

A review of the performance of two large sub-stations and eight large dams during the Chi Chi Taiwan earthquake

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**Report to
The Institute for Catastrophic Loss Reduction¹
by
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1 Introduction

Soon after 1:00 a.m. local time on September 21, 1999, a magnitude 7.6 earthquake struck Taiwan, the strongest earthquake to hit Taiwan during the 20th century. More than 2,200 people were killed as a result of the earthquake, known as the Chi-Chi Earthquake, and more than 8,000 people were injured. In addition to the thousands of buildings that were damaged, reports indicated an extensive loss of electrical power due to damage to a large substation and other electrical facilities and the “failure” of a concrete dam. The substation damage showed the vulnerability of an electrical grid to earthquakes and the dam failure would have been the first time a concrete dam had failed anywhere in the world as a result of an earthquake. In addition, several other large concrete and embankment dams were affected.

We were fortunate in being able to go to Taiwan in November to observe the residual effects of the earthquake on the central substation at Chungliiao, eight dams and the Mingtan substation and underground powerhouse. Staff from the Taiwan Water Conservation Agency and Taiwan Power briefed us and escorted us to eight dams, including the damaged Shih Kang concrete dam. It was a prime opportunity to assess the actual situation at the substation, place this particular dam failure into perspective, and to observe the successful performance of several other dams.

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2. The Earthquake Location and Details

The main event of the “Chi Chi” earthquake occurred at 01:47 Taiwan time on 21 September 1999. The magnitude was assessed at $M_w = 7.6$ at a depth of 7 km with an epicenter near Chi Chi, 160 km south of Taipei and close to the major city of Taichung and several of the dams the authors visited as shown in **Figure 1**. Several aftershocks were felt with magnitudes as large as $M_w = 6.8$.

The earthquake had a thrust fault mechanism and occurred on the known Chelongpu fault. The fault rupture was up to 105 km long with vertical offsets of 2 to 3 m, as shown in **Figure 2**, and up to approximately 10m in some locations, see **Figure 3**. Near fault peak ground accelerations (PGA) were up to approximately 1.0g. The trace of the rupture is shown in **Figure 4** and includes a bend to the Northeast, with multiple thrusts at the northern end.

A variety of concrete and embankment dams of different types, heights and ages were subjected to strong shaking. Two months after the earthquake, the authors visited eight dams in the epicentral region (**Table 1** and **Figure 1**) to review their performance with the dam owners. Other dams in the region were also shaken but were not visited because of time constraints and limited road access.

2.1 Chungliao Substation

The Chungliao substation is located in central Taiwan as shown in **Figure 5**. This substation is a key station at the intersection of several major high voltage transmission lines in the Taiwan grid. All the 345kV circuits from southern and central Taiwan run through the Chungliao substation. These circuits carry about half of the power from the central and southern region power plants to northern Taiwan.

The substation was located very close to the epicenter. PGA values were estimated to be about 0.5g or more based on preliminary maps. Much of the substation was constructed on a cut and fill base, as shown in **Figure 6**.

Extensive shaking-related damage occurred to station equipment including transformer bushings, live tank circuit breakers, porcelain and rigid buswork as shown in **Figures 9** and **10**. More serious damage, however, was caused by a landslide that developed under the substation. The southern end of the station is constructed on fill that was placed to provide level ground on a natural hillside. Part of the fill moved laterally, displacing at least half the station and several transmission towers by about 1m, as shown in **Figure 7**. The back scarp of the slide passed directly through the station, tearing apart rigid buswork, displacing and rotating major equipment, and severing cable ducts. Some SF6 bus pipes were displaced, their footings were dragged and rotated, and couplings were buckled as shown in **Figure 11**, although the pipes were

not ruptured. The slide scarp also cut directly through the station access road. Liquefaction of the fill likely was a contributing cause of the slide, as there was at least one local area with sand boils within the slide area as shown in **Figure 8**.

There was differential vertical ground displacement of about 1m adjacent to the one-story substation control building, as shown in **Figure 12**, although the building itself was not significantly damaged. Inside the building it was interesting to note the performance of control equipment. One battery rack had no end rails, as shown in **Figure 13**, and the batteries reportedly toppled out of the rack during the earthquake. Another rack had end rails, as shown in **Figure 14**, and performed well. Damage to equipment cabinets was minimized because cabinet base anchors had limited movements as shown in **Figures 15** and **16**, although it was reported that some cabinets had toppled due to minimal anchorage.

2.2 Shih Kang Dam

The most spectacular damage to a dam occurred at Shih Kang Dam, the downstream dam of five on the Tachia River. The dam is a 25m high, gated spillway structure as shown in **Figure 17**. This dam retains a 2.7 million m³ reservoir that is one of the main sources of water for Taichung. The structure consists of a low concrete roadway, 18 spillway bays, 2 sediment sluice bays and a concrete bridge deck. Downstream of the dam is a concrete stilling basin apron and energy dissipation structure.

Shih Kang Dam is located near the north end of the Chelongpu fault. Near this location the surface fault rupture turns to the northeast. Several fault strands and very large surface displacements characterize the rupture zone. The main fault rupture is about 300m north of the dam. One strand of the fault rupture passed directly through the dam, rupturing the three spillway bays closest to the right end of the dam. On the left abutment another fault strand cut the diversion tunnel that directs the water to the two treatment plants. It was reported that no faults had been identified in the dam foundation during construction in the 1970s, and the strand through the dam was considered to be a “first time” rupture.

Figure 18 shows a view of the damaged spillway sections that were located directly over the fault rupture. Most of the dam is on the hanging wall of the fault zone and was uplifted by 9.8m. The footwall was also uplifted, but only by 2.1m, resulting in a differential vertical displacement of 7.7m at the ruptured section of the dam. In addition, the entire left side of the dam and its foundation was laterally shifted about 6m cross-valley and about 2m upstream. There was no strong motion instrumentation at the dam, however a PGA of 0.56g was reported in the town adjacent to the dam.

Some cracking and minor displacements occurred along horizontal construction lift joints in the roadway monoliths close to the fault rupture, as shown in **Figure 19**. Other than the section overlying the fault rupture, the remainder of the dam survived remarkably well. Diagonal cracks

that appear to have steep dips occurred in several rollway monoliths, as shown in **Figure 20**, and in some cases continued up into the piers. Significant cracking was observed in only one pier between adjacent bays as shown in **Figure 21**. It was reported that one of the two sluice gates and six of the 18 spillway gates were operable following the earthquake. An example of damage to a spillway gate bearing is shown in **Figure 22**. Except for the gates in the three destroyed bays, all other gates had either been repaired or could be repaired with limited work.

The bridge deck on the dam showed a consistent pattern of right lateral offsets of up to about 150mm at the deck joints as shown in **Figure 23**. This pattern suggests clockwise rotation of the deck, but the bridge deck movements may not be representative of the dam movements.

On the downstream side of the dam, there were several left lateral offsets in the sluice bay walls as shown in **Figure 24**. This indicates that there were probably local differential movements within the rock mass of the hanging wall. In the concrete stilling basin, there were other interesting examples of differential displacements. For example, the concrete slab on one side of a vertical construction joint was severely buckled, but the slab on the opposite side of the joint was intact as shown in **Figure 25**. At another location where two orthogonal vertical joints intersected, adjacent slabs had shifted to leave a square opening at the intersection. Waterstops were ruptured in many locations. It is believed that the stilling basin concrete had been placed directly on rock, and the differential displacements imply that substantial debonding of the concrete from the rock had occurred.

From a dam safety perspective, the rupture and collapse of the three spillway bays, plus other local damage to the dam, had minimal downstream consequences. Most of the reservoir basin was uplifted with the hanging wall of the fault, and the entire reservoir was released through the damaged section of the dam. However, the damaged concrete bays and spillway gates remained in place and constrained the release to an estimated rate of about $200\text{m}^3/\text{s}$. It was reported that the reservoir drained over a period of about 3 hours, and that the maximum downstream river rise was about 1m. This rate of release is substantially less than spill releases during the monsoon season, so the river remained within its channel and there was no downstream damage or loss of life.

Other than the damage to the dam, the most serious effect was the rupture of the water supply penstock from the reservoir to the water treatment facility. Because of its importance for city water supply, the owner implemented a crash program to repair the penstock. The owner also intends to repair the dam and return it to service, at least for the near future. By the time of the site visit, diverting some water from the upstream river channel to the water treatment plant via existing irrigation channels had restored partial water supply. Repairs to the fault-damaged tunnel were also in progress. Various options were being considered to deal with the damaged section of the dam. We suggested that a “roller compacted dam” could be considered, since it would be quick to build and, due to its mass, would be very robust in the event of additional movements. One interesting consideration is that some of the local residents are not in favor of removing the damaged section to allow its replacement because the dam has become a tourist attraction.

2.3 Liyutan Dam

The reservoir retained by Liyutan Dam is another important source of water for Taichung. Although the dam was completed in 1992, the reservoir will not be filled to full pool level until a diversion project, that will provide additional water, is completed. At the time of the earthquake, the reservoir was about 31m below its target normal maximum level.

Liyutan Dam is shown in **Figure 26**, and has a wide central impervious core, a downstream shell of rockfill and an upstream shell of gravel, plus filters between these zones. A 40m high cofferdam is incorporated into the upstream toe. The dam is well instrumented with digital strong motion accelerographs (SMA's), piezometers, inclinometers, strain gages, earth pressure cells, weirs and survey monuments. Strong motion records were obtained by 7 SMAs at several levels of the dam and its foundation, with typical accelerations as summarized in Table 2. The data clearly indicate that ground motions were significantly amplified between the base of the dam and the crest.

The crest of the dam settled from about 50mm to 80mm. On the upstream slope, measured settlement ranged from about 56mm near the reservoir level up to a maximum of 96mm near the crest. On the downstream slope, measured settlement was about 50mm to 70mm on the upper slope, about 25mm to 35mm near midslope, and about 20 mm on the lower slope. The upstream slope, the crest, and the upper portion of the downstream slope moved upstream by up to approximately 60mm. The lower portion of the downstream slope moved downstream by approximately 50mm. One possible explanation for this deformation pattern is that the cofferdam in the upstream toe may not have been as well compacted as the rest of the dam, which could cause the dam to preferentially settle and deform towards that area. The different properties of the upstream and downstream shells may also have influenced the deformation pattern.

There was minor opening and closing of several expansion joints in the crest sidewalks. The only significant crack was a transverse crack across the crest road close to the left abutment. A test pit found that the crack was only a few meters deep and formed at the location where the bedrock foundation profile steepens. This crack probably resulted from differential settlement of the embankment fill at this point.

Several piezometers in the core of the dam showed increases of about 1m to 4m of head. Seepage from toe drains temporarily increased by about 20 percent then returned to normal over a one-week period. Seepage from the grout gallery increased from about 8 l/min to 30 l/min, then decreased over the next several weeks.

Based on a cursory visual inspection, the concrete labyrinth spillway on the left abutment shown in **Figure 27**, and a bridge across the spillway chute did not appear to have experienced any damage.

2.4 Mingtan Pumped-Storage Power Station

The Mingtan pumped-storage project is located about 15km east of the earthquake epicenter. The major features of the project are the upper storage reservoir (Sun Moon Lake), the lower reservoir, two penstocks and a six-unit 1,600MW underground powerhouse. Sun Moon Lake is a natural lake that was raised in 1934 by the Japanese with the construction of the Shui Shih and Toushuh embankment dams. The Mingtan concrete gravity dam retains the lower reservoir. The penstocks connecting the two reservoirs are mostly located underground, except at one location where they cross a river valley on two 130m long concrete bridges. Based on preliminary peak ground motion contour maps, all of these structures experienced PGAs estimated to be in the range of 0.3g to 0.4g.

Mingtan Dam is a modern concrete gravity dam with a high, steep, heavily supported rock face above the left abutment as shown on **Figure 28**. The reservoir was near full pool level and the powerplant was operating at the time of the earthquake. Following the earthquake, the reservoir was lowered by about 14m as a precautionary measure.

There were no visible signs of damage to the dam, and all the radial gates, as shown in **Figure 29**, were tested and found to be fully functional. A tall rail-mounted gantry crane on top of the dam, as shown in **Figure 30**, was reported to have been locked to the rails at the time of the earthquake and was undamaged. Some foundation drainholes were found to be blocked, and foundation piezometric pressures recorded in the dam gallery increased above a predefined acceptable level in one section of the dam. Drilling of about eight new drainholes from the gallery was in progress at the time of our inspection, and this work was apparently lowering the piezometric pressures back to acceptable levels.

Several cracks were observed in an unreinforced concrete apron that forms part of the channel protection immediately upstream of the dam, however that minor damage was of no concern. A four-story reinforced concrete site office building had a number of cracks on inside walls but was still in service.

The 345kV SF6 substation located at the toe of the dam, as shown in **Figure 31**, was reported to be undamaged.

The underground powerplant, as shown in **Figures 32** and **33**, was in operation at the time of the earthquake and was undamaged although there was a loss of power supply and lighting underground. Several people were working in the plant at the time and apparently felt only minor shaking. It was reported that several lunch boxes were stacked against a wall and remained in place through the ground motion. This is consistent with the notion that seismic ground motions at depth are less than those at surface.

At the penstock river crossing, as shown in **Figure 34**, a strong motion instrument located under a penstock at the mid-span of one bridge recorded PGAs of 0.48g (cross valley), 0.63g (upstream/downstream) and 0.60g (vertical). Each of the 6.8m diameter penstocks has an expansion coupling at one end of the river crossing. On both penstocks, all six of the keeper lugs at the expansion coupling were bent due to excessive longitudinal strain as shown in **Figure 35**. However, the couplings did not fail and the penstocks were quickly returned to service after the bent lugs were replaced. The only other damage observed at the river crossing was minor cracking of a few concrete penstock footings located on the bridges.

2.5 Shui Shih Dam

Sun Moon Lake is a popular tourist area and the local highway traverses the crest length of Shui Shih Dam, one of the two embankment dams retaining the lake. The dam has a simple cross-section consisting of a clay core with a central reinforced concrete core wall, and upstream and downstream shells. The upstream slope protection consists of a masonry layer of boulders and cobbles set into concrete or cement grout, with horizontal and vertical construction joints. The dam foundation consists of gravel overlying shale bedrock; the concrete core wall penetrates into the gravel, but not to bedrock along its entire length.

A survey indicated that Shui Shih Dam experienced a maximum settlement of about 13cm, which could be visually observed just below the downstream side of the crest where the dam settled around a steel piezometer pipe. Seven longitudinal cracks formed in the dam, with three cracks along the crest, three along the upstream slope, as shown in **Figure 36**, and one along the downstream slope. The cracks on the upstream slope were typically expressed by widening of construction joints in the masonry slab. These cracks are interpreted to be caused by settlement and shallow slumping of the shells. Two of the longitudinal crest cracks were near the centerline of the paved road, and appear to be located directly above the edges of the concrete core wall.

After the earthquake, there was an unusual bulge in the middle of the crest pavement near the center of the dam as shown in **Figure 37**. Subsequent excavation revealed that the bulge was due to settlement of the dam around two 400mm diameter vertical pipes located just below the road surface. The pipes were likely related to the original construction of the concrete core wall.

No change in seepage downstream of the dam was reported, however, any changes in seepage through the foundation gravel layer would be difficult to detect. It would be interesting to know how the concrete cutoff wall performed during the earthquake, and whether any cracks were developed at the foundation contact. Access for inspection of the concrete cutoff wall is clearly difficult, and because any increased leakage that may be occurring would be in the foundation, we really don't know how well it performed.

Following the earthquake, the cracks in Shui Shih Dam received substantial local media attention. Sun Moon Lake was temporarily lowered several meters as a precautionary measure while the damage to the dam was investigated, and all of the cracks were infilled

2.6 Toushih Dam

Toushih Dam is of similar construction to Shui Shih Dam, except that it is founded on sandstone. One difference is that the original upstream slope protection consists of a reinforced concrete grid infilled with crushed rock, and covered with concrete blocks. About 10 years ago, a large volume of spoil materials excavated from the penstock intake was dumped to form a berm that has covered the upstream slope protection.

Following the earthquake, a maximum settlement of 22cm was measured on the dam crest as shown in **Figure 38**. Several longitudinal cracks 10cm to 50cm wide formed in the upstream berm, which was probably not well compacted when it was placed. It was reported that seepage on the downstream side of the dam carried some fine sediment for one day after the earthquake, but there was no increase in seepage volume. By the time of the site visit, all of the cracks in the berm had been infilled. One open hairline crack about 8m long was observed in the soil on the crest of the dam.

2.7 Techu Dam

Techu Dam is the upstream dam of five dams on the Tachia River, and is located in a narrow steep-sided canyon as shown in **Figure 39**. The main access to the dam from Taichung via Highway 8 along the river, was still out of service two months after the earthquake due to major rockfalls and bridge damage. Alternate road access to the dam was from Puli in the south via Highway 14, over a 3400m high mountain pass.

The reservoir was near full pool level at the time of the earthquake and was lowered several meters as a precautionary measure. The dam is instrumented with SMAs at several levels on the dam and its bedrock abutments, however due to loss of power at the time of the main earthquake, only one strong motion record was obtained. An SMA located in an instrument gallery near the top of the dam, just below the crest spillway as shown on **Figure 40**, recorded a PGA of 0.87g. Based on preliminary peak ground motion contour maps, the bedrock PGA is estimated to be in the range of 0.2g to 0.5g. Numerous aftershocks were recorded, both by this instrument and another SMA at the base of the dam, and may provide useful information for back-analysis of amplification effects in the dam.

Walkways at several levels provided good access to the downstream face of the dam. There was no visible damage to the dam, and no evidence of cracking at the vertical joints on the downstream face. However, it was reported that some joint meters, installed in the concrete about 0.5m in from the upstream and downstream faces of the dam, indicated small displacements. On the bridge above the crest spillway, concrete at the ends of the roadway construction joints was cracked as shown in **Figure 41**. The cracks suggest compression caused by upstream-

downstream flexing of the bridge but the bridge deck may have moved independently of the dam. It was reported that the spillway gates below the bridge were tested with no problems.

A multistage pendulum showed no permanent deformation of the crown cantilever; this type of instrument does not provide information about the behavior of the dam during the earthquake. Seepage measured in the lower dam galleries and sump temporarily increased, and then began to decrease on its own.

Several concrete buildings at the site had numerous cracks but were still in service. At the entrance to the site, a small building, as shown in **Figure 42**, consisting mostly of a heavy concrete roof supported on a central column, experienced enough deformation to break all of the large windows and twist the metal window frames.

Asphalt pavement on access roads at the site was cracked at several locations. Rockfalls damaged the access road to the powerhouse and the road was still closed due to the hazard. Along the steep mountain slopes near the dam, there were numerous rockslides, as shown in **Figure 43**, in the reservoir immediately upstream of the dam, and the one on **Figure 44** downstream. Many local sections of the highway were partially destroyed or undermined.

2.8 Jenyitan and Lantan Dams

Jenyitan and Lantan Dams impound reservoirs that supply municipal water to the city of Chiai and are located only about 3km apart. Not only were these dams shaken by the Chi Chi earthquake, they were more strongly shaken by an M6.4 earthquake that struck southern Taiwan on 22 October 1999. A peak shock recorder installed in the administration building adjacent to Jenyitan Dam recorded PGAs of 0.22g N-S and 0.26g E-W for the Chi Chi earthquake. PGAs of 0.69g N-S and 0.69g E-W were recorded for the 22 October earthquake followed by PGAs of 0.64g N-S and 0.24g E-W for an aftershock less than one hour later. From 21 September to 08 November, this instrument recorded about 360 aftershocks. Most were less than 0.01g, but there were three events with PGAs between 0.05g and 0.10g, and seven events between 0.10g and 0.20g.

Jenyitan reservoir was near full pool level at the time of both earthquakes. The main portion of the dam consists of a zoned embankment with a wide, central impervious core as shown in **Figure 45**. The only damage observed in this portion was a narrow longitudinal crack along the downstream side of the paved crest road as shown in **Figure 46**. Some settlement and a temporary increase in seepage were reported in this portion, but full details were not available.

The right end of the dam near the spillway had several significant longitudinal cracks and a maximum settlement of about 220mm. A 130m-long crack up to 160mm wide along the downstream side of the pavement on the crest, had been temporarily sealed with concrete as shown in **Figure 47**. On the upstream slope of the dam, there was a local slump to the right of the

spillway, and several open longitudinal cracks from 80mm to 350mm wide to the left of the spillway as shown in **Figure 48**. This upstream slope area was reported to have been underwater at the time of the Chi Chi earthquake. There were other local cracks in the crest pavement and sidewalk, and local offset joints in the upstream concrete parapet wall. Except for minor local cracking at the top of one wall, the concrete spillway chute and the single vertical lift spillway gate appeared to be undamaged. It was not clear which of the earthquakes had caused the various observed damage.

Apparently, the right end of Jenyitan Dam consists of a low embankment founded on native soil. It is interpreted that the cracking and settlement in this portion of the dam were due to settlement of the foundation soil.

It was interesting to observe the pedestal footings used for the transmission tower foundations in the Jengitan reservoir as shown in **Figure 49**. These appeared to have performed well at this site, although we had no specific information. There were reports that similar towers located on steep slopes or near the surface fault rupture had been damaged but we did not see these.

Lantan Dam is another embankment dam constructed by the Japanese, with the typical reinforced concrete core wall that appears to have been locally favored at the time of its construction. The only damage observed in a brief inspection of this dam was a longitudinal crack about 50m long and up to 12mm wide, along the centerline of the paved crest road as shown in **Figure 50**. The location of the crack appears to match the location of the concrete core wall, the top of which is about 6m below the dam crest. A small test pit was in progress at the time of the site visit, and the crack could be seen extending to at least 0.6m deep in the gravelly road base under the pavement.

3. Observations

1. It is clear that the lack of redundancy in the major transmission grid made much of Taiwan vulnerable to an earthquake close to the central substation at Chungliiao. This also limited the ability of the grid to quickly restore power to customers.
2. Substation damage due to strong shaking at Chungliiao was typical of that experienced in other major earthquakes. The landslide is a classic case in a cut and fill site, and may have been predictable. The damage caused by the landslide was costly and time consuming to repair and is indicative of the damage that could occur at level ground sites subject to liquefaction induced lateral spreading. The benefits of anchoring and bracing control equipment and batteries were demonstrated. The SF6 substation at Mingtan, which was subjected to severe shaking but no apparent ground displacement, was undamaged.
3. The rupture of Shih Kang and water supply penstock was due to unexpected fault offsets directly through the dam and penstock. The main fault 300 meters away was known, but

we believe that anticipating the branches was not possible. However, even if the branches had been known, no practical design would have been possible to deal with the problem at this location.

4. The Shih Kang structures were subjected to a PGA of about 0.56g and, aside from the end bays and stilling basin, the decks, piers, gates and monoliths survived very strong vibratory ground motions reasonably well and are repairable at moderate cost. The performance of these structures supports the record of robustness of well-designed concrete structures. The extent of repairs required to the stilling basin is less obvious, and is related to decisions on the operation of the facility.
5. The Liyutan embankment dam is a well-designed and well-instrumented dam. The reservoir was only partially full at the time of the earthquake and it experienced some small permanent deformations, however it performed well during the earthquake. Back analyses of the behavior of this dam would provide a valuable case history.
6. The good performance of the Mingtan concrete gravity dam is also encouraging. We are proposing that a back analysis be performed to better understand the design performance.
7. The behavior of the Mingtan penstock couplings warrants more investigation. Large dynamic displacement of such items clearly needs to be accommodated within the detailed design.
8. The lack of damage in the underground powerhouse at Mingtan is noteworthy. Further examination of this experience may be worthwhile.
9. The Shui Shih and Toushuh embankment dams at Sun Moon Lake are older dams but performed acceptably. Some deformations and cracks occurred but were repaired quickly. It would be interesting to know more about the performance of the concrete cutoff walls, however because access is very difficult this is unlikely.
10. Tchi Concrete Arch Dam appears to have survived the earthquake very well. Better estimates of the bedrock ground motion are needed to understand the good performance of this dam. The loss of power to SMA's during the earthquake highlights the importance of backup power supplies for such instrumentation. We are suggesting that some back calculations would be worthwhile if the ground motion estimates can be tightened.
11. Both the Chi Chi earthquake and a major aftershock shook the Jenytan and Lantan embankment dams. The main section of the Jenytan dam performed very well. The deformations and repairs were limited to the right end abutment section and its more complex interface with the original foundation. The Lantan dam suffered a minor longitudinal crack at the crest above the concrete core wall. In this case also, further

knowledge of the behavior of the core wall would also be of interest, but will likely only be gained by inference from general performance.

12. There were many landslides throughout Taiwan. These affected access to many powerplants, switchyards and dams. No landslide damage to dams or power-plants was observed although the slides that occurred in the Terchi reservoir are indicative of the potential threat.

4. Acknowledgements

We wish to again offer our sympathies to the victims of this disaster and record our sincere appreciation for the access and assistance provided by the Taiwan Water Conservancy Board and Taiwan Power to allow us this opportunity to learn from their experience.

We also wish to record our appreciation to The Institute for Catastrophic Loss Reduction, BC Hydro and Acres International for their financial and logistical support for this valuable investigation.

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- Figure 43 - Rockslide, Techu Reservoir
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Table 1
Dams Visited

Dam	Type	Height (m)	Date Constructed	Location
Techi	Double curvature arch	181	1973	Upper Tachia River, Taichung County
Shih Kang	Gated concrete spillway structure	25	1976	Lower Tachia River, Taichung County
Mingtán	Concrete gravity	62	1984	Near town of Shuili, Nantou County
Shui Shih	Embankment with clay core & concrete core wall	30	1934	Sun Moon Lake, Nantou County
Toushíh	Embankment with clay core & concrete core wall	19	1934	Sun Moon Lake, Nantou County
Liyutan	Zoned embankment	96	1992	Tributary of Taan River, Miaoli County
Jenyitan	Zoned embankment	28	1987	Near Chiay, Chiai County
Lantan	Embankment with clay core & concrete core wall	31	1944	Near Chiay, Chiai County

Table 2
Liyutan Dam – Typical Peak Ground Accelerations

Location	Cross-Valley	Upstream-Downstream	Vertical
Rock Base	0.10g	0.05g	0.10g
Grouting Gallery	0.08g	0.05g	0.09g
Rock Abutment	0.14g	0.11g	0.09g
Dam Crest	0.22g	0.24g	0.15g