Seismic Performance of Earth Structures during the February 2010 Maule, Chile, Earthquake: Dams, Levees, Tailings Dams, and Retaining Walls

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The 27 February 2010 Maule, Chile, earthquake occurred during the driest time of the year, which implied that most of the soils in the slopes were not saturated and that the dams had extra freeboard. This may explain the small number of slope failures caused by the earthquake. However, two important earth dams suffered seismically induced permanent ground movements, but no catastrophic damage was reported because the reservoirs levels were low. Five medium-sized mine tailings dams failed due to liquefaction; one of them tragically caused four casualties. Retaining structures of all types performed well and no failures were observed. [DOI: 10.1193/1.4000043]

INTRODUCTION

The region strongly shaken by the 27 February 2010 Maule, Chile, earthquake ($M_w = 8.8$) has a high rate of rainfall during winter, but it is dry during the summer season. At the time of the earthquake, many slopes were either dry or, at most, partially saturated, and consequently, more stable. As a result, most slopes performed well, and while some large failures did occur, overall, the number of slope failures was remarkably small considering the intensity and duration of strong ground motion and the topography of the region.

The region affected by the Maule earthquake contains a number of large earth dams associated with irrigation systems owned by the Chilean Government. Additionally, there are more than 500 private, small earthen dams built by farmers, usually constructed without engineering studies. In addition, there are private dams associated with hydroelectric power plants, but information regarding their seismic performance is not publicly available, although no significant damage has been reported.

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Since the earthquake occurred the dry season, the reservoirs were well below their maximum capacity and the dams had significant freeboard. Thus, while a large number of small dams built by farmers suffered damage due to liquefaction, slope failure, and settlement, no catastrophic failures occurred because reservoir levels generally were low. However, two government-owned reservoir dams, Coihueco and Lliu-Lliu, did suffer damage, although there was no water release because the reservoir water levels were low.

There were eight major mine tailings dams and more than 50 small to medium tailings dams, some of which had rather limited engineering design, in the region affected by the Maule earthquake. While the major tailings dams all performed well, five medium tailings dams developed seismic distress; four of the latter experienced localized failure, and one experienced a liquefaction-induced flowslide that resulted in four fatalities.

Finally, due to the nature of the terrain, there are many retaining structures for slopes and embankments. Overall, the performance of retaining structures of all types was good, and no failures were observed. For brevity, this paper emphasizes select performance observations. Additional information on the sites described herein is contained in the Geotechnical Extreme Events Reconnaissance (GEER) post-earthquake reconnaissance report by Bray and Frost (2010).

GROUND SHAKING DURING THE 2010 MAULE EARTHQUAKE

The ground motions that affected the central-south part of Chile on 27 February 2010 produced moderate values of peak ground acceleration (PGA) on rock outcrops, which were amplified in soil deposits (Boroschek et al. 2010). The available PGA values measured on rock outcrops are shown in Figure 1, and primarily range from 0.13 g to 0.20 g. The exception is the PGA of 0.32 g (EW component) recorded at Santa Lucia. The acceleration-time histories recorded at Cerro Santa Lucía and Convento Viejo are presented in Figure 2, together with the acceleration response spectra. These records illustrate the variation in frequency content measured at rock outcrops during the earthquake. Another important aspect of the earthquake is related to its long duration, which approached two minutes, as substantiated by the acceleration records.

Considering the large area of the fault rupture, the usual characterization of the epicenter by a single point does not adequately represent the actual phenomenon of seismic energy generation-propagation. Thus the epicenter has been replaced by the "fault trace," which corresponds to the locus that represents the projection at the surface of the probable initiation of the rupture. Therefore, the fault trace shown in Figure 1 was created by drawing a straight line throughout the epicenter and parallel to both sides of the rupture zone reported by seismologists (USGS 2010).

SEISMIC PERFORMANCE OF EARTH DAMS

The locations of the earth dams constructed for irrigation purposes that were strongly shaken by the earthquake are shown in Figure 3. With the exception of the Los Cristales Dam, which has an asphaltic concrete core, the dams are either homogenous or zoned-earth fills. Concrete-faced rockfill dams (CFRD) are not present in the affected area.

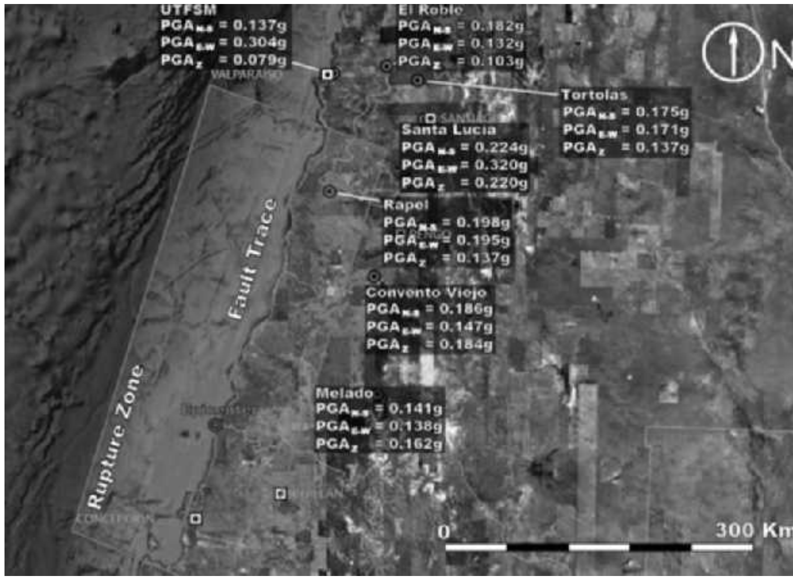


Figure 1. Peak ground accelerations recorded on rock outcrops (background image courtesy of Google Earth).

Two dams that were not damaged during the earthquake are of particular interest: one because of its proximity to the fault rupture and the other as a result of its asphaltic concrete core.

- Tutuvén Dam was the closest dam to the fault plane generated by the earthquake, with similar distances of about 40 km to both the conventional epicenter and to the fault trace as illustrated in Figure 3. According to the limited available information, this dam was built in 1951 as a homogeneous dam. It was repaired in 1979 due to serious damage caused by a flood. Its crest length is close to 1,000 m, and its maximum height is 17 m. It is interesting to observe that the cross section shown in Figure 4 indicates a systematic reduction of the downstream slope, which likely ensured its seismic stability.
- The dam with the asphaltic core is Los Cristales Dam, which was built in 1976 with the spillway in a rock outcrop located close to the center, dividing the dam in two sectors (Figure 5a). It has a total crest length of 310 m and a maximum height of 27 m. The upstream and downstream slopes are $1.9H:1V$ and $1.75H:1V$, respectively (Figure 5b). The asphaltic vertical concrete 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 close to one of the abutments, as shown in Figure 5c.

In contrast to the satisfactory performance of most earth dams, the Coihueco and Lliu-Lliu Dams (Figure 3) experienced significant seismic deformations. Coihueco Dam, which

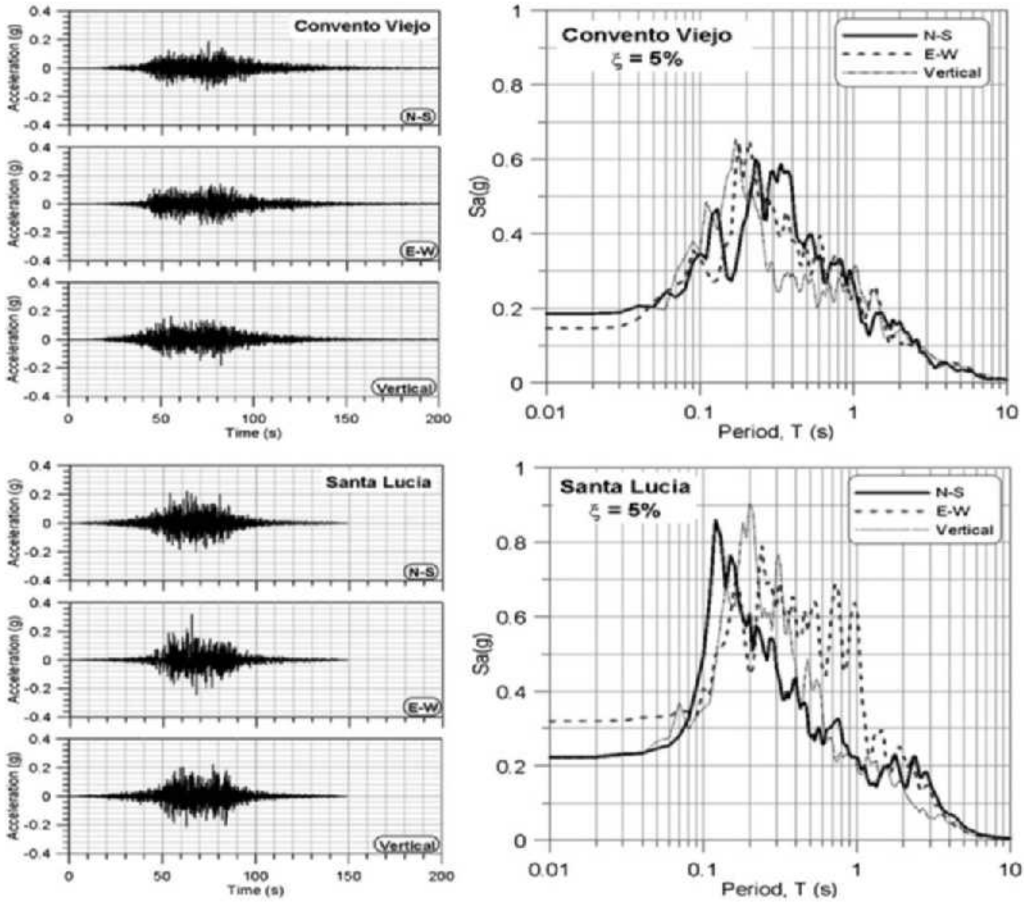


Figure 2. Acceleration-time histories and 5% damped elastic acceleration response spectra recorded at Santa Lucia and Convento Viejo rock outcrops

was built in 1970, is located approximately 28 km east of Chillán. It is a zoned earthfill dam that has a crest length of 1,040 m long and a maximum height of 31 m. Figure 6 shows the design section of the dam. Following the earthquake, a portion of the upstream slope experienced a failure that generated significant deformations and cracks as shown in Figures 7a and 7b. Maximum vertical displacements of the upstream slope were observed towards the left abutment, reaching values close to 4 m.

To assist in investigating the cause of the observed failure, a limited geotechnical exploration was performed, consisting of two boreholes, SEC-01 and SEC-02, of 25 m and 32 m depth, respectively, and one test pit of 5 m depth (CEC-01). The location of the borehole and test pit logs are illustrated in Figure 8 relative to the dam cross section, and the stratigraphic profile with measured standard penetration test (SPT) blow counts (N-values) and fines contents (F.C., percent passing #200 sieve) are shown in Figure 9. The Atterberg Limits

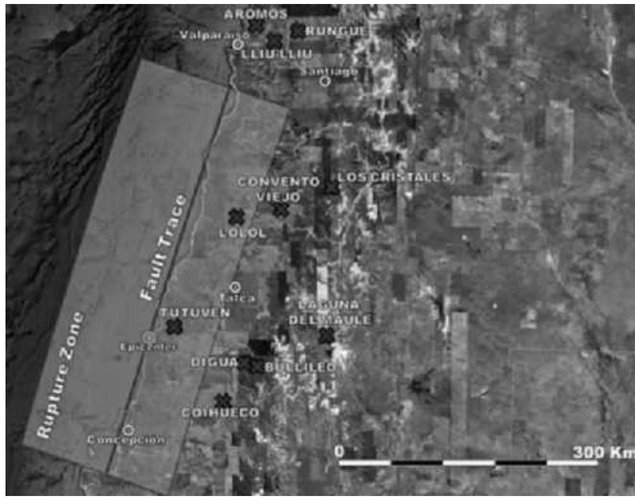


Figure 3. Locations of earth dams (identified by "X" symbols) affected by the 2010 Maule earthquake (background image courtesy of Google Earth).

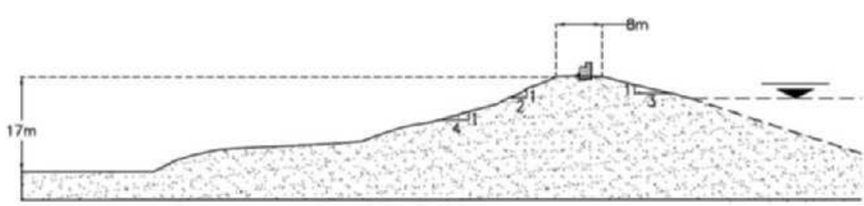


Figure 4. Cross section of Tutuven Dam with no damage.

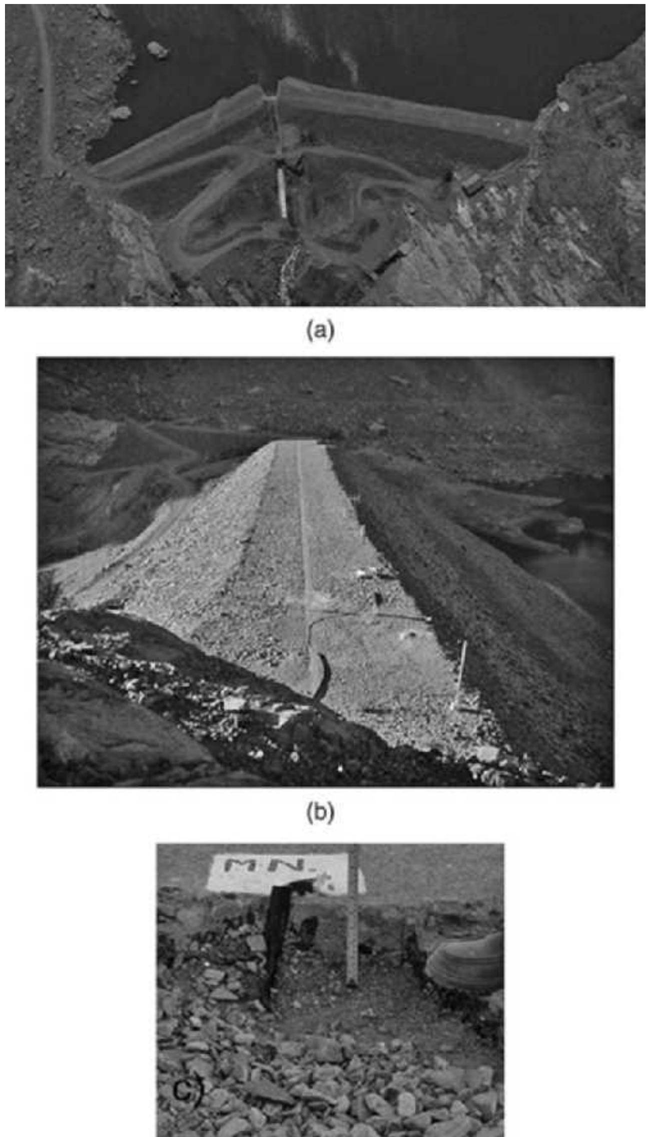
are plotted in Figure 10. The MH soil likely contains allophane, which is commonly present in southern regions of Chile. The foundation materials encountered in the exploration consist of the following:

Layer H1 (~10 m thick): Dark brown silts of high plasticity (MH), low strength and homogeneous structure.

Layer H2 (~2 m thick): Light gray clayey silts (ML), of medium to high plasticity and water content close to the plastic limit.

Layer H3 (~5 m thick): Grayish silty sand (SM) with non-plastic fines. N-SPT values vary from 20 to 100 blows/foot.

Layer H4: Yellowish brown cemented silt (MH). The silty cemented material is interspersed with layers of dense gravel with maximum particle diameter of 60 mm. The N-SPT systematically encountered refusal in this layer.



(a)

(b)

(c)

Figure 5. (a) Plant view of Los Cristales Dam (Google Earth). (b) Photograph taken from left abutment; (c) settlement of the asphaltic concrete core.

It is important to note that based on the experience of the first author with Chilean dams built before the 1960s, project designs were commonly for zoned dams, but the actual as built conditions were closer to homogeneous dams. This seems to be the case of Coihueco Dam also.

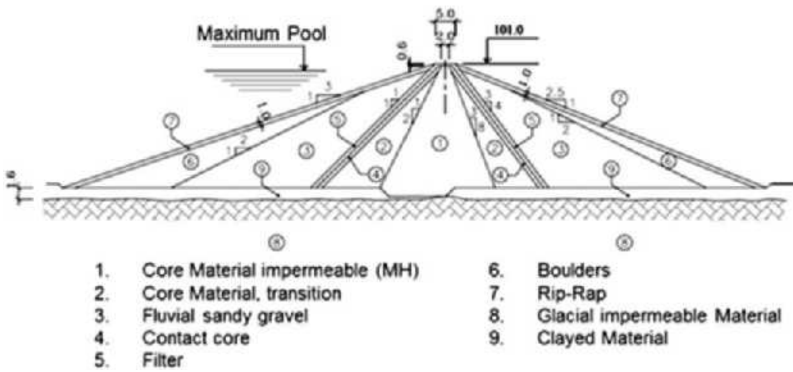


Figure 6. Design cross section of Coihueco Dam (all units in meters).

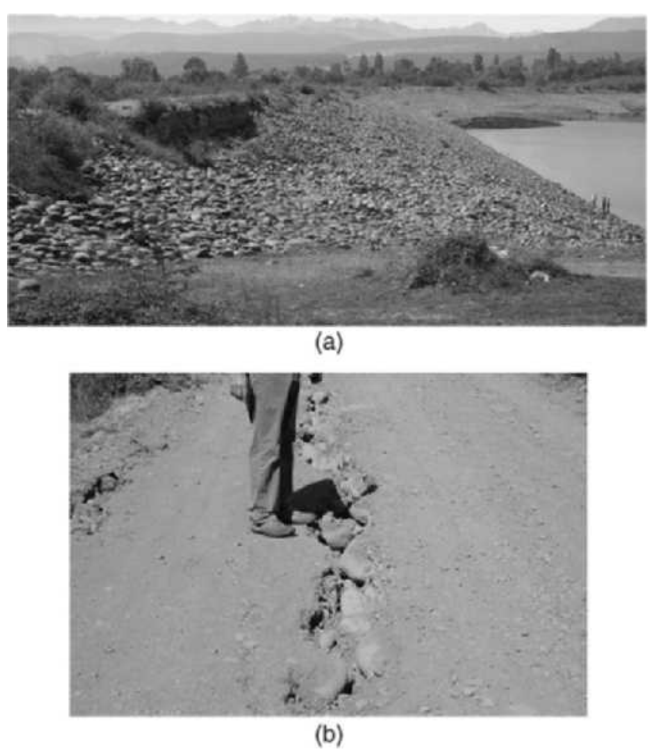


Figure 7. (a) Failure of upstream slope of Coihueco Dam. (b) Longitudinal crack along crest.

Isotropically consolidated, undrained triaxial tests with porewater pressure measurements (CIU) were performed on "undisturbed" samples retrieved with Shelby tubes. The stress-strain and pore water pressure responses for two samples are presented in Figures 11 and 12, corresponding to materials from pit CEC-01 (depth 4.6–5.0 m) and borehole

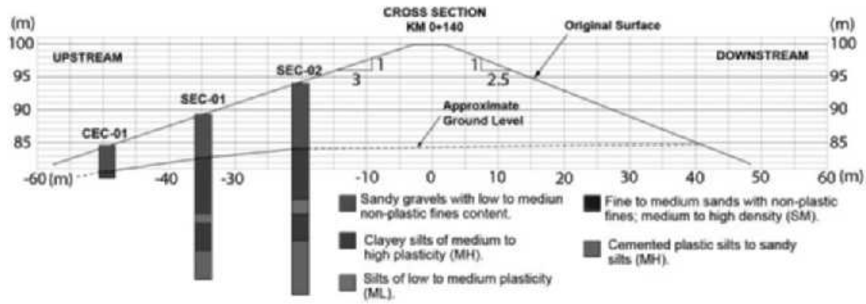


Figure 8. Soil identified in the exploration of Coihueco Dam.

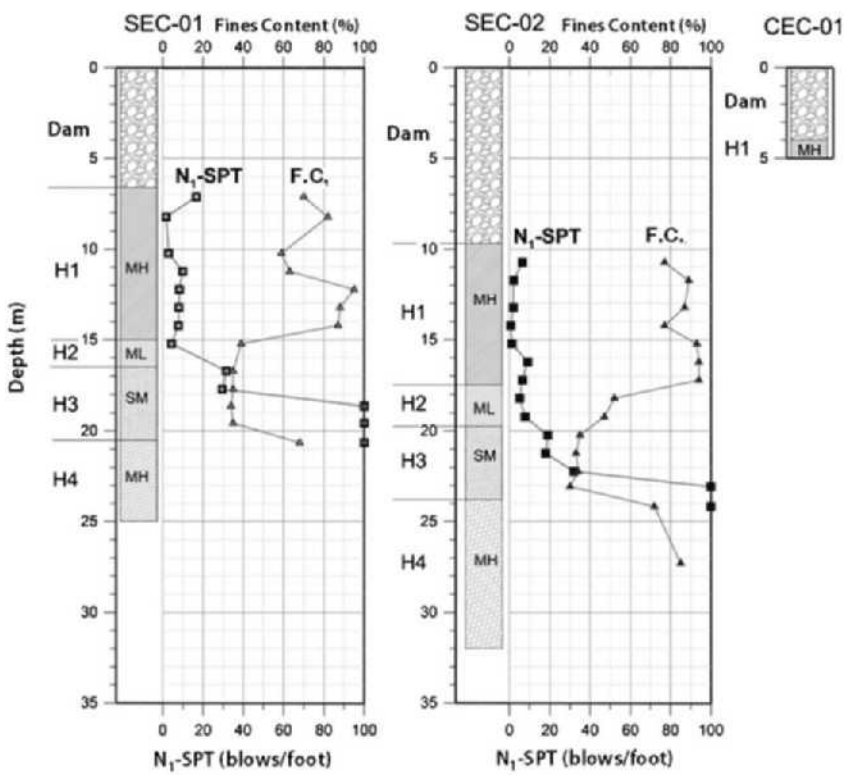


Figure 9. Soil profile at Coihueco Dam.

SEC-02 (depth 15.6–16.0 m), respectively. The results of the CIU triaxial tests show a dilation even under a confining pressure of 3 kg/cm^2 (300 kPa), suggesting that the silty soil is slightly overconsolidated for the range of confining pressure used. Considering the low hydraulic conductivity of this silty soil, its undrained response and thus use of undrained strength (Su) is appropriate for seismic stability analysis. Measured undrained strengths

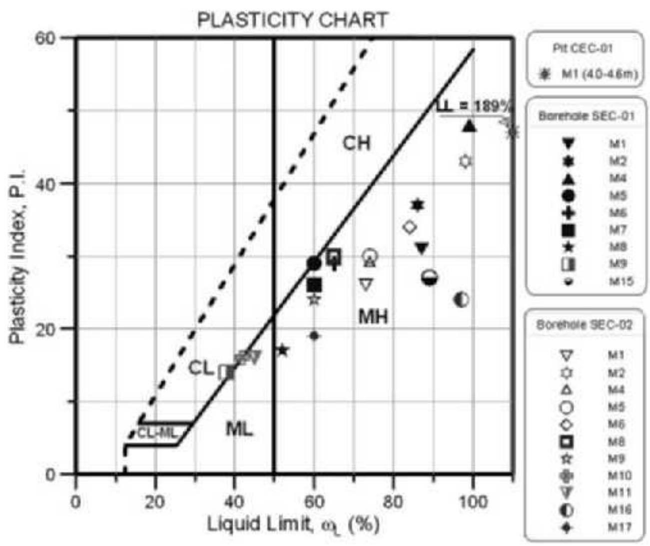


Figure 10. Plasticity of samples retrieved from Coihueco Dam.

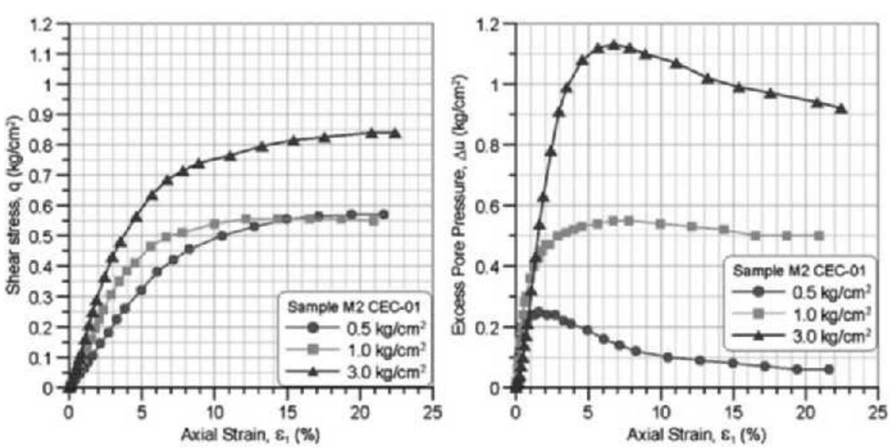


Figure 11. Triaxial test results, sample M2 CEC01. Coihueco Dam.

are shown in Table 1 and plotted in Figure 13. The geotechnical model adopted for Coihueco Dam based on the available information is shown in Figure 14. The geotechnical properties of these materials used in the model are summarized In Table 2.

A pseudostatic slope stability analysis was performed considering both horizontal (K_h) and vertical (K_v) seismic coefficients, with $K_v = 0.5K_h$. The limit equilibrium slope stability analysis using Janbu's method applied to non-circular surfaces was performed using commercial software. The results show that failure ($FOS \leq 1$) occurs for seismic coefficient:

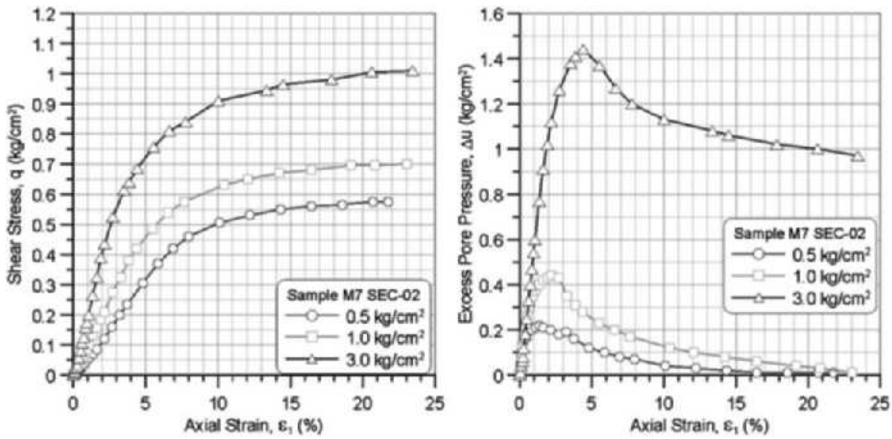


Figure 12. Triaxial test results, sample M7 SEC02. Coihueco Dam.

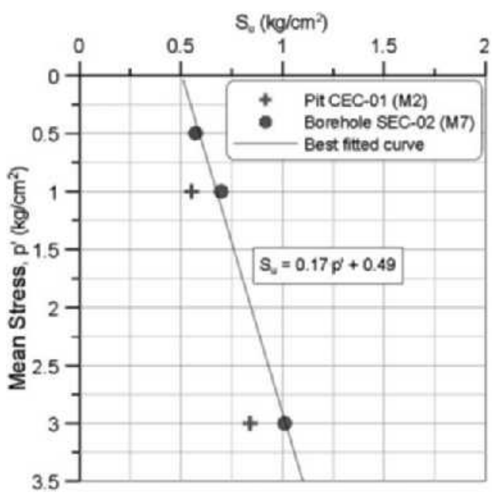


Figure 13. Undrained strength as function of mean initial stress. Coihueco Dam.

$K_h \geq 0.22$ and $K_v \geq 0.11$. 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 earthquakes (i.e., maximum credible earthquakes) on earth structures. The authors are sensitive to the importance of the actual acceleration levels that occurred on the dam and the resulting magnitude of displacements of the soil mass involved in the failure. In this context, the condition of $FOS = 1$ is necessarily associated with the lower boundary of the inertial forces that the dam suffered. Considering the stratigraphy of the dam site, consisting of several layers, including soft MH soils, the use of horizontal time acceleration histories recorded for different soil deposits does not represent the dam site

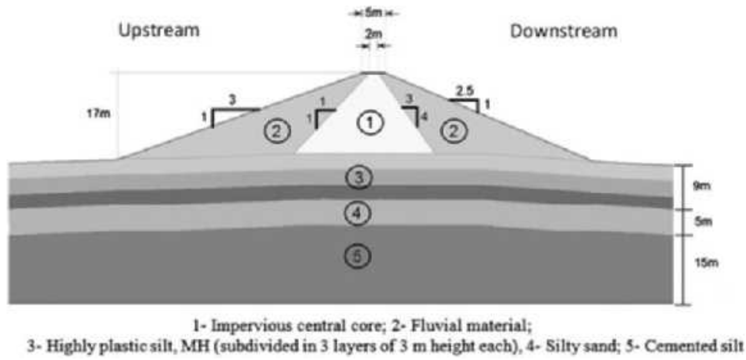


Figure 14. Geotechnical profile used in the stability analysis. Coihueco Dam.

conditions well, and so the classical Newmark sliding block calculation is not presented. Accordingly, the available data only permit the conclusion that the horizontal and vertical seismic coefficients were greater than 0.22 and 0.11, respectively.

In the case of Lliu-Lliu Dam the available information is also quite limited. Two localized seismic slope failures took place in the upstream slope as shown Figures 15. The length along

Figure 15. Seismic slope failures occurred in Lliu-Lliu Dam.

the crest of these failures was 36 m and 21 m, respectively. Additionally, a series of longitudinal cracks along the crest with a maximum lateral openings of 10 cm were observed (Figure 16). Lliu-Lliu Dam was constructed around 1920 and has a crest length of about 500 m and a maximum height of 18 m. The upstream and downstream slopes had similar inclinations of about 1.5:1.0 ($H:V$), and a crest width of 8 m. The 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 bentonite. All the aftershocks, including one of Magnitude 6.9 (11 March 2010), with an epicenter located approximately 160 km south of the dam, did not induce any additional movements of the upstream slope.

SEISMIC PERFORMANCE OF TAILINGS DAMS

Eight major tailings impoundments within the affected region (i.e., Carén, Colihues, Cauquenes, Barahona, Los Leones, Piuquenes, Ovejería and Las Tórtolas) did not exhibit any problems. Carén, Colihues and Los Leones dams are constructed with compacted borrow

Figure 16. Open cracks observed along the crest of Lliu-Lliu Dam.

materials, while Piuquenes, Ovejería and Las Tortolas are tailings dams constructed using the downstream method. Cauquenes Dikes No. 1 and 2 were constructed with cycloned tailings sand using the downstream method, but two subsequent dikes to raise the impoundment were built using the upstream method. Barahona Dike No. 1 is a tailings dam constructed using the upstream method, which experienced a serious seismic failure in 1928, as a result of a breach of the dikes, which allowed tailings to flow towards the valley. This tailings dam was repaired and although it is now abandoned, its crest is part of the route normally used to access part of the mine facilities. It is noteworthy that Barahona Dike No. 1 and Cauquenes Dikes No. 1 and 2 exhibited stable seismic behavior under the strong ground motions generated by the Maule earthquake. In contrast, five other tailings dams constructed using the upstream method experienced varying levels of seismically induced failure (i.e., Chancón, Bellavista Dike No. 1, Veta del Agua Dike No. 5, Alhué, and Las Palmas Dam; Ramirez 2010). The locations of all 13 tailings dams are shown in Figure 17.

The tailings dams farthest from the epicenter that experienced seismic distress were Veta del Agua Dike No. 5 and Bellavista Dike No. 1. However, the distance of these structures to the fault rupture was only 100 km and 120 km, respectively. The other tailings dams that suffered failures have fault rupture distances of 57 km (Las Palmas Dam), 80 km (Alhué Dam), and 100 km (Chancón Dam). Figure 18 compares these distances to the relationship suggested by Conlin (1987) that relates potential for tailings dam failure to earthquake magnitude and epicentral distance.

Bellavista Dike No.1 collapsed and a significant volume of tailings flowed into Dike No. 2, which acted as a buffer, stopping the flow of the tailings (Figure 19a). Dike No. 5 of Veta del Agua experienced a rather limited seismic failure, with the toe of

Figure 17. Large tailings dams with stable response (square) and tailings dams that experienced seismic failures during 2010 Maule Earthquake (circles).

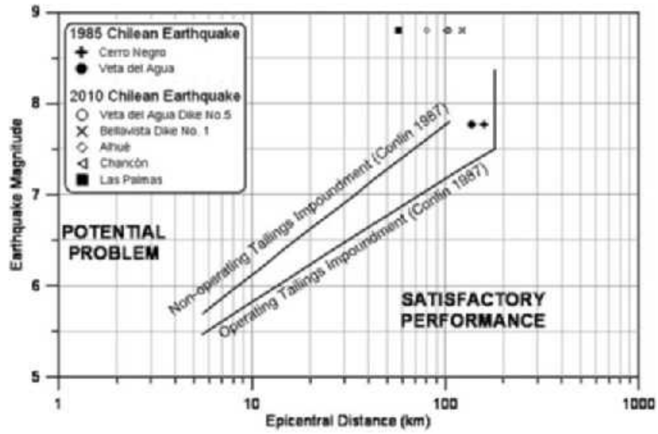


Figure 18. Magnitude and distance to potential for tailings dam failures (Conlin 1987).

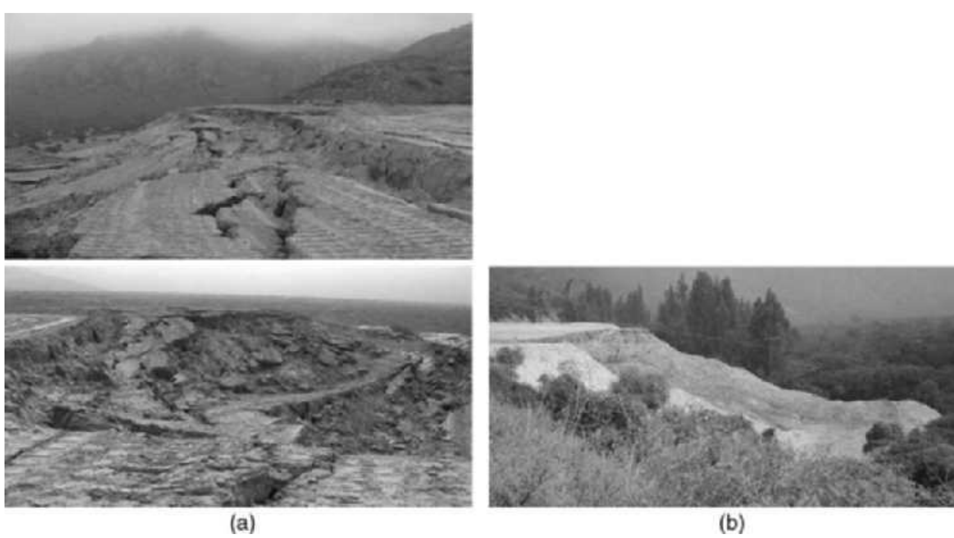


Figure 19. (a) Flow failure of Bellavista Dike No. 1 (Ramirez 2010). (b) Flow failure of Veta del Agua Dike No. 5 (courtesy of Prof. R. Espinace).

the dike flowing for a distance of 100 m, reaching El Sauce Creek. Figure 19b shows Dike No. 5 after the earthquake. Las Palmas tailings dam experienced the most catastrophic seismic failure triggered by the 2010 Chilean earthquake. The mine was operating from the early 1980s 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 earthquake, 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 four people in a house that was buried under four meters of tailings (Figure 20). The tailings liquefied again (less severely) during the aftershocks of Magnitude greater than 5. 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

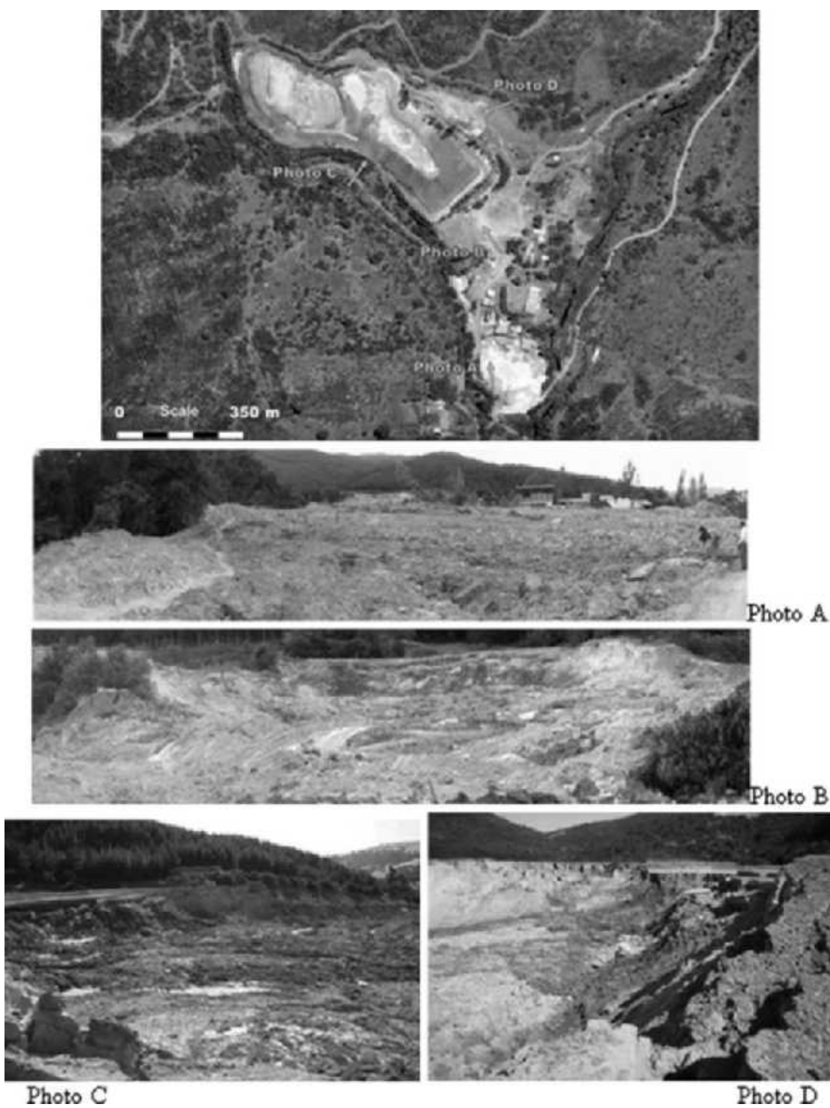


Figure 20. Las Palmas flow failure.

impoundment area even though no full breach of the structure occurred at that location. Field surface wave measurements conducted in the unfailed tailings material indicated shear wave velocity of the order of 250 m/s, and dynamic cone penetrometer soundings performed in both unfailed and failed tailings materials yielded blow counts of about 10 (Bray and Frost 2010).

SEISMIC PERFORMANCE OF EARTHEN LEVEES

Earthen levees generally performed well during the 2010 Maule earthquake. Of the many kilometers of levees surveyed, only two cases of poor levee performance were observed. An earthen levee section west of Colbún failed (as described below), and a minor liquefaction-induced slump occurred along the Bío-Bío River levee north of the Juan Pablo II Bridge's NE abutment in Concepción (S36.814555°, W73.084624°; Bray and Frost 2010).

A 15 m-wide breach of an engineered earthen levee (Figure 21) occurred west of Colbún (S35.698672°, W71.487294°) at 6:50 pm on 13 March 2010, two days after the $M_w = 6.9$ aftershock (epicenter at S34.259°, W71.929°) that occurred on 11 March 2010 at 11:40 a.m. The levee was inspected soon after the $M_w = 8.8$ main shock of 27 February 2010 and after the 13 March 2010 aftershock, and no evidence of distress was noted. The breach produced a localized discharge of water 80 cm high that flooded a few homes. The levee fill material is poorly graded sandy gravel with a significant amount of cobbles that has about 10% low plasticity fines (i.e., GP-GC with cobbles). The levee is approximately 7 m high with a crest width of 6 m, and $1.5H:1V$ side slopes with a cobble facing on the water side. The levee was reported to have been built 25 years ago. Whereas at other locations along the levee, existing material was excavated to build the levee, it is reported that little or no excavation was required at this location because the original ground surface elevation was already low. The actual cause of the levee breach is not clear.

SEISMIC PERFORMANCE OF NATURAL SLOPES

In south-central Chile, the overall geologic setting is largely controlled by the long-term, repeated occurrence of aseismic uplift of the continent, punctuated by sudden, coseismic coastal uplift and inland subsidence. These processes affect the geologic characteristics

Figure 21. Earthen levee breach west of Colbún (S35.698672°, W71.487294°; photograph taken by Mr. Juan Carlos Romo of *El Mercurio* on 12 March 2010; http://diario.elmercurio.cl).

of onshore regions, and thus affect the geotechnical responses to strong ground motion and permanent ground deformation. Regionally, south-central Chile consists of four primary geologic domains (Melnick et al. 2009): (1) the Coastal Platform, consisting of Cenozoic marine deposits and terraces, (2) the Coastal ranges, consisting of Permo-Triassic metamorphic rocks and older granitic rocks, (3) the Central Depression, including Cenozoic volcanic rocks overlain by semi-consolidated and unconsolidated alluvial sediments in the central valley, and (4) the Main Cordillera (Patagonian Andes), consisting of Mesozoic and Cenozoic volcanic rocks. The Coastal Platform is underlain by variable materials that are comparable, as a first approximation, with NEHRP soil classes ranging from type B to C to D; the Coast ranges are probably underlain by NEHRP soil classes B and BC; the Central valley is underlain by classes C and D; and the Patagonian Andes are underlain mostly by classes B and perhaps BC. These relationships suggest that the Central valley, which is an area of long-term sediment accumulation, likely experiences substantial amplification of strong ground motions with respect to the adjacent areas (Bray and Frost 2010).

At the time of the earthquake, most natural slopes were dry or at most partially saturated, which seems to be consistent with the observed conditions and the most likely explanation for the relatively small number of slope failures caused by the earthquake. Nevertheless, significant slope failures occurred on the Arauco Peninsula following a relatively straight line (Figure 22). The material involved in these failures was a young sandstone, with a rather low level of cementation. It is important to note that similar types of material also exist

Figure 22. Massive slope failure occurred in Arauco area.

in other areas where no failures were observed. The particular condition that affected the area of Arauco, where these massive seismic slope failures occurred, is the significant tectonic uplift that reached nearly 3 m near the coast. It is hypothesized that the superposition of the inertial forces of the earthquake plus the distortion generated by the crustal deformation could be responsible for the observed slope failures. It is interesting to remark that many steep slopes in other areas such as those shown in Figure 23 did not fail, and only limited surficial displacements were observed.

RETAINING STRUCTURES

Many different types of retaining structures, ranging from modern mechanically stabilized earth (MSE) bridge abutments and walls to older, cantilever and gravity concrete structures were subjected to strong shaking during the event. The seismic performance of these structures was uniformly good, if not excellent, and no significant problems were reported or observed following the earthquake. Similarly, no basement wall damage of any kind was reported. The rare reports of damage appeared to be the result of construction defects.

MECHANICALLY-STABILIZED EARTH (MSE) WALLS

Mechanically stabilized earth walls have been used quite extensively in Chile for highway embankments, particularly for overpasses and bridge abutments. In the greater Santiago metropolitan area, there are numerous underpasses, overpasses and sidehill fills retained with Reinforced Earth™ walls. Their performance was excellent even when bridges supported by the abutments failed by sliding off their seats (e.g., Figure 24). Similarly, nearly all MSE walls perfomed satisfactorily along the major north–south freeway, Route 5. For example, Figure 25 shows a Reinforced Earth™ abutment of a collapsed bridge across a rail line. At this location, the approach embankment north of the failed railroad overpass also experienced a major failure; however, the abutments themselves showed no signs of appreciable deformation. Interestingly, the adjacent older bridge with multiple supports also performed very well and showed no evidence of damage.

(a) (b)

Figure 23. Steep slopes without evidence of major seismic displacements. (a) Near Barahona tailings dam. (b) City of Constitución.

Figure 24. Intersection of Américo Vespucio Norte and Independencia, Santiago. The Reinforced Earth™ abutment performed well despite bridge collapse.

Figure 25. Reinforced Earth™ bridge abutments at a collapsed rail crossing on Route 5, north of Parral.

CONVENTIONAL CONCRETE GRAVITY AND CANTILEVER STRUCTURES

Conventional concrete gravity and cantilever retaining structures similarly performed very well in many different settings. Figure 26 shows an overpass structure on the coastal highway just south of Coronel, about 140 km south of the epicenter in an area that experienced significant shaking. The structure experienced no distress and the bridges were fully functional after the event.

Importantly, there were numerous bridge approach embankments that were shaken with sufficient intensity to experience surficial slumping and many bridge decks were offset during

Figure 26. Multiple retaining structures for highway overpass structures south of Coronel.

Figure 27. Surficial sloughing on a Route 5 overpass bridge approach embankment north of Chillán, with the abutment showing no evidence of damage.

shaking with concrete seats that were spalled or completely fractured, while the concrete abutment retaining walls performed very well and showed no evidence of damage. Figure 27 shows one such bridge approach embankment and abutment wall on Route 5 north of Chillán. Here, the approach embankment at one end suffered a surficial slump and the bridge deck had temporary scaffolding supporting it at the other abutment as it was close to sliding off its seat. In contrast, the abutment walls showed no evidence of distress.

A rare failure of the uppermost section of a reinforced concrete retaining structure was observed in San Pedro, about 110 km south of the epicenter. The wall (Figure 28) was located within 3 km of the south approach to the Llacolen Bridge the Bío-Bío River connecting Concepción and San Pedro. The 16-inch thick wall was reinforced vertically with No. 3 or No. 4 bars on 12-inch centers. Failure occurred at the uppermost "cold joint," where

Figure 28. Failed reinforced concrete retaining structure in San Pedro. Failure occurred along a concrete "cold joint" and showed complete corrosion of the reinforcements (right photo).

the top wall section toppled to the east apparently due to inertial forces. The reinforcement at this cold joint was badly corroded (Figure 28) and likely was the primary cause of the failure. The granular backfill is still clearly self-supporting.

CONCLUSIONS

The maximum peak acceleration recorded on rock outcrop reached 0.32 g at the station Santa Lucia located approximately 105 km northward to the fault trace (from its northern most point), while the maximum peak acceleration recorded on a natural soil deposit reached a value of 0.92 g in Angol City, that is located 80 km southward to the fault trace (from its southern most point). The main earthen structures that were visited and analyzed all have distances in the range of 0 km to 150 km from the fault zone.

Overall, given the large number of earth structures including earth dams, tailing dams, and levees, as well as the topography of the region, the number of failures of man-made and natural slopes was remarkably low for the intensity and duration of the event. In part, this good seismic performance can be attributed to the fact that the earthquake occurred during the dry season and many of the slopes were relatively dry and reservoir levels were low. Nevertheless, several smaller tailings dams constructed using the upstream method experienced various levels of poor performance, including one tailings dam flowslide that caused four fatalities. These tailing dam failures support the contention that the upstream construction method should not be used in a region with high seismicity.

The seismic performance of large dams in the region was good, with only two important dams experiencing some distress: the Coihueco and Lliu-Lliu Dams. Testing and analysis of the upstream slope failure of Coihueco dam provided results consistent with observations made after the earthquake. A large number of small, unengineered dams built by farmers for irrigation suffered extensive damage due to liquefaction, slope failures, and settlement. Fortunately, no catastrophic water releases were reported, because reservoir levels were low and the heights of the dams were rather small. However, the earthquake clearly showed that the design and construction of these types of private dams can be inadequate and poses a potential future risk. Only one earth levee section failed and released water, but flooding damage was minimal in this agricultural region. Lastly, the performance of a wide range of retaining structures was uniformly excellent even when adjacent structures or embankments experienced significant distress.

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REFERENCES

Boroschek, R., Soto, P., and León, R., 2010. *Registros del Terremoto del Maule, Mw=8.8, 27 de Febrero de 2010*, RENADIC Report 10/05, August 2010.

Bray, J. D., and Frost, J. D., Eds., 2010. *Geo-Engineering Reconnaissance of the 2010 Maule, Chile Earthquake*, a report of the NSF-sponsored GEER Association Team, primary authors: Arduino et al., available at http://www.geerassociation.org/.

Conlin, B., 1987. A review of the performance of mine tailings impoundments Under Earthquake Loading Conditions, in *Proceedings, Vancouver Geotechnical Society Seminar on Earthquake Geot.*, Canada.

Melnick, D., Bookhagen, B., Strecker, M. R., and Echtler, H. P., 2009. Segmentation of megathrust rupture zones from fore-arc deformation patterns over hundreds to millions of years, Arauco peninsula, Chile, *J. Geophys. Res.* **114**, B01407, doi:10.1029/2008JB005788.

Ramirez, N., 2010. Effects of the 2010 earthquake on tailings disposals located on the South-Central Chile and its relation with Decreto 248, in *Seminar: Proposals for the Operation of Tailings Disposals According to Recent Experiences* (in Spanish).

U.S. Geological Survey (USGS), 2011. *Magnitude 8.8. – Offshore Bio-Bio, Chile*, available at http://earthquake.usgs.gov/earthquakes/recenteqsww/Quakes/us2010tfan.php#maps.

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