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Observed seismic behavior of three Chilean large dams

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ABSTRACT: Seismic shaking of large dams constructed with coarse material, such as rockfill, cobbles and gravels, is normally associated with settlements that, depending on both shaking intensity and degree of compaction of the coarse material, can be significant. The observed seismic response of Cogoti and Santa Juana dams, with heights of 83 and 113 m, respectively, is presented. In the case of Cogoti Dam, an accumulated seismic settlement of 1.2 m has been measured, which induced damage in part of the concrete face, without involving any mechanical instability. On the other hand, Aromos Dam, a zoned dam with a height of 42 m, supposed to be founded on liquefiable ground according to SPT data, underwent seismic settlements smaller than 10 cm during the 1985 Chilean earthquake of Magnitude 7.8. The post seismic analysis suggested that liquefaction was actually restricted because the liquefiable soils were confined by rather dense sandy materials.

1 INTRODUCTION

Chile is located on the southwestern portion of the American Continent, and a significant part of its territory is controlled by a subductive seismic environment associated with the collision between Nazca and South America plates. The convergence rate among these plates is estimated to be in the range of 65 to 90 mm/year. As a result of this tectonic interaction, most of Chilean territory have a high rate of seismicity that includes the largest ever recorded ground motion; the 1960 Valdivia earthquake with an estimated Magnitude 9.5 (Madariaga, 1998).

On the other hand, an important number of large dams are located along the Los Andes Range taking advantage of both available natural hydraulic resources and elevation. Moreover, there also exist some dams located in different valleys. Considering the high seismic activity that normally takes place in Chile, all these earth structures are supposed to be designed to withstand strong earthquakes. In this context, the seismic response observed at three different Chilean high dams, Cogoti, Santa Juana and Aromos, is presented.

2 COGOTI DAM

2.1 *Technical characteristics of the dam*

Cogoti Dam is located in the confluence of Pama and Cogoti rivers, in Limari Province of Chile, about 65 km South of Ovalle city (see Fig. 1), at an elevation close to 575 m above sea level and. It retains a reservoir capacity of 150 millions m^3 when the water is at the spillway crest elevation (654.8 m). In the left abutment, a lateral spillway excavated in rock and without gate allows a water flow of 5000 m^3/s.

Cogoti Dam is a concrete face rock fill dam (CFRD) completed in 1940, although the main body was finished earlier in 1938. The 160 m long embankment, with a maximum height of 82.7 m and a crest width of 8 m, was constructed by placing blasted rock in the site without compaction. The upstream slope has a 1.42–1.47: 1 (H:V) inclination and the downstream slope has 1.47–1.50:1 (H:V). A cross-section through the embankment and the plan view are shown in Figs. 2 and 3, respectively.

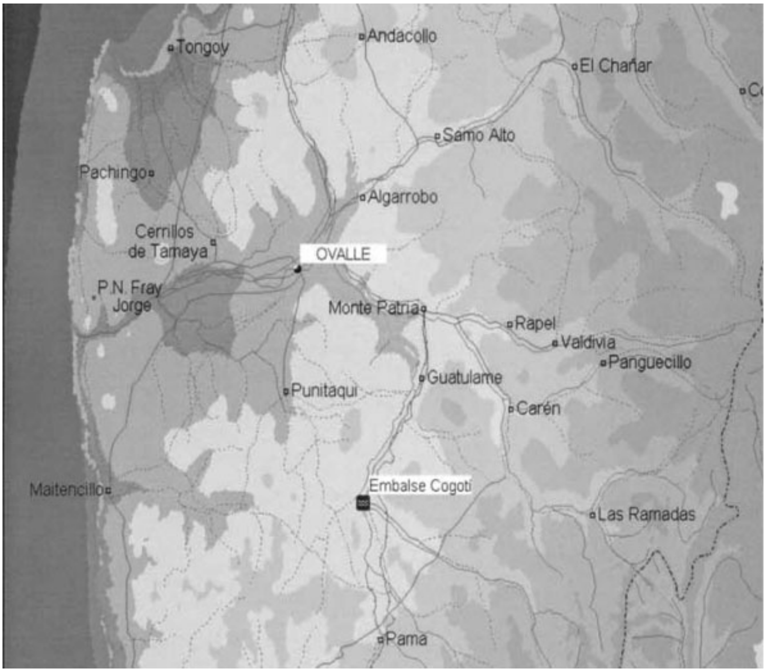

Figure 1. General location of Cogoti Dam.

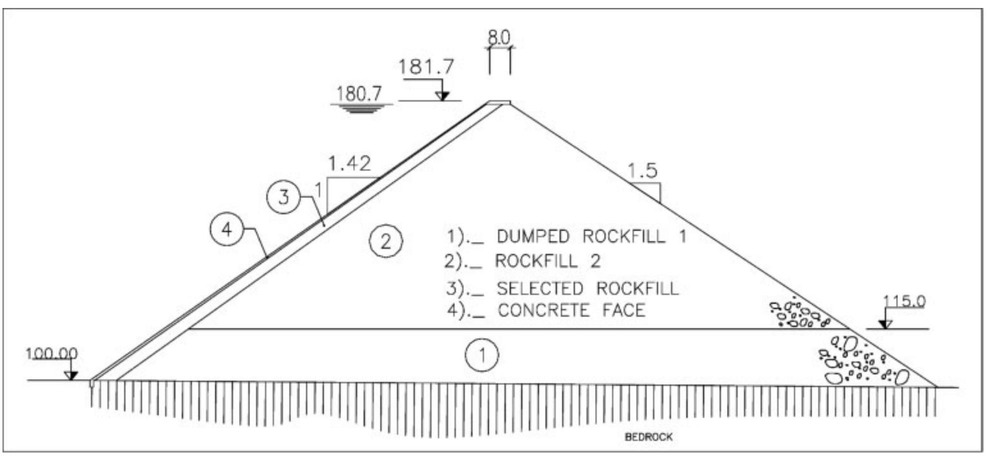

Figure 2. Cross-section of Cogoti Dam.

In the first 15 m of the dam height rock particles with a maximum size of 1.5 meters were used, which were just dumped by gravity at the dam site. Then, the same material limited to a maximum size of 1.3 meters was placed by mechanical means, which induced a slight compaction generated by the passage of trucks during the construction. The technical specifications required that each layer of rock-fill be washed thoroughly with water pressure (water head of 60 m) using a quantity equivalent to three times the volume of the sluiced layer. However, the available technical records of construction indicate that most of the time the required washing was not fully satisfied. Therefore, it is possible to conclude that the body of Cogoti dam is associated with a rock-fill material which is under a poor state of densification.

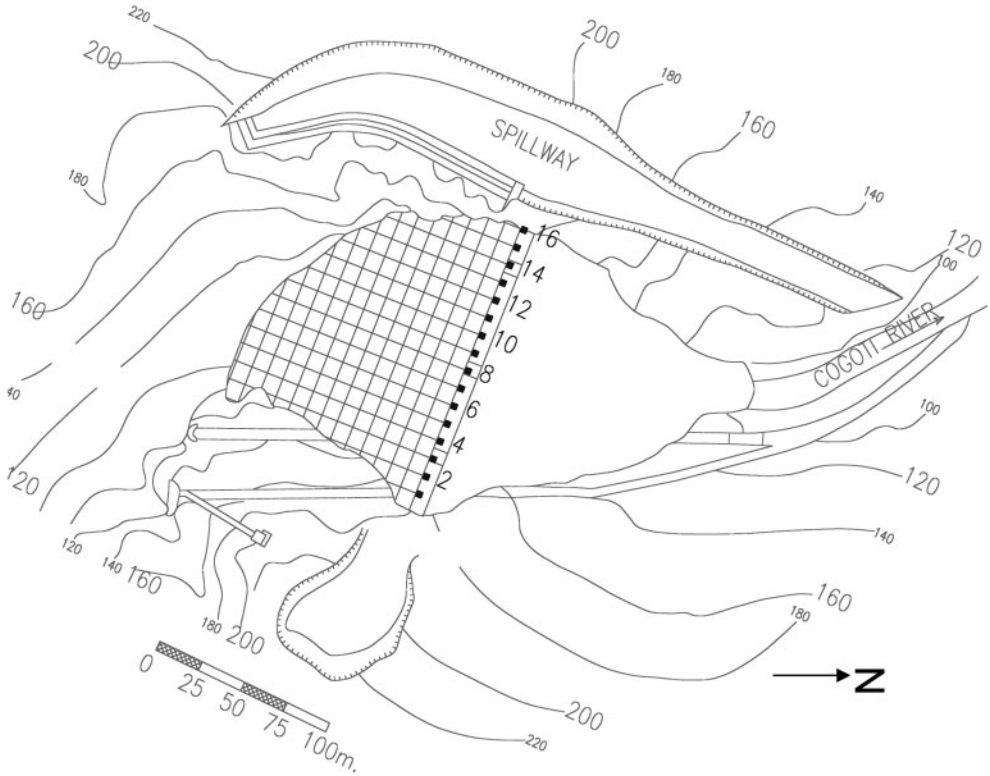

Figure 3. Plan view of Cogotí Dam.

The cross-section of the gorge is shown in Fig.4, where it can be seen that the profile is a rather unsymmetric, with the right abutment less steep and with a change in its inclination. It will be shown later that a concentration of the crest settlements occurred above this point of slope change. The face slab is made of a series of square concrete plates of 10 m × 10 m, which have a thickness varying from 80 cm at the base to 20 cm 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 and consisting of hand-placed small rock particles. Just beneath the vertical and horizontal joints a continuous ditch was constructed to provide a better support to the joints. This ditch system is approximately 0.7 m deep and 1.2 m wide. The joints between concrete plates were sealed by means of copper waterstops of 1.5 mm thickness and 60 cm wide. Additionally, the openings between plates were filled with asphalt. The distribution of the individual concrete plates is shown in Fig. 3.

The base of foundation has a reinforced concrete cut-off wall that was initially designed with a depth of 3 m, but was finally built with a maximum depth of 20 m and an average depth of 7 m. On the other hand, a concrete wall with a variable depth in the range of 1.4 to 2 m is basically the plinth along the abutments.

Regarding the geology of the site, the embankment was placed in a deep and narrow gorge eroded by Cogoti River, which consist mainly of andesitic rocks. Three important structural alignments have been recognized at the dam site: $N45^0 - 75°E$, E-W and N-S. These systems have been identified as part of the historical leakages observed throughout the abutments and foundation of the dam.

A general view of this dam is shown in Fig. 5.

2.2 *Seismic history of Cogoti Dam*

Cogoti Dam has been shaken by several earthquakes, being three of them being serious in terms of causing damages to the embankment. These earthquakes are described below.

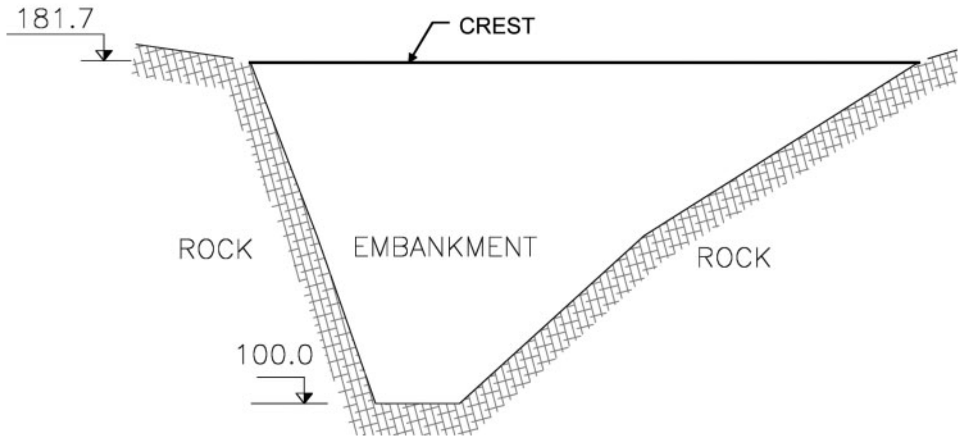

Figure 4. Gorge profile.

Figure 5. General view of Cogoti Dam.

The first shaking that stroke this dam occurred on April 6, 1943, and known as Illapel earthquake, with a 7.9 Magnitude and an epicentral distance from the dam site of about 90 km. The peak ground acceleration (PGA) estimated at the dam site is 0.19 g (Arrau et al., 1985).

The second important earthquake that affected the dam seems to have happened in 1949. However, no record of this seismic event is available.

The third seismic event that hit the dam causing substantial settlements occurred on October 14, 1997 at 22:03, local time. This event has been reported with Magnitudes 7.1 and 6.8 by the USGS and Servicio Chileno de Sismología, respectively. The hypocenter has been located at Latitude −30.9, Longitude −71.2 and at 58 km depth. The resulting epicentral distance from the dam site is 16 km. In the seismic station of Illapel, the peak ground acceleration reached 0.27 g.

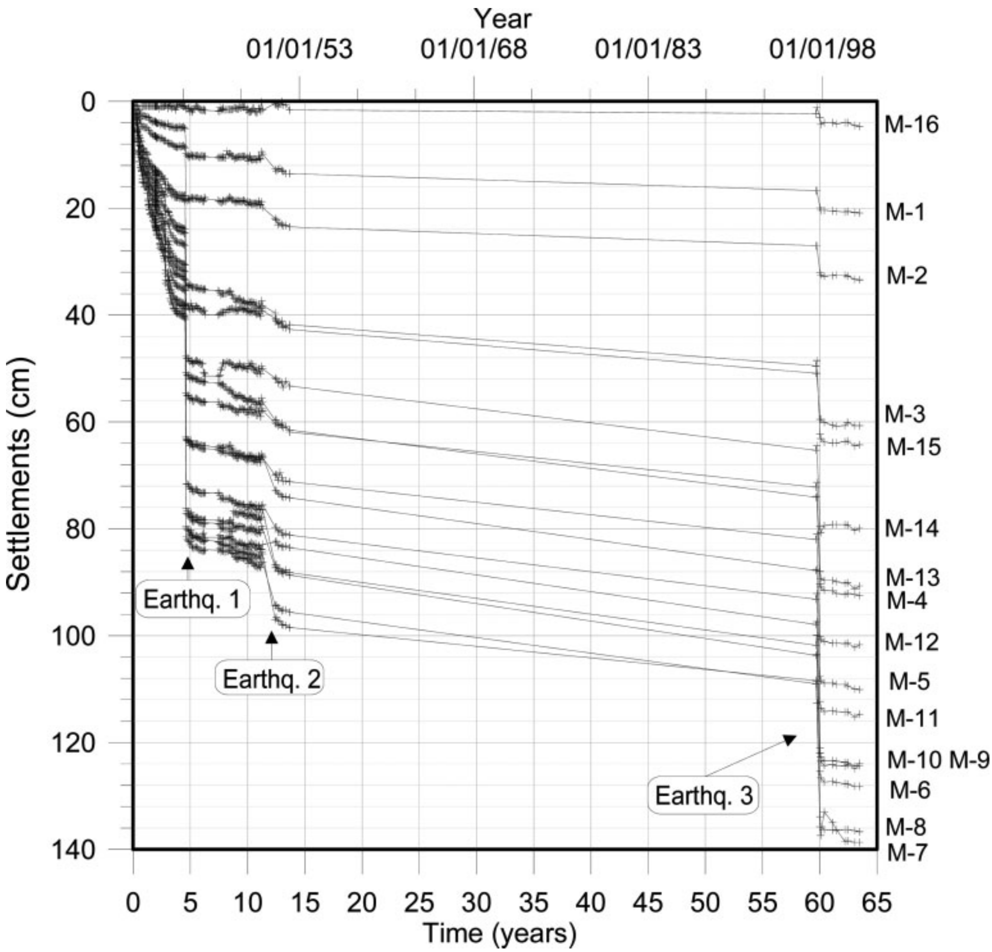

Figure 6. Chronological history of settlements.

2.3 *Seismic settlements*

The chronological history of measured settlements since 1938 at different control points at the crest of the embankment is presented in Fig. 6. The location of the control points are shown in Figs. 3 and 7. Additionally, the distribution of the settlements along the crest at different times is shown in Fig. 8.

These data indicate that the maximum vertical settlement does not take place in the section associated with the maximum height of the rockfill. Unexpectedly, at least for the authors, the maximum vertical settlement occurs above the point where the bedrock undergoes a slope change. The maximum settlement corresponds to control point M7, whose chronological history is shown in Fig. 9.

The maximum seismic settlements, Δ_{SS}, are:

- 1943 Earthquake $\Delta_{SS} = 41.7$ cm, (Control point M-10)
- 1949 Earthquake $\Delta_{SS} = 12.3$ cm, (Control point M-7)
- 1997 Earthquake $\Delta_{SS} = 25.3$ cm, (Control point M-8)

At present, the total maximum seismically-induced settlement reaches a value close to 79 cm, which represents almost 1% of the dam high.

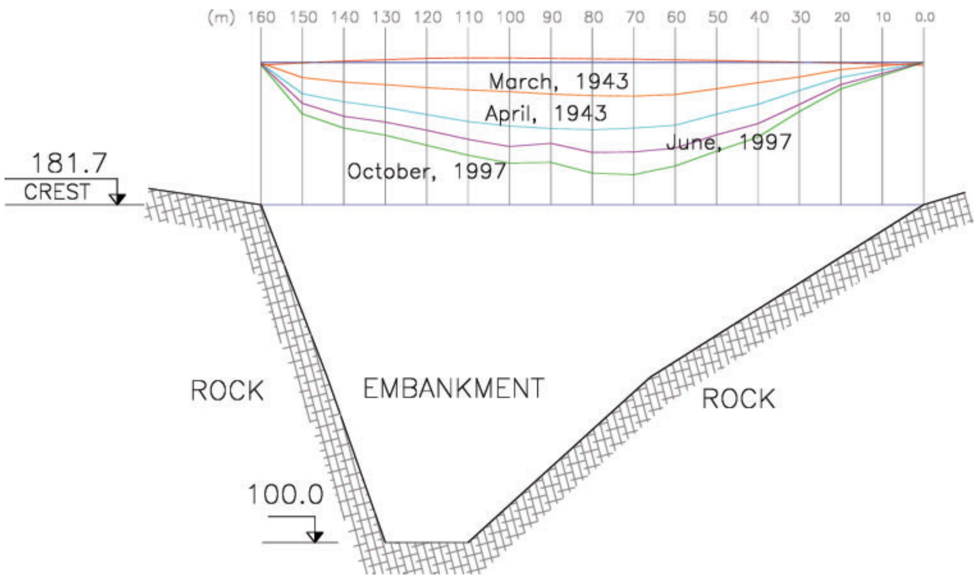

Figure 7. Control points and settlement distribution along the crest.

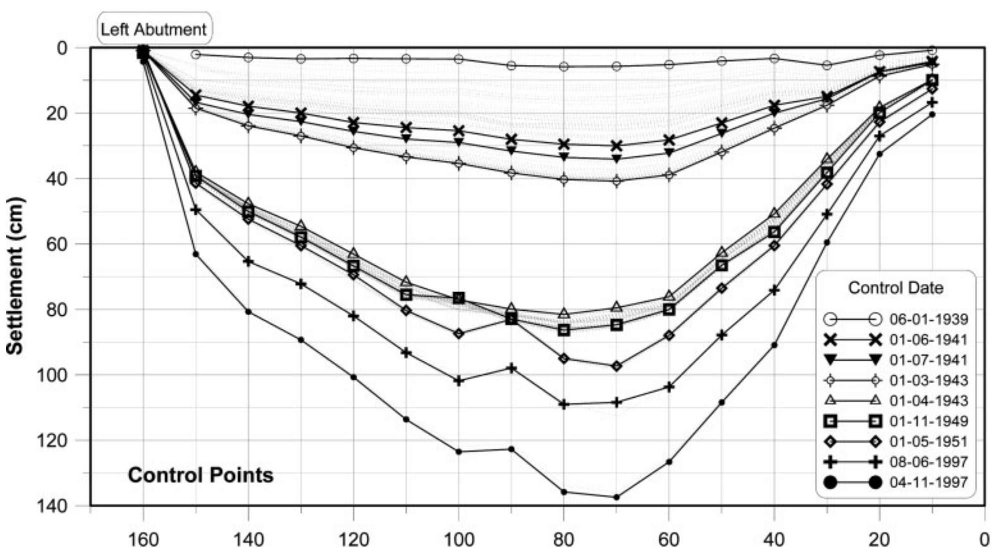

Figure 8. Settlements along the crest at different times.

At the end of 2001, the total maximum settlement reached 138.7 cm, representing a settlement of 1.7% of the height of the dam after 63 years.

The analysis of the static settlements is presented elsewhere (Verdugo, 2001).

2.4 *Observed damages*

After the 1943 earthquake, several longitudinal cracks showed up along the crest, with lengths of 30 to 40 m. Also some transversal cracks were visible in different sections of the crest. Settlements in the concrete face were reported. The most important damage was associated with a large displacement of the rockfill that involved the whole downstream slope. This situation was considered risky and therefore, the slope was immediately repaired.

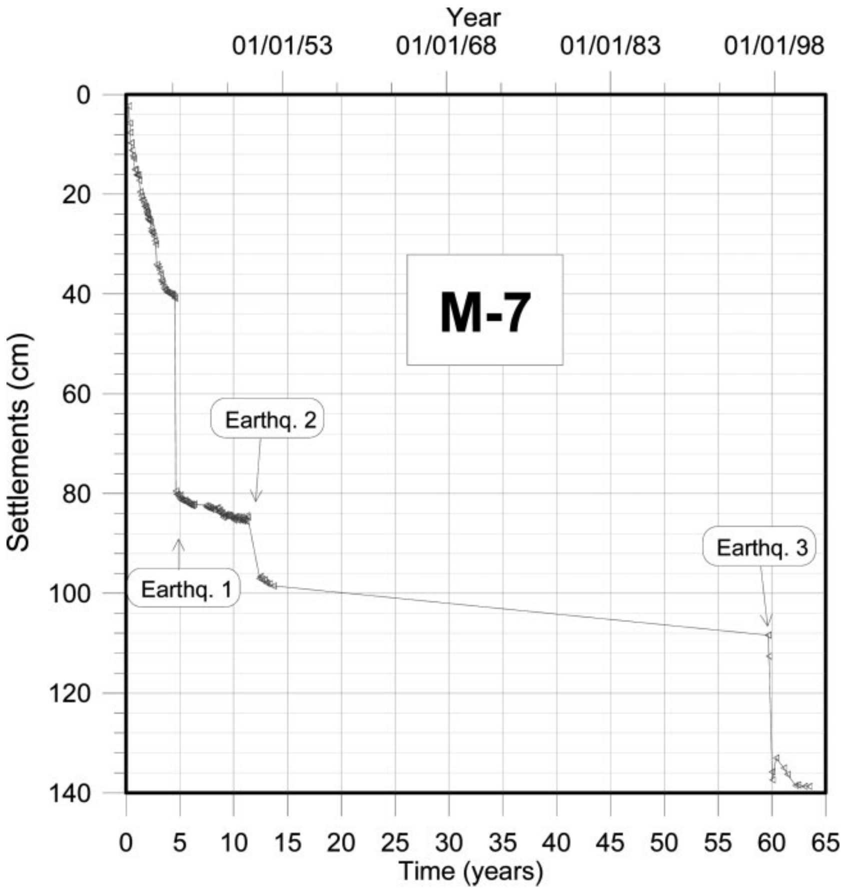

Figure 9. Chronological history of maximum settlement.

The leakage of the dam was increased from 500 liters/sec in 1944 to 2600 liters/sec in 1988, when the concrete face was repaired.

The other seismic events that hit the dam have not caused any further significant damage as compared to what happened during the 1943 earthquake.

3 SANTA JUANA DAM

3.1 *Dam description*

Santa Juana is a rockfill concrete face dam located in the bed of Huasco River and located approximately 17 km east of Vallenar City, II Region of Chile. It was completed in 1995, and it has a capacity of 166 millions m^3.

Santa Juana Dam, with a height off 113.4 m and a crest length of 390 m, was constructed with rock particles with a maximum size of 1 and 0.65 meters in the upstream and downstream supporting shoulders, respectively. All the materials were adequately compacted. A cross section showing the distribution of these two materials is presented in Fig. 10. Inclination of 1.5:1 (H:V) and 1.6:1 (H:V) were designed for the upstream and downstream slope, respectively.

The concrete face has a variable thickness from 45 cm at the base to 30 cm at the top, where it is transformed into a parapet wall.

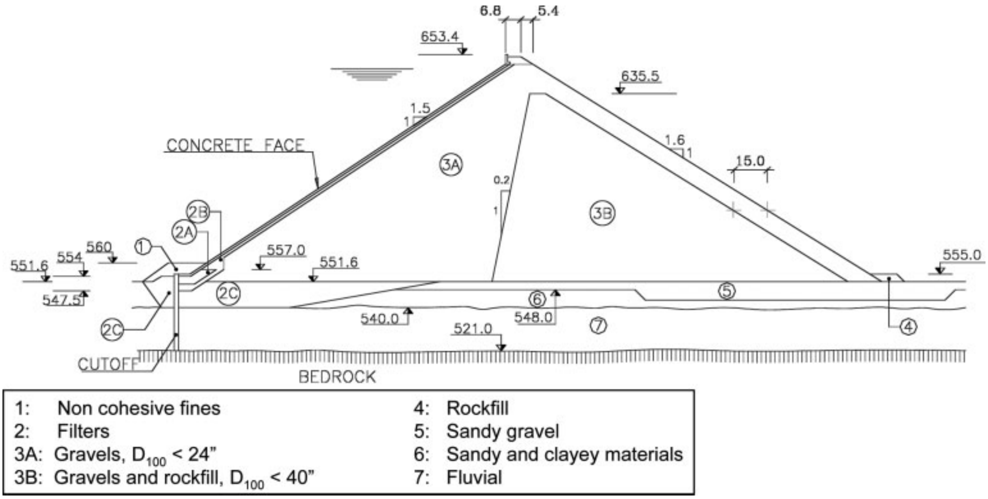

Figure 10. Cross-section of Santa Juana Dam.

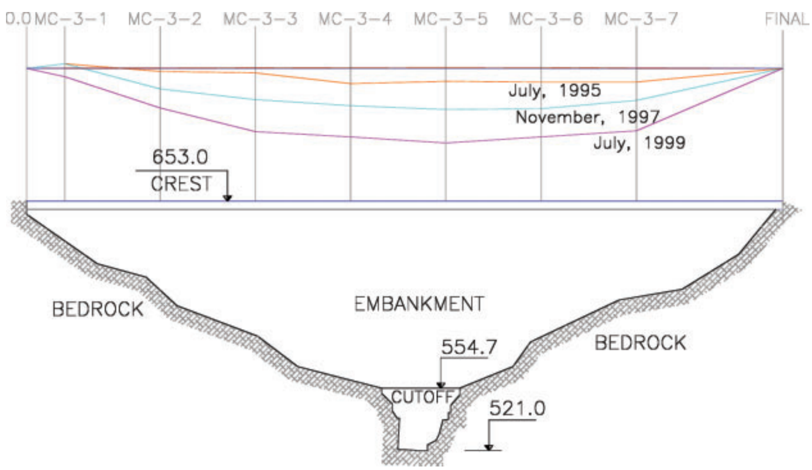

Figure 11. Profile of the valley and settlement distribution.

The shape of the throat of the rather narrow valley eroded by Huasco River is shown in Fig. 11. The profile is rather regular and it is possible to assume that the profile totally consist of bedrock. A cutoff wall made of plastic concrete was built in the fluvial material located at the river bed.

Control points along the crest of the dam were installed to monitor the settlements, which are indicated in Fig.12. Accelerometers were installed at different locations in order to study the seismic response of the dam, and the locations are indicated in Fig. 12. The accelerometer A3 is installed in rock.

3.2 *Observed settlements*

Fig. 13 presents the measured settlements at different control points located along the dam crest. It is observed that the vertical deformation is in some way shifted to the right, confirming the results observed in Cogoti 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 takes place at the control point MC-3-5, and the chronological history is shown in Fig. 14. It can be observed that the settlements after 4 years of operation are quite small

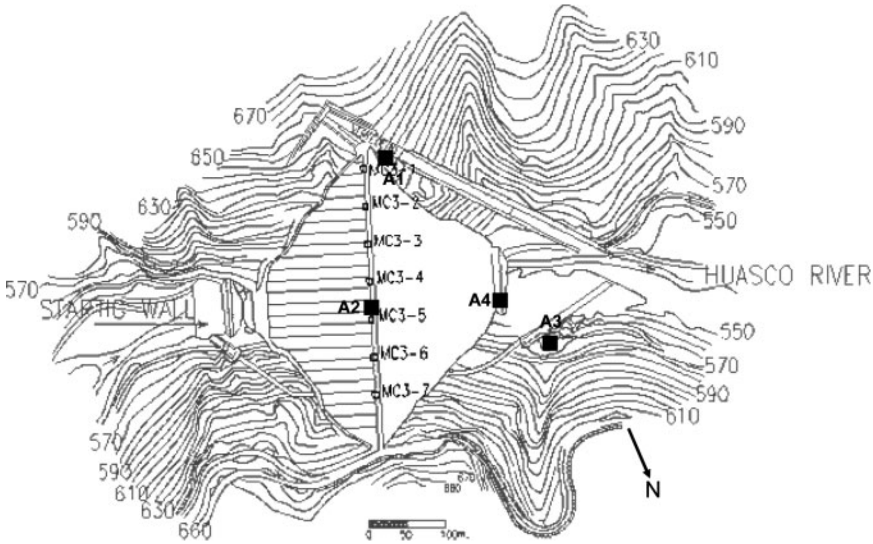

Figure 12. Accelerometers and control points of settlement.

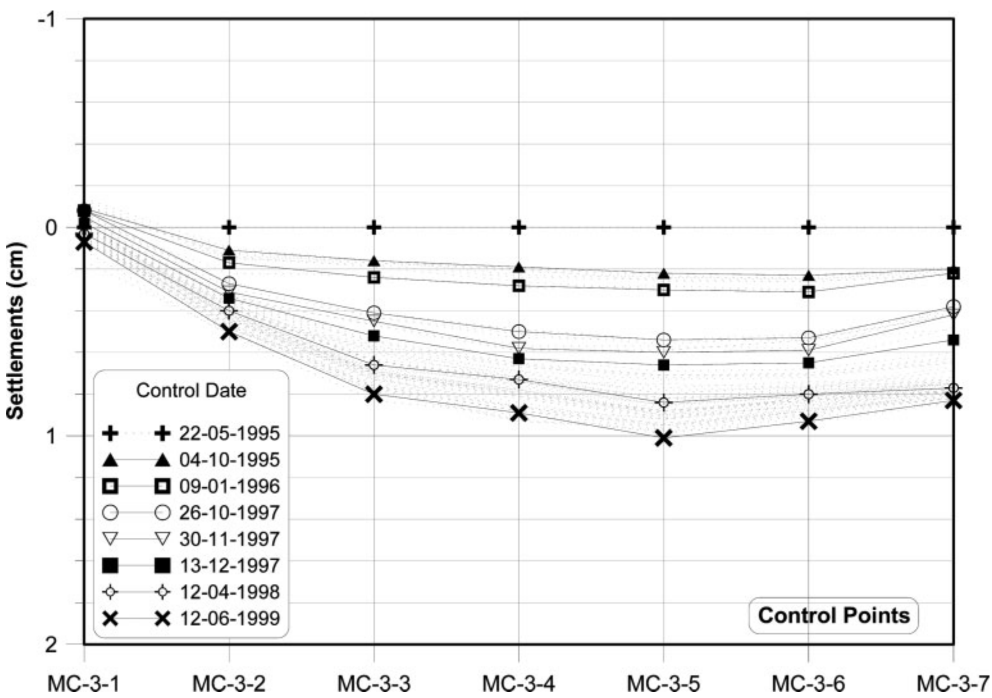

Figure 13. Vertical settlements along the crest.

(1 cm) and it seems that the earthquake that hit the dam did not generate any particular change in the pattern of deformation.

3.3 *Recorded earthquakes*

Between 1997 and 2001, a total of 22 seismic events were recorded by the accelerometers installed in Santa Juana Dam. Table 1 lists the recorded earthquakes sorted according to their peak

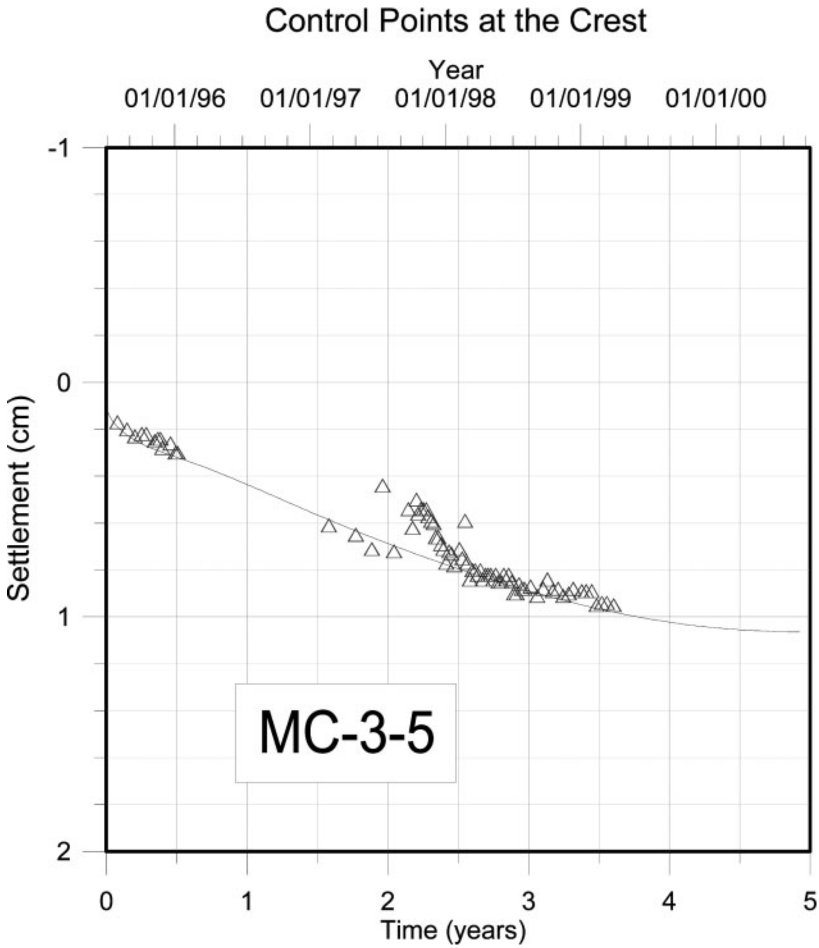

Figure 14. Chronological history of observed settlements at MC 3-5 control point.

accelerations. Earthquake No. 1 has peak acceleration at the crest of 0.226g (E-W), while earthquake No. 22 has peak acceleration at the crest 0.0088 (E-W).

3.4 *Observed seismic response*

The largest earthquake that has hit the dam occurred in October 14, 1997 at 22:03, local time, which has been already described above. This earthquake induced peak accelerations recorded by the instruments and these are presented in Table 2.

According to these data the component E-W is amplified at the top:

$$AMPLIFICATION_{ROCK-DAM\ CREST} = \frac{0.226}{0.048} = 4.7 \tag{1}$$

$$AMPLIFICATION_{TOE-CREST} = \frac{0.226}{0.104} = 2.2 \tag{2}$$

An attempt to evaluate the natural period of the dam has been carried out. Accordingly the transfer functions between “rock – crest” and “toe – crest” were computed. This can be done by dividing the amplitudes of the Fourier transforms of the motions recorded at crest and rock and

Table 1. Recorded earthquakes.

Earthq.	Lat.	Long.	Depth (km)	Date	Mag.		Crest	Spillw.	Toe	Rock
1	30.445	71.197	55	14/10/1997	6.8	E-W	0.226	0.078	0.104	0.048
						Vertical	0.157	0.056	0.111	0.04
						N-S	0.124	0.072	0.073	0.049
2	28.343	72.029	95	18/07/1998	5.1	E-W	0.132	0.132	0.051	0.132
						Vertical	0.087	0.064	0.036	0.132
						N-S	0.064	0.067	0.040	0.132
3	28.456	70.797	124.2	24/12/2001	4.9	E-W	0.113	0.114	0.078	0.054
						Vertical	0.104	0.103	0.09	0.033
						N-S	0.091	0.117	0.075	0.054
4	29.268	71.299	54.3	08/10/2000	4.9	E-W	0.104	0.051	0.046	0.026
						Vertical	0.084	0.034	0.042	0.016
						N-S	0.075	0.046	0.056	0.025
5	29.253	71.638	33.7	09/07/2000	4.8	E-W	0.067	0.039	0.018	0.03
						Vertical	0.081	0.046	0.011	0.018
						N-S	0.037	0.054	0.01	0.032
6	30.442	71.315	50.2	06/11/2001	5.1	E-W	0.051	0.009	0.018	0.005
						Vertical	0.026	0.004	0.013	0.005
						N-S	0.022	0.01	0.011	0.005
7	29.399	72.128	0	14/09/1998	4.5	E-W	0.049	0.018	0.022	0.011
						Vertical	0.039	0.017	0.029	0.011
						N-S	0.047	0.024	0.034	0.018
8	28.078	71.257	228.6	16/06/1998	4.5	E-W	0.044	0.049	0.033	0.01
						Vertical	0.038	0.026	0.019	0.007
						N-S	0.026	0.038	0.021	0.014
9	29.477	71.821	43.2	04/09/2001	4.6	E-W	0.033	0.038	0.014	0.005
						Vertical	0.013	0.005	0.009	0.003
						N-S	0.017	0.009	0.013	0.006
10	27.879	70.421	94	09/08/2001	4.6	E-W	0.033	0.01	0.018	0.007
						Vertical	0.017	0.005	0.012	0.006
						N-S	0.015	0.01	0.015	0.008
11	27.041	70.741	222.8	22/11/1997	4.9	E-W	0.026	0.007	0.018	0.006
						Vertical	0.025	0.008	0.015	0.013
						N-S	0.018	0.009	0.015	0.01
12	29.567	71.9	0	01/09/2007	4.8	E-W	0.025	0.013	0.019	0.006
						Vertical	0.02	0.009	0.013	0.005
						N-S	0.013	0.012	0.014	0.008
13	28.502	71.39	23.8	17/08/2000	4.4	E-W	0.025	0.007	0.018	0.009
						Vertical	0.014	0.005	0.011	0.006
						N-S	0.016	0.009	0.01	0.008
14	31.099	71.86	25.8	12/01/1998	6	E-W	0.023	0.004	0.011	0.003
						Vertical	0.014	0.004	0.008	0.01
						N-S	0.013	0.004	0.009	0.003
15	28.655	69.982	37.2	21/08/2001	3.9	E-W	0.023	0.011	0.014	0.005
						Vertical	0.015	0.005	0.012	0.003
						N-S	0.016	0.013	0.015	0.006
16	28.741	71.935	224.6	05/09/1997	4.8	E-W	0.021	0.006	0.011	0.005
						Vertical	0.013	0.006	0.015	0.004
						N-S	0.016	0.008	0.012	0.006
17	29.513	72.26	34.2	03/09/1998	4.9	E-W	0.02	0.009	0.015	0.007
						Vertical	0.016	0.007	0.009	0.006
						N-S	0.014	0.009	0.01	0.008
18	28.365	68.951	131.6	16/10/2001	5.3	E-W	0.018	0.006	0.01	0.005
						Vertical	0.02	0.004	0.009	0.004
						N-S	0.016	0.009	0.011	0.006

(*Continued*)

Table 1. (Continued)

Earthq.	Lat.	Long.	Depth (km)	Date	Mag.		Crest	Spillw.	Toe	Rock
19	28.95	71.181	163.1	05/10/1998	4.1	E-W	0.018	0.011	0.015	0.007
						Vertical	0.017	0.007	0.014	0.005
						N-S	0.014	0.014	0.014	0.01
20	29.162	70.802	218.4	28/09/1998	4.4	E-W	0.016	0.007	0.014	0.006
						Vertical	0.015	0.005	0.011	0.003
						N-S	0.015	0.012	0.012	0.006
21	28.152	71.148	87.7	12/08/2001	4.3	E-W	0.012	0.006	0.009	0.008
						Vertical	0.013	0.005	0.007	0.005
						N-S	0.01	0.01	0.008	0.011
22	28.049	68.12	394.2	29/01/1999	5	E-W	0.009	0.001	0.003	0.001
						Vertical	0.007	0.001	0.003	0.003
						N-S	0.005	0.002	0.004	0.001

Table 2. Peak recorded accelerations (g).

Accelerometer	E-W	N-S	U-D
A1	0.078	0.072	0.056
A2	0.226	0.124	0.157
A3	0.048	0.049	0.040
A4	0.104	0.073	0.111

crest and toe. In the present analysis, the ratio of the undamped velocity response spectra of the two records were used. As shown by Hudson (1956; 1979), the undamped velocity response spectrum, given by:

$$\left|\dot{x}(t)\right|_{max} = \sqrt{\left[\int_0^{t^*} a(\tau)\cos\omega\tau\, d\tau\right]^2 + \left[\int_0^{t^*} a(\tau)\, sin\omega\tau\, d\tau\right]^2} \tag{3}$$

is an upper bound of the Fourier amplitude spectrum:

$$FA(\omega) = \sqrt{\left[\int_0^{L} a(t)\cos\omega t\, dt\right]^2 + \left[\int_0^{L} a(t)\, sin\omega t\, dt\right]^2} \tag{4}$$

The values of the peaks are almost identical in most cases and the numerical smoothing that is always necessary to apply to the Fourier Transform is avoided. Additionally, the ratio of response spectra are less sensitive to noise and the peaks are easier to identify. Typical results are shown in Fig. 15.

The periods associated with the maximum value of the response spectral ratio were identified and considered as predominant periods. The analysis of the available data is summarized in Fig. 16. It can be seen that a systematic predominant period for different earthquakes and different ratios ("crest – rock" and "crest – toe") is obtained. According to these results, the predominant period of Santa Juana Dam in the E-W direction would be: T = 0.4 seconds.

If the classical expression of the predominant period is used:

$$T = 2.61\frac{H}{V_S} \tag{5}$$

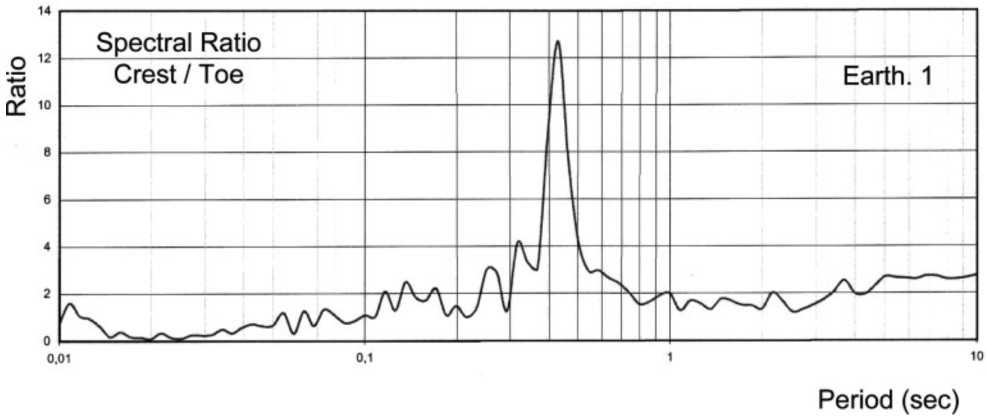

Figure 15. Typical result of spectral ratio Crest – Toe.

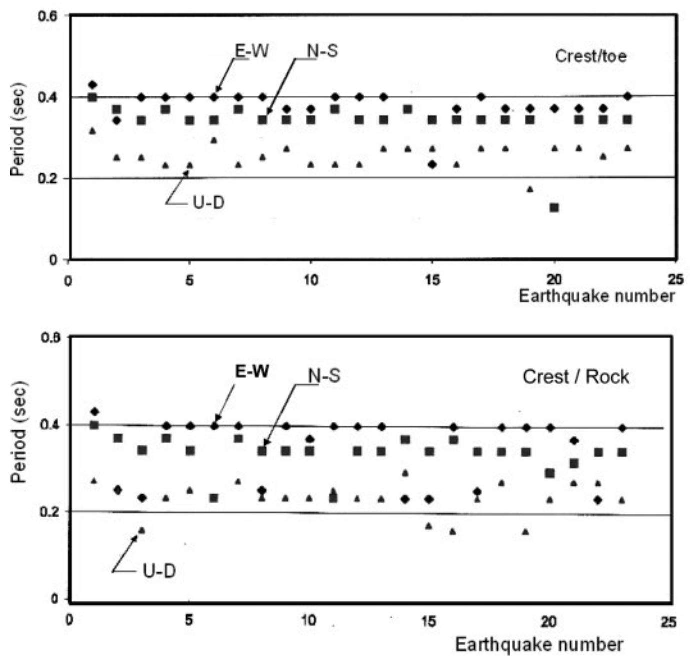

Figure 16. Predominant period computed in each earthquake.

Where, H and Vs represent, respectively, the height of the dam (113.4 m) and the shear wave velocity of the fill ($\approx$700 m/sec). The theoretical predominant period would be: T = 0.42 seconds.

The match between empirical and theoretical values is quite good, although the geometry of Santa Juana Dam (crest length/high: 390/113.4 = 3.4) does not satisfy the condition of plane deformation (crest length/high > 4).

4 AROMOS DAM

4.1 *Dam description*

Completed in 1979, Aromos Dam is located in the Province of Quillota, approximately 90 km to the north-west of Santiago (capital city of Chile). The reservoir has a capacity of 60.3 million m^3.

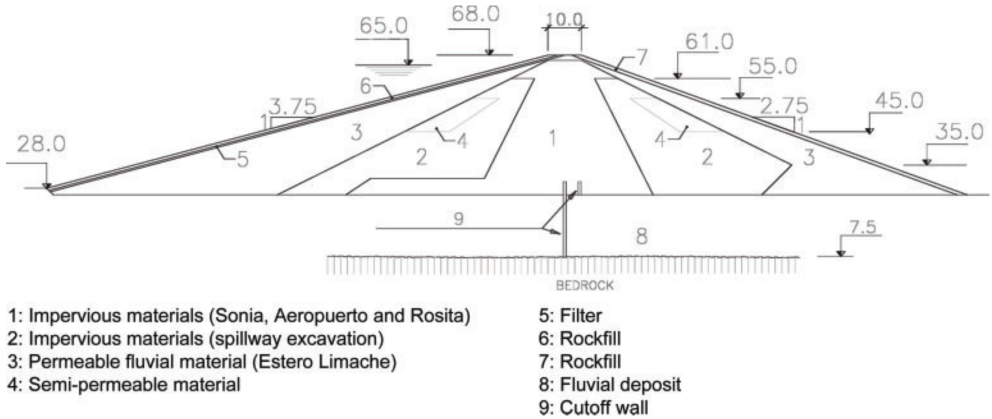

Figure 17. Cross-section of Aromos Dam.

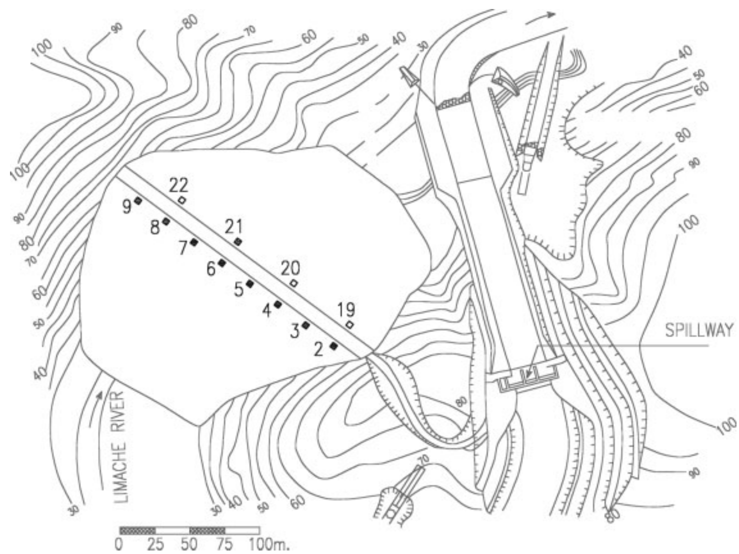

Figure 18. Plan view of dam-site.

The embankment is placed in a narrow zone of Limache Creek, about 5 km downstream of the confluence with Aconcagua River. The cross section and plan view are presented in Fig. 17 and 18, respectively.

Aromos is a zoned dam 42 m high and 220 m long along the crest. The upstream and downstream 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 consist of sandy soil materials. Both abutments consist of weathered granite that improves with depth. The shape of the throat is shown in Fig. 19.

A plastic concrete wall of 80 cm in thickness and a maximum depth of 22.5 m is buried into the foundation ground, with a part into the clay core, as impervious system to reduce infiltration throughout the fluvial deposit existing below the dam. It is important to mention that in several sectors the plastic concrete wall did not reach the bedrock; however, the information indicates that these sectors were injected. Detail of this injection is not available.

The grain size distribution bands of the soil materials constituting zone 2, zone 3 and core are presented in Figs. 20. The Atterberg Limits indicate that the fines of the core material are classified

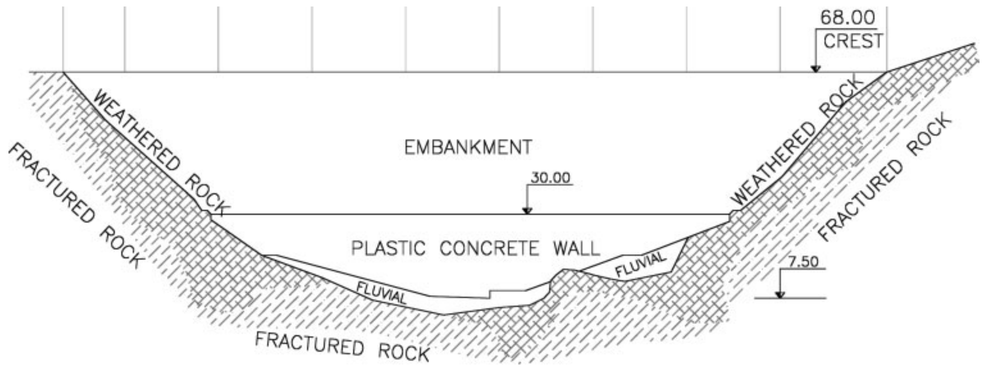

Figure 19. Profile of the gorge.

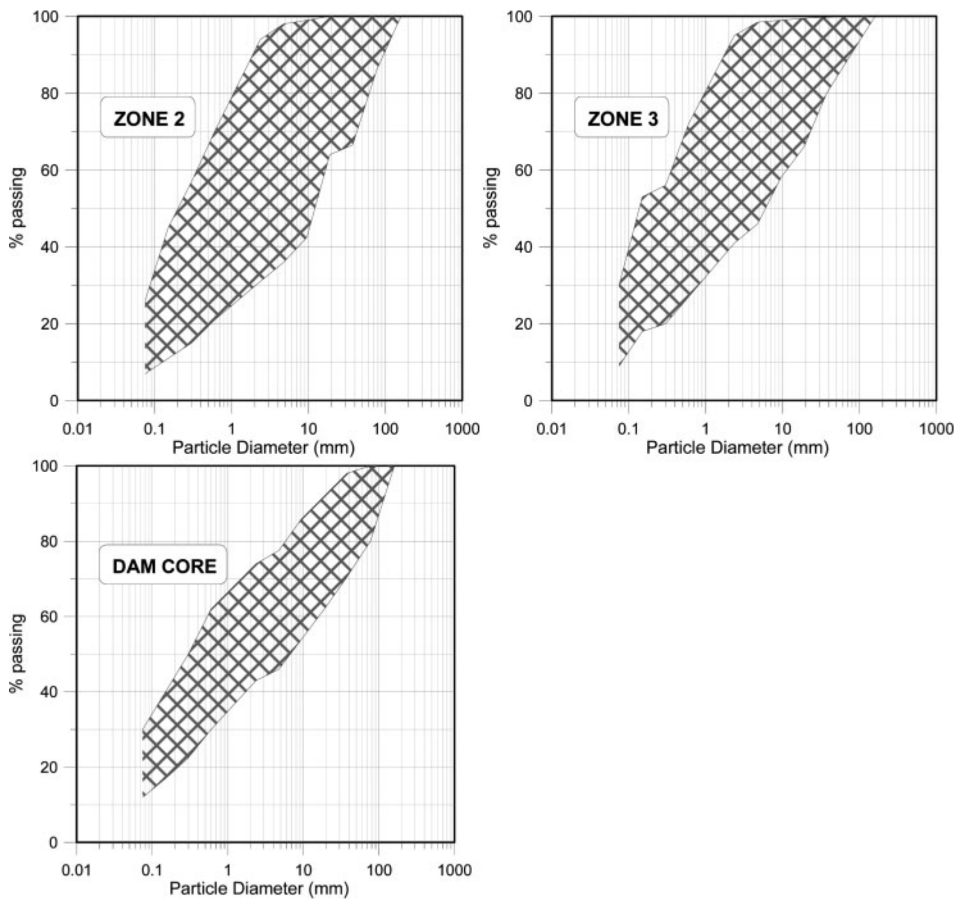

Figure 20. Grain size distribution bands of materials used in the dam.

as CL. Therefore, the core material is actually clayey sand. It is interesting to mention that materials 2 and 3 are rather similar to each other.

4.2 *Amazing finding at the end of construction*

When the construction of the embankment was almost finishing, a bulldozer operating in the river bed, near to the downstream toe, suddenly sank several centimeters. This evidence, together

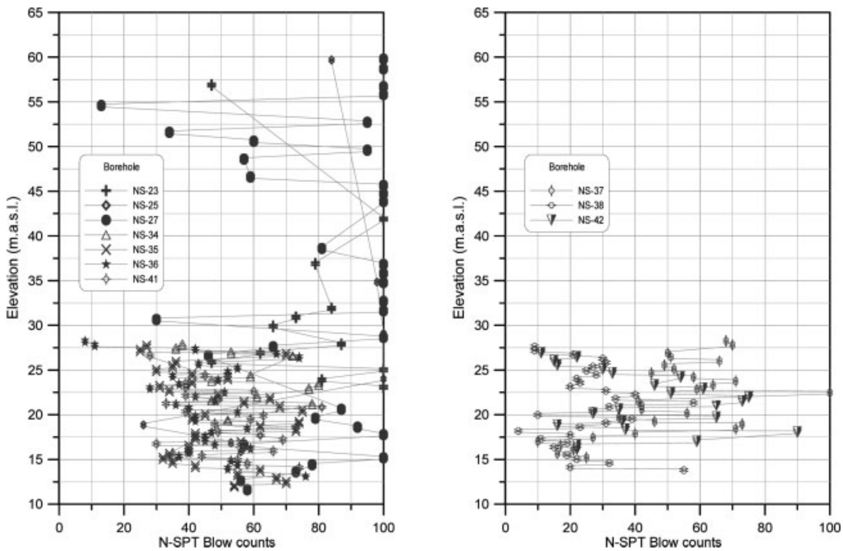

Figure 21. SPT N-values measured in the upstream zone.

with the efforts performed to pull out this equipment revealed that the ground was susceptible of undergoing liquefaction. Consequently, the engineers in charge of the dam construction worried about this issue and a great concern regarding the seismic stability of the dam arose.

As a result of this anomalous situation, geotechnical services of a well reputed international consulting office were appointed, so an external evaluation about the actual risk of liquefaction could be obtained. To assess the required seismic stability of Aromos dam, a complementary geotechnical site investigation was performed. Boreholes with Standard Penetration Testing (SPT) were carried out. The available information associated with site investigation is presented below.

The Aromos dam was completed and its operation was delayed until the results of the requested seismic stability analysis became available.

4.3 *Geotechnical site investigation*

The main concern of the site investigation was to clarify the potential occurrence of liquefaction in the foundation that could trigger a major seismic failure. Two field exploration programs were carried out; in 1979 and 1981, which basically consisted of standard penetration testing.

According to the area where the SPT were performed, the results were separated into three groups:

– upstream zone
– downstream zone
– spillway zone

Additionally, the results for each sector were separated into two sets:

– Those where SPT N-values are predominantly greater than 15–20 blow/feet
– Those where there are several SPT N-values smaller than 15–20 blow/feet

The results are shown in Figs. 21 to 23 for the upstream zone, downstream zone and spillway zone, respectively. These results are not normalized to an overburden pressure of 1 kg/cm^2.

These data show significant number of boreholes where the SPT N-values presents values below 15–20 blow/30 cm, suggesting that liquefaction phenomenon could takes place under a strong motion as the one expected for the area.

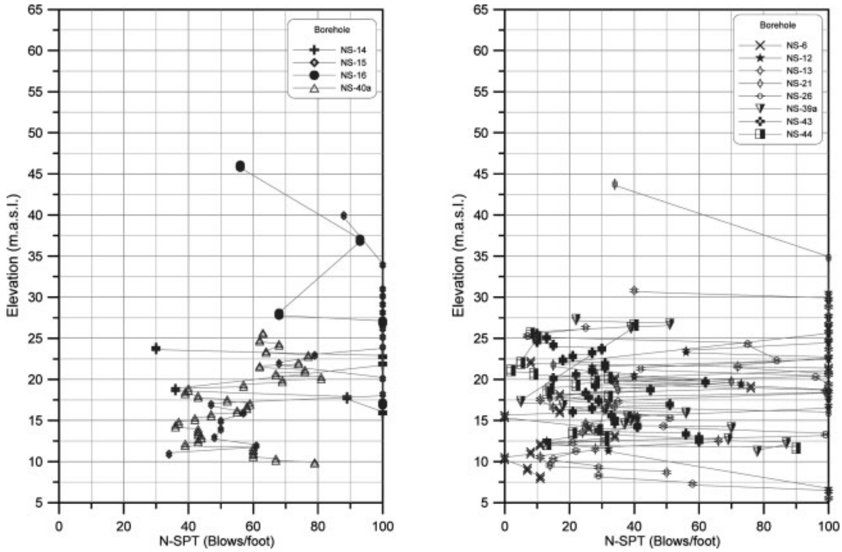

Figure 22. SPT N-values measured in the downstream zone.

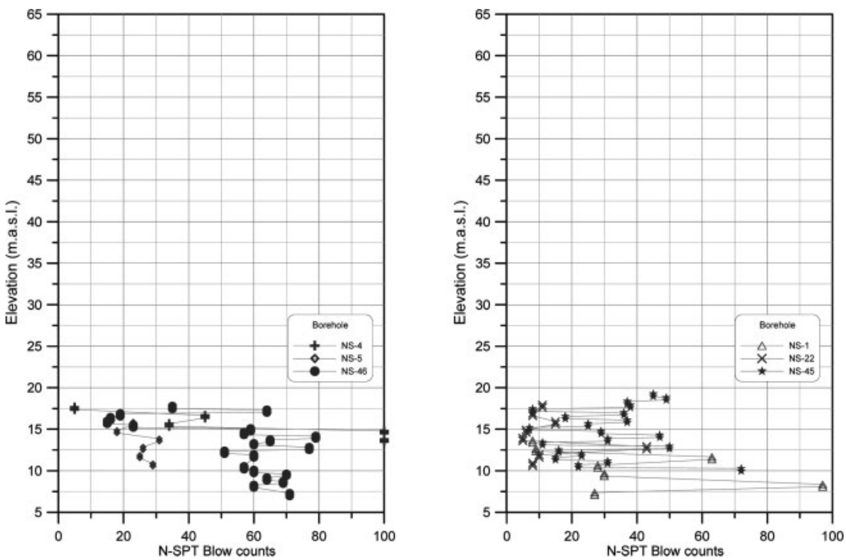

Figure 23. SPT N-values measured in the spillway zone.

4.4 *Results and recommendations proposed in 1981*

In August 1981, the international specialists concluded that the soil deposits where the Aromos Dam had been founded presented a risk of liquefaction, although they recognized that it should not comprise the whole foundation. According to this result, the following two main countermeasures were proposed:

- A berm of compacted material confining one third of the upstream shell, with a height of 13 m
- Drill a battery of drainage columns in the area between the toe of the dam and the spillway.

The available information indicates that only some drainage columns were actually installed at the toe of the dam. Due to the economic situation existing in Chile at that time, nothing else

Table 3. Recorded peak acceleration near Aromos Dams (March 3, 1985).

Station	Latitude	Longitude	Component H1 (g)	Component H2 (g)	Vertical Component (g)
Llay-Llay	−32°50′	−70°58′	0.335 (N80W)	0.486 (S10W)	–
Viña del Mar	−33°02′	−71°35′	0.228 (N70W)	0.356 (S20W)	0.171
Almendral	−33°01′	−71°38′	0.293 (N50E)	0.163 (S40E)	–
Universidad T.F.S.M	−33°01′	−71°38′	0.164 (S20E)	0.179 (N70E)	0.125

Location of Aromos Dam: −32°57′ −71°23′

was done in the already constructed dam. Initially, it was decided to operate the dam with a small amount with respect to its capacity, but little by little it was finally used at its full capacity of water storage.

4.5 *The 1985 Chilean earthquake*

In March 3, 1985, a strong earthquake of Magnitude 7.8 hit Central Chile. The maximum intensity (modified Mercalli) in the epicentral area reached VIII. Peak accelerations measured in seismic stations close to the site of Aromos Dam are presented in Table 3 (Saragoni et. al, 1986).

As can be observed, peak ground acceleration in the area of Aromos Dam would likely be in the range 0.3 to 0.4 g. This level of acceleration triggered liquefaction phenomenon in La Marqueza and La Palma Dams, which almost collapsed after developing significant deformations (De Alva et al., 1987).

4.6 *Seismic behavior of Aromos Dam during March 3, 1985 earthquake*

Although there was a concern regarding the potential seismic instability of the foundation of the dam, and the proposed improvements were almost no adopted at the time. Of the 1985 earthquake, the dam was operating under regular basis.

The settlements recorded at the crest of dam are shown in Fig. 24. It can be observed that a regular development of the settlements before and after the 1985 earthquake ocurred. Again, it is observed that the maximum vertical deformation takes place above an irregularity in the bedrock.

The chronological history of measured settlements in different control points at the crest of the embankment is presented in Figs. 25 and 26 for the upstream and downstream side, respectively.

These data indicate that the 1985 earthquake induced vertical settlements in the embankment that were smaller than 10 cm. This behavior does not fit with what was somehow predicted in 1981 with the available information at that time.

Additionally, it is important to mention that the drainage columns located at the toe of the dam worked and water was observed coming up from these wells. Moreover, the intensity of the ground motion in the area of Aromos Dam is somewhat reflected in the relative displacements observed in the concrete walls of the spillway after this earthquake, as shown in Fig. 27.

4.7 *The unexpectedly good seismic response of Aromos Dam*

Definitely the extremely good seismic response of Aromos Dam, with maximum settlement less than 10 cm, was something unexpected, due to the results of previous analysis.

The boreholes with SPT N-values preponderantly lower than 15–20 blow/feet were identified and they are marked with black squares in Fig. 28. On the other hand, those boreholes with most of their SPT N- values greater than 20 blow/feet are indicated with black circles.

It can be observed that the areas of low SPT N-values are surrounded by areas with high SPT N-Values. This particular geotechnical condition can be the explanation of the good behavior of this site. The areas with high SPT N-values should have two important contributions:

– They dilate reducing the pore pressure build-up in the loose zones
– Because they are stiffer, they take more of the dynamic load, reducing the disturbances of loose zones.

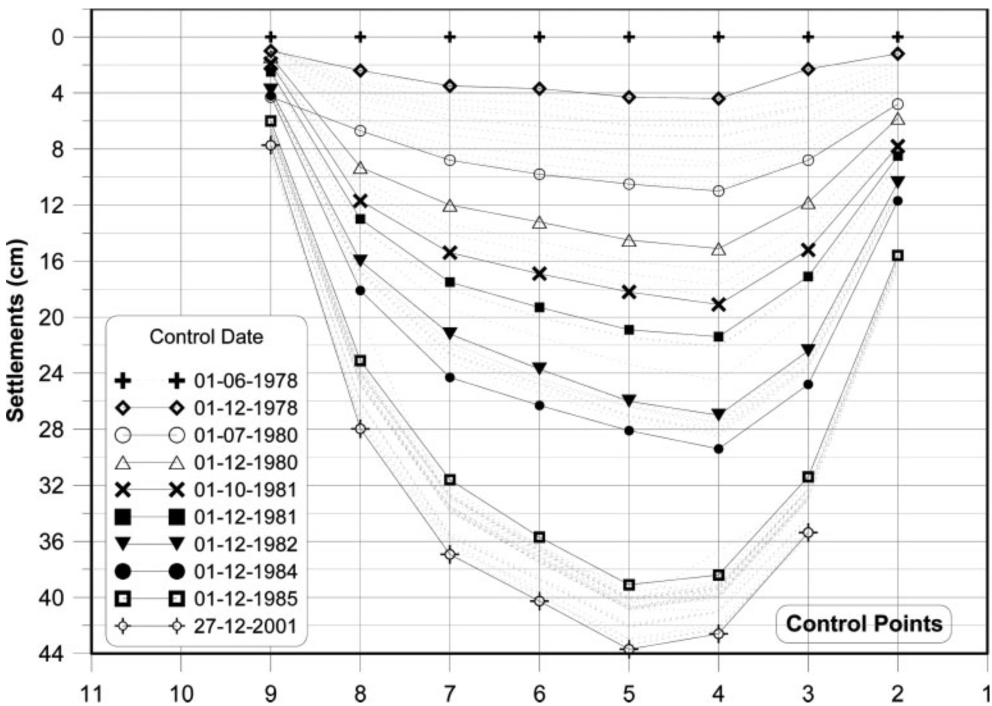

Figure 24. Settlements recorded in the crest of Aromos Dam.

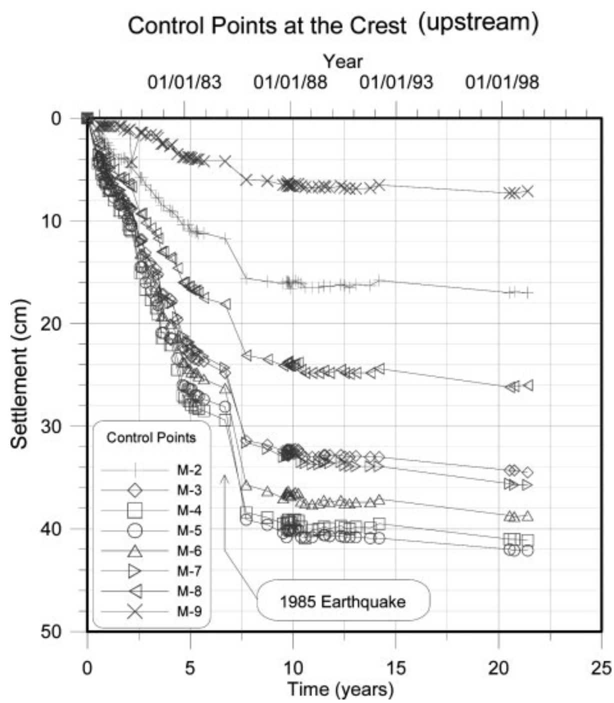

Figure 25. Chronological history of settlements at the upstream side of the crest.

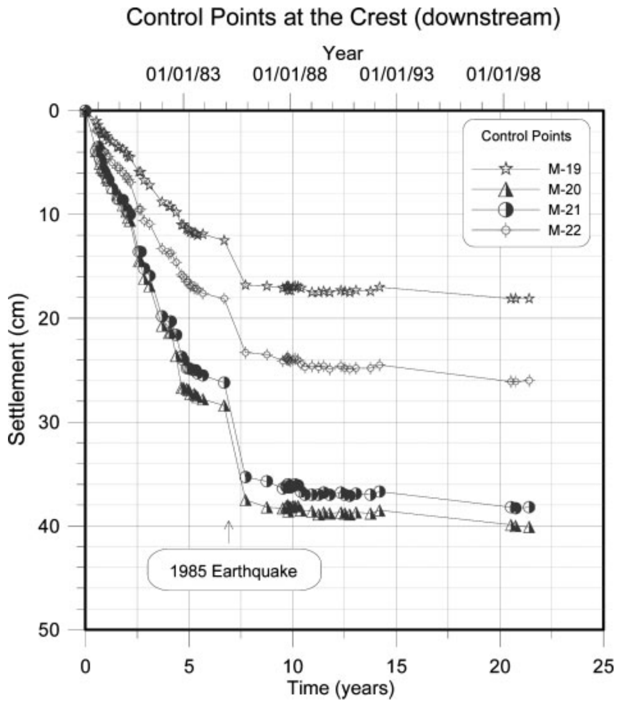

Figure 26. Chronological history of settlements at the downstream side of the crest.

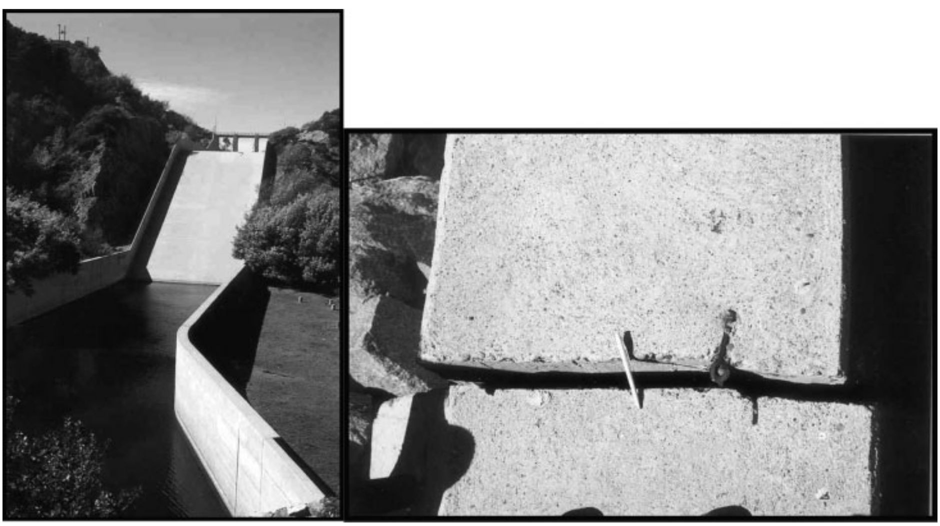

Figure 27. Relative displacements in the spillway walls.

More studies being conducted in this area in order to have a better understanding of the absence of liquefaction in sandy soils with very low SPT N-values.

5 CONCLUDING REMARKS

The observed seismic behavior of three existing dams located in Chile has been presented.

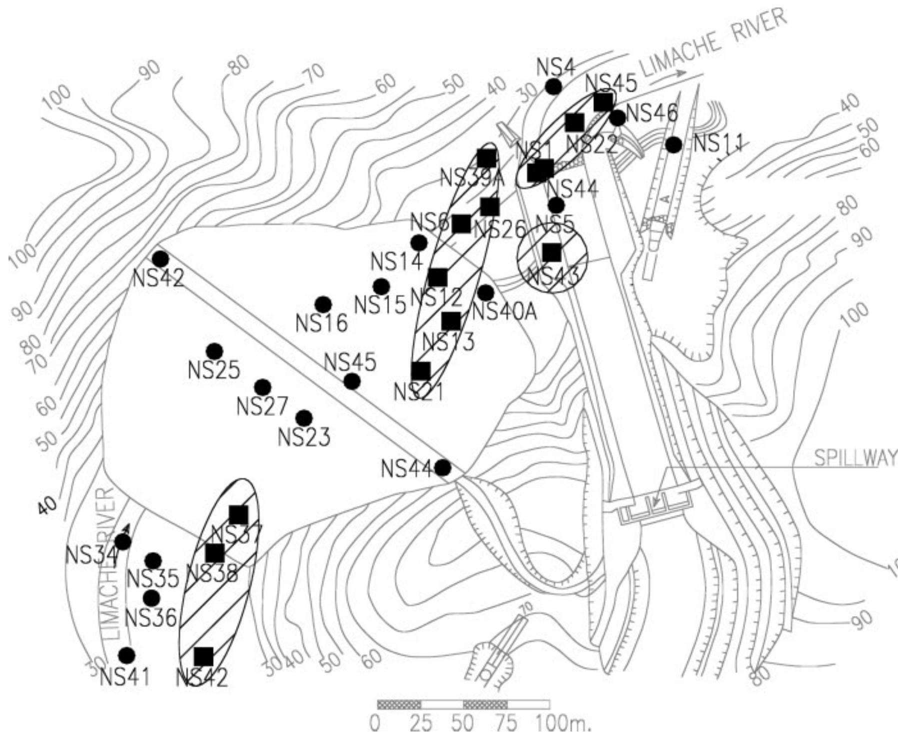

Figure 28. Identification of boreholes with SPT N-values lower than 15–20 blow/30 cm.

Cogoti Dam completed in 1938 is a good example of the magnitude of seismic settlements and its relation with static settlements in a non compacted rockfill. Additionally, it shows the important damage that may occur on the concrete face caused by seismic settlements. Moreover, the behavior of Cogoti Dam shows how noble is a rock-fill material for being used in dam construction, since even large seismic settlements did not cause any problem in the dam stability.

Santa Juana Dam completed in 1995 has been subjected to several earthquakes, which have been recorded by four accelerometers distributed in the dam and in the site. The data clearly show the phenomenon of amplification and the existence of a predominant period which is quite similar to the theoretical value. These results are very useful for future earthquakes because any change in this natural period can be quickly interpreted in terms of change in the mechanical behavior of the body of the dam. Accordingly, it is strongly recommended the use of this type of instrumentation in dams that permits to detect any potential local failure immediately after an earthquake.

Aromos Dam represents a remarkable case to learn that the actual occurrence of liquefaction in a ground with heterogeneous conditions still requires a deeper analysis and a better understanding of the interaction phenomena. It seems that whenever liquefiable materials are enclosed or surrounded by dense soils which are not liquefiable, the whole behavior of the ground is controlled by the more competent material and liquefaction can not take place.

ACKNOWLEDGEMENTS

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