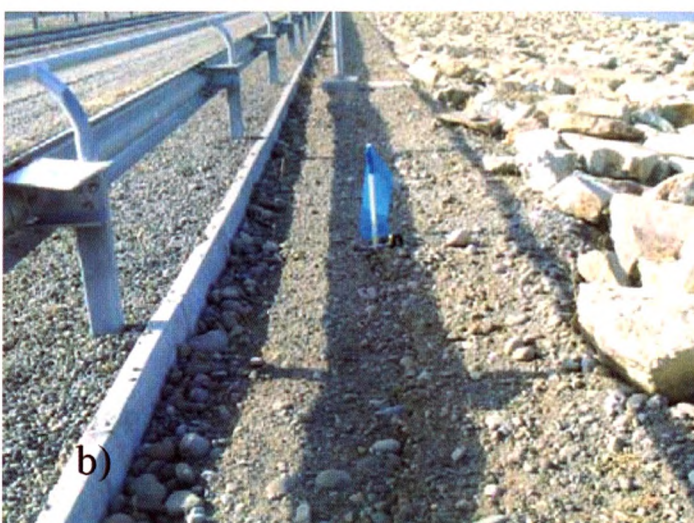


Figure 11: Cracks observed at the crest of the earth dam: a) downstream slope, b) upstream slope.

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 247 m 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 220 m 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. Piezometers P-7 and P-9 are located in 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, the ground-water level increase was 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 of Pichilemu area, with magnitudes Richter higher than 4 and a maximum magnitude of 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 dam water level. The latter indicates that piezometer P-9 was working correctly given that its oscillations were associated to changes in the border conditions, such as seismic cyclic loads 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.

Recorded accelerations

The main parameters of the design earthquake and the parameter values recorded at the foot of the dam (accelerograph A2) during the earthquake are compared in Table 6. The earthquake identified in the seismic risk study as an earthquake in “hard soil” and with “maximum free field acceleration” can be compared with the seismic register in accelerograph A2. It is important to note that the accelerometer A2 is located 75 m downstream of the earth dam footprint and therefore, the register of maximum free field acceleration is less likely to be influenced by the earth dam.

During the February 27th earthquake, at the foot of the dam the maximum free field acceleration recorded was 0.49g, resultant from the E-W and N-S components. This value is 11% higher than the maximum acceleration for the 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 $47 \cdot 10^{-4} \text{ gs}^3$ for the design earthquakes. The February

27th earthquake presented a destructive potential equal to $63 \cdot 10^{-4} \text{ gs}^3$, which means a value 34% higher than

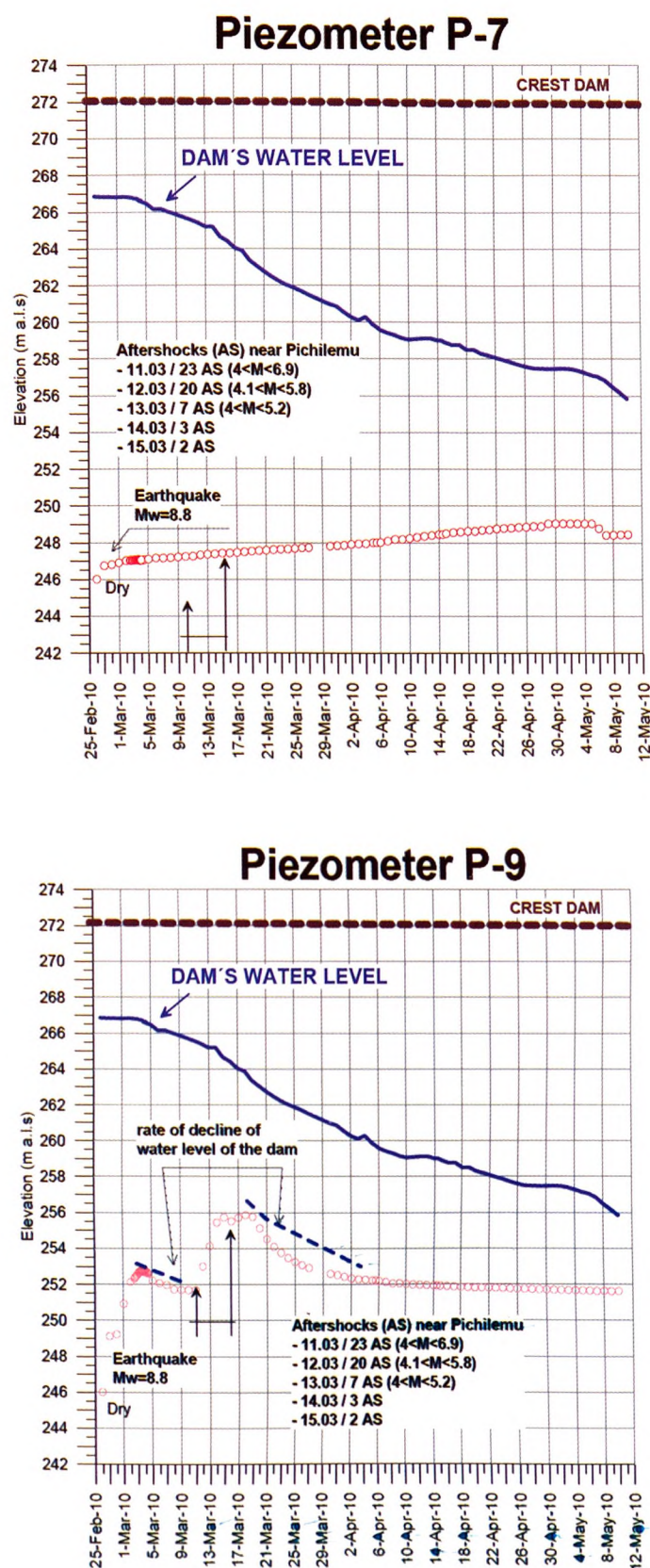


Figure 12: Recorded variations in piezometric levels (Valenzuela, 2010)



the estimated. 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%.

The Arias Intensity I_A is defined by:

$$I_A = \frac{\pi}{2g} \int_0^{t_0} [a(t)]^2 dt \quad (1)$$

where t_0 is the total duration of the ground motion, g is the acceleration of gravity and $a(t)$ is the ground acceleration. The Destructive Potential P_D is defined by:

$$P_D = \frac{I_A}{v_0^2} \quad (2)$$

where v_0 is the intensity of zero-crossings per second obtained from the acceleration record.

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

Parameters		Seismic Risk Study		Dam Foot 1 - Accelerograph A2		
		Design EQ	Maximum credible EQ	E-W	N-S	Vertical
Maximum acceleration	a_{max}, g	0,40	0,44	0,29	0,39	0,28
Maximum velocity	$v_{max}, cm/s$	27,2	22,6	24,6	38,4	22,6
Maximum displacement	D_{max}, cm	4,5	4,7	9,0	10,0	7,9
Arias intensity	$I_A, m/s$	9,0	10,5	5,05	6,47	3,68
Predominant period 1	T_{p1}, s	0,27	0,27	0,34	0,61	0,28
Destructive potential	$P_D, 10^{-4} gs^{-3}$	45,9	46,7	63		10,2

Conclusions

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

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 soil behavior, made possible to produce a model able to adequately predict the overall behavior of the dam under 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 obtain feedback from 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 analyses a horizontal seismic coefficient equal to $k_H = 0.14$ was adopted, which gave safety factors of $FS > 1.2$.

Acknowledgements

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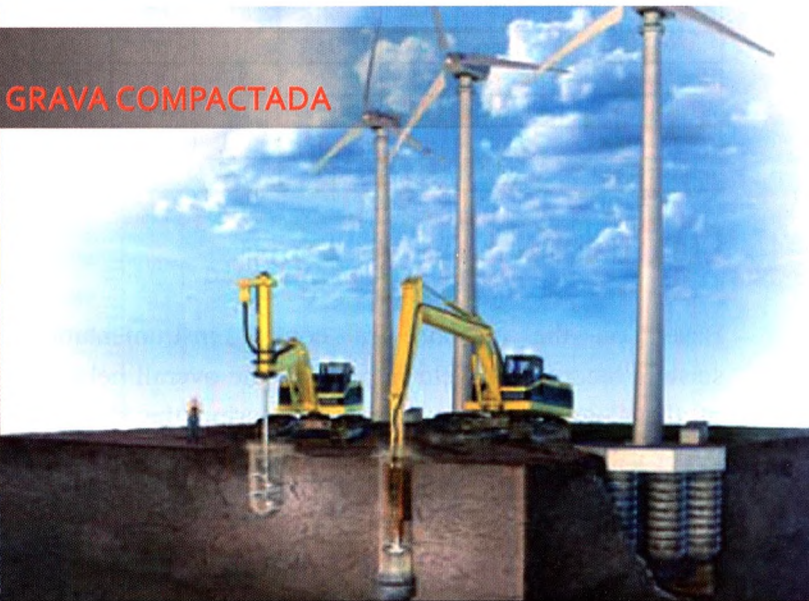
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