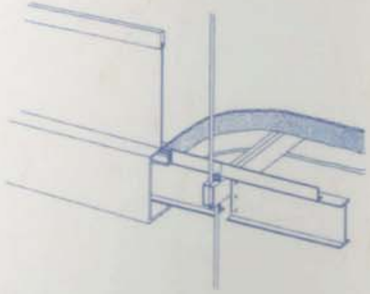
neering • Forensic engineering • Engineering failures

Engineering failures

get a lot of attention—inciting morbid curiosity and fueling concern over the condition of our infrastructure. But every engineering loss is the start of a forensic investigation into how, why, and what can be done to prevent future failures. As with scientific failures, engineering failures can be very instructive in teaching us what does not work.



Beyond Failure presents the circumstances of important failures that have had far-reaching impacts on civil engineering practice. Each case study narrates the known facts: design and construction, the failure, subsequent investigation or analysis, and, where appropriate, additional issues such as technical concerns, ethical considerations, professional practice issues, and long-term effects. The case studies are organized around eight common topics of undergraduate engineering courses and include teaching points and a reading list, so this book is useful to engineering faculty and students. With more than 40 full cases, including the Silver Bridge collapse in Point Pleasant, West Virginia; the levee breaches in New Orleans; and the *Challenger* space shuttle explosion, this book will also appeal to practicing engineers with an interest in forensic investigations or the analysis of historic failures.

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Beyond Failure

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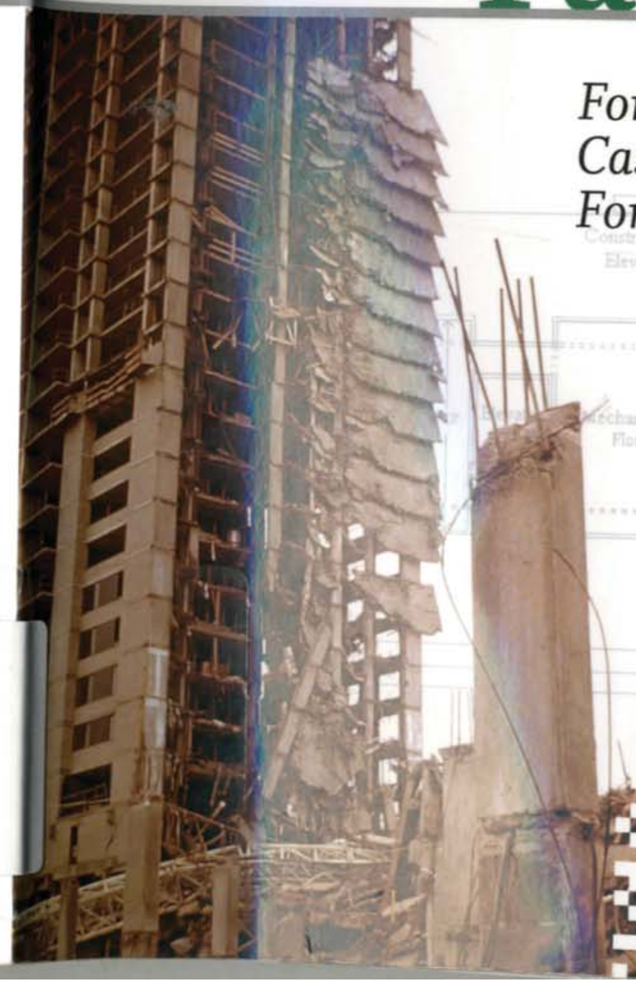
Forensic Case Studies For Civil Engineers



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> were destroyed and removed by the outrush of reservoir water, (3) that openings existed through inadequately sealed rock joints, and may have developed through cracks in the core zone of the key trench, (4) that, once started, piping progressed rapidly through the main body of the dam and quickly led to complete failure, (5) that the design of the dam did not adequately take into account the foundation conditions and the characteristics of the soil used for filling the key trench, and (6) that construction activities conformed to the actual design in all significant aspects except scheduling. (Independent Panel 1976, pp. iii–iv)

In the design and construction of earthen dams, it is necessary to select proper materials that are sufficiently resistant to piping and to ensure that they are compacted to the proper density. If a grout curtain is used, it is necessary to ensure that it is continuous and forms a seal with the underlying rock. The design should incorporate adequate defense against cracking and leakage. Finally, dams must have sufficient instrumentation to provide early warning of piping and impending failure.

Essential Reading

The most valuable single reference on this case study is the independent panel's 1976 report, "Report to the U.S. Department of the Interior and State of Idaho on Failure of Teton Dam." The case study has also been published by Solava and Delatte (2003).

Vaiont Dam Reservoir Slope Stability Failure

A slope stability failure is more commonly known as a landslide, particularly among nonengineers. This type of failure occurs when the weight of a soil mass overcomes the soil's shear resistance along a failure plane. Water within soil increases its unit weight while reducing the shear strength. As a result, water and water pressures often play a role in triggering a slope stability failure.

The Vaiont Dam disaster of 1963 was a classic slope stability failure. This is also called the Vajont Dam in some references; in Italian they are pronounced the same. Ironically, the dam itself did not fail and still stands today. The dam is a thin concrete wedge in a narrow gorge. A vast soil mass falling into the reservoir triggered a massive wave that blew over the dam and destroyed villages downstream.

Design, Construction, and Operation

The Vaiont Dam was part of an extensive system of dams, reservoirs, and hydroelectric powerhouses located in the Piave River Valley, high in the Italian Alps. The elements of this system were linked by tunnels and pipelines (Ross 1984, p. 131). The Piave River Valley is roughly 100 km (60 mi) due north of Venice, near the Austrian border.

The thin arch dam was 262 m (858 ft) high, but the arc around the crest was only 190 m (625 ft), and the chord was 169 m (555 ft). The dam was a double curvature type, 22 m (73 ft) thick at the base but only 3.4 m (11 ft) thick at the crest. It was the highest arch dam in the world at the time of construction, exceeded only by the 284-m (932-ft) Grand Dixence concrete gravity dam in the Swiss Alps (Ross 1984, pp. 132–134). The full reservoir was intended to contain a volume of 169 million m^3 (6 billion ft^3 or about 138,000 acre-ft) of water (Genevois and Ghirotti 2005).

The designer of the dam, Carlo Semenza, had reservations about the dam site as early as June 1957, when the dam owner proposed increasing the dam height by 30% to triple storage and power generation capacity. The owner was the private power company SADE, for Società Adriatica di Elettricità (Adriatic Electric Society). A March 22, 1959, landslide at the nearby Pontesei Reservoir of 3 million m^3 (106 million ft^3) of rock had killed one person (Wearne 2000, pp. 206–207).

The Pontesei landslide started slowly, but then began moving very rapidly. The flow over the Pontesei Dam was only a few meters deep and caused no damage to the valley below. The Vaiont Dam was already at an advanced state of construction, so this landslide caused some concern. Leopold Müller was responsible for studying the stability of the Vaiont Reservoir, in light of the concerns (Semenza and Ghirotti 2000).

Müller asked Carlo's son Edoardo, a recently graduated geologist, to investigate the site. Carlo identified an "uncemented mylonitic zone" running about 1½ km (1 mi) along the gorge, as well as the site of an ancient landslide. He identified layers typical of river sedimentation down to about 30 m (100 ft), showing where the ancient landslide had buried a riverbed. Edoardo took borings as deep as 171 m (561 ft). Edoardo was young, however, and the utility was reluctant to take his opinions seriously. They sought another opinion from a Professor Calois, who claimed that there were only 10–20 m (33–66 ft) of loose slide material over firm in situ rock (Wearne 2000, pp. 208–209).

Clearly, the issue was whether the reservoir was the site of an ancient landslide. If it was, the landslide could move again under the right conditions. The size of the landslide and its speed of movement would determine the extent of the damage that the slide would cause. Edoardo Semenza in fact identified a number of ancient landslides, but he considered only one to be potentially dangerous. The ancient landslide had pushed uphill on the other side of the valley, and the river channel had subsequently cut off and isolated a hill from the original slide. Edoardo Semenza cited this hill as proof of the ancient slide (Semenza and Ghirotti 2000).

Edoardo Semenza identified the following features:

- the 1.5-km (1-mi) zone of uncemented cataclasites along the base of the left wall of the valley, along with solution cavities, sinkholes, and springs;
- ancient landslide masses that had filled the valley and then had been cut into two by the new Vaiont stream;
- the southern slope of Mt. Toc, which had a "chair like" structure of bedding planes, dipping steeply at the top and more shallowly near the base; and
- a fault separating the in situ rock mass from the ancient landslide (Genevois and Ghirotti 2005).

Genevois and Ghirotti state that the dam's designers concluded that a landslide was not likely to occur, "mainly because of both the asymmetric form of the syncline . . . and the good quality of in situ rock masses" (2005, p. 41). In other words, because the form of the landslide was broken up and difficult to make out, it was thought that it would probably not move as a mass again.

In 1960, SADE began slowly filling the reservoir and monitoring earth movements. Elevations of 594 m (1,950 ft) and 650 m (2,133 ft) were reached. Throughout September, the movement rate increased from 5 to 10 mm/day (0.2 to 0.4 in./day), reaching 20–40 mm/day (¾–1½ in./day) in early October (Genevois and Ghirotti 2005).

On November 4, 1960, 750,000 m^3 (27 million ft^3) of rock fell into the Vaiont reservoir after a week of heavy rain. The landslide caused a 2-m (7-ft) wave in the reservoir, but no one was injured. Creep of the soil mass was observed over a large area. The recommendation was to lower the reservoir to slow the slide and to add drainage tunnels under the slide mass to reduce water pressures (Wearne 2000, pp. 209–210).

Müller had been asked to study the problem and propose remedial measures. Measures such as draining the mountainside, removing millions of cubic meters of soil, cementing the sliding mass along the failure surface, and buttressing the foot of the slide were considered but rejected as impractical. Müller believed that it would not be possible to completely stop the

slide but that its movements could be monitored and controlled (Semenza and Ghirotti 2000).

SADE implemented the plan. A 1.6-km (1-mi) bypass tunnel 5 m (16 ft) wide was built. The purpose of this tunnel was to ensure water flow to the dam even if part of the reservoir were blocked by a small landslide. A grid of sensors was installed to monitor earth movement. Drill holes more than 90 m (300 ft) deep explored the mountainside, and two drainage tunnels were provided. The water level was dropped to 600 m (1,968 ft), and creep slowed. At about this time, Carlo Semenza died, and his voice of caution was lost. As the reservoir was gradually raised again, a pattern seemed to be established—higher reservoir level, more movement. Therefore, it seemed that any future movement could be handled safely by lowering the reservoir (Wearne 2000, p. 211).

Müller observed that when the movement exceeded 15 mm/day (0.6 in./day) on the second filling, the lake level was 100 m (330 ft) higher than it had been when that amount of movement had occurred on first filling. He therefore hypothesized that the initial saturation of the materials was causing the movement and that it was safe to raise the lake level (Semenza and Ghirotti 2000).

In March 1963, SADE was nationalized and absorbed into the national power grid. The new owners, the Italian electric monopoly ENEL (for Ente Nazionale per L'Energia Elettrica), emphasized increasing electrical power production. It was assumed that the reservoir was safe up to 700 m (2,268 ft) and dangerous at 715 m (2,346 ft). However, by July 1963, the water in the reservoir was muddy, and people reported noises from within the mountain. Movements of as much as 570–700 mm (22½–27½ in.) per day were measured. Rain increased the level of the reservoir to 720 m (2,361 ft). Obviously, this was of concern. Movements were approximately 200 mm (8 in.) per day. The new owner began lowering the reservoir approximately 1 m (3 ft) per day (Wearne 2000, pp. 212–213). It is possible that the transfer of ownership delayed the decision to lower the reservoir (Semenza and Ghirotti 2000).

Failure

On October 9, 1963, at 10:41 P.M., approximately 270 million m^3 (9,535 million ft^3) of rock fell into the reservoir, moving as fast as 25 m/s (82 ft/s). A tremendous wave of water blew over the dam, virtually the entire reservoir, sending a 70-m (230-ft) wall of water down Vaiont gorge. It destroyed the town of Longarone downstream, with a population of 4,600, and severely damaged or destroyed the hamlets and villages of Villanova, Codissago, Pirago, and Fraseyn. There were 2,043 people killed, including 58 of the utility's employees (Wearne 2000, pp. 213–214). The flood also knocked out many access routes, hampering rescue operations (Ross 1984, p. 132).

The slide moved a 250-m (820-ft) thick mass of rock about 300–400 m (980–1,300 ft) horizontally. It pushed the old slide mass up the far slope. Trees and soil along the Vaiont Valley were removed as high as 235 m (770 ft) above the reservoir level (Hendron and Patton 1985, p. 8)

The dam, however, stood. It had withstood a force of approximately 4 million metric tons (4 million tons) of water, roughly eight times the force for which it had been designed. The dam is still there, but there is no water behind it (Wearne 2000, pp. 217–219). There was a small gouge in the concrete about 1.5 × 9 m (5 × 30 ft) along the crest near the left abutment (Ross 1984, p. 132).

The volume of the slide was slightly larger than the working volume of the Vaiont Reservoir. It was more than twice the volume of the largest earthen dam ever built, the Fort Peck Dam on the Missouri River in Montana. Because of the great volume, it was not practical to remove the material from the reservoir to restore the dam's function (Ross 1984, p. 134).

Investigations and Repercussions

About a week after the disaster, *Engineering News Record* (*ENR*) reported that the new owner, ENEL, had anticipated a slide but only expected about 19 million m^3 (670 million ft^3). They claimed that it was impossible to foresee that the slide would be so large. The engineer in charge of the dam had reportedly phoned the electric company asking for permission to evacuate the entire area on account of the earth movements. *ENR* noted that the electric company was said to have told him "to stay calm and sleep with his eyes open" (Ross 1984, pp. 132–133).

Investigations started almost immediately. An Italian government committee of inquiry with four members was charged to determine:

- whether the hydrogeological examination of the dam area was given proper consideration in planning and construction, and whether the previous landslides in the area were taken seriously,
- whether the dam's testing was still continuing at the time,
- the level of the reservoir in the 10 days before the disaster, and whether safety recommendations for the level were followed,
- whether a previous landslide in the area a few days before the disaster should have warranted an evacuation order downstream, and
- whether officials acted properly (Ross 1984, p. 133).

The commission's report was released four months after the failure. It blamed "bureaucratic inefficiency, muddled withholding of alarming information, and buck-passing among top officials." The prime minister of Italy suspended a number of public officials, including the province chiefs and civil engineers of Belluno and Udine (Ross 1984, pp. 134–135).

More than four years after the disaster, the public prosecutor of the province of Belluno charged 11 men with crimes ranging from manslaughter to negligence. By that time, two had already died. A third committed suicide the day before the trial was to begin. The charges included ignoring consultants' cautions and failing to fully investigate the earlier earth movements in the slide area. The prosecutor asserted that each of the men charged could have controlled the situation and prevented the disaster. The charges for each individual were outlined in detail in the December 7, 1967, issue of *Engineering News Record* (Ross 1984, pp. 135–137).

Three of the 11 charged were found guilty, and two served short jail terms. The proceedings, however, focused more on assigning blame than on determining the technical cause of the failure. Technical papers were written, but the conclusions did not agree (Wearne 2000, pp. 217–218).

Engineering Analyses

Although a landslide had been feared, the size and velocity were surprising. The mass and velocity pounded considerable kinetic energy into the reservoir, which was the main cause of the height of the wave in the lake and the extent of the destruction downstream of the dam. Müller's hypothesis that the movements only took place when the material was first saturated was obviously disproved. The phenomena involved proved to be complex.

The following causes have been suggested for the slide's triggering mechanism:

- the creation of the lake basin, as well as the variations in the level of the reservoir;
- the clay seam along the failure surface;
- the ancient landslide;
- the geological structure;
- seismic action; and
- a confined aquifer behind and below the failure surface (Semenza and Ghirotti 2000).

Seismic action has generally been ruled out, but the other factors all seem to have been involved in the landslide.

Müller wrote several papers over the five years after the landslide, contending that once a certain limit velocity had been achieved, some type of thixotropy must have occurred. Perhaps water in the joint had made the mass of the slide buoyant. Müller continued to favor the hypothesis of a first-time slide and argued that the slide that took place had not been predictable (Genevois and Ghirotti 2005). *Thixotropy* is shear-thinning material behavior, a decrease in shear flow resistance once a certain strain rate is achieved. The phenomenon is seen when ketchup suddenly flows quickly from a bottle, drenching a plate.

The lack of agreement among the studies of the Vaiont slide was a cause of concern for the engineering geology community.

Despite the substantial literature on the topic, no study had previously taken account of:

- the three-dimensional shape of the slide surface,
- actual laboratory shear strengths of material from the site, and
- piezometric levels taking into account rainfall and reservoir levels (Hendron and Patton 1985, p. 2).

Frank Patton, a consulting engineering geologist, and his colleagues began investigating the slide in 1975 and visited the failure plane. They found a layer of plastic, low-strength clay (also known as fat clay) at the base of the slide, between 13 and 100 mm (½ and 4 in.) thick. The clay would have reduced friction along the failure plane and also acted as a membrane to hold back water (Wearne 2000, pp. 218–220).

Surprisingly, many of the earlier investigations had reported that there was no clay layer. The Hendron and Patton (1985) report contains dozens of photographs of the soft clay layer, along with a map providing the locations. The clay showed up everywhere they looked along the failure plane. They noted that where the clay was exposed, it was rapidly eroded by rainfall. Also, for the first few years the clay had been covered up by slide debris that was gone by the time Hendron and Patton surveyed the hillside. There was some confusion because of the different terms used in technical writings for the soft clay layer.

> The basic structures affecting the slide are: (a) the steep back of the slide which provided the driving forces, (b) the pronounced eastward dip of the seat of the slide, (c) the continuous layers of very weak clays within the bedded rocks, and (d) the faults along the eastern boundary of the slide. (Hendron and Patton 1985, p. 21)

It had been observed that movements increased as the reservoir level increased, but the reservoir level also increased when it rained. Therefore, two possible causes for the increased movement were higher water pressures caused by higher water levels within the reservoir, or increased pressures within the mountain from rainfall against the fat clay. To determine the cause, it would be necessary to know water pressures behind the clay before the failure (Wearne 2000, pp. 220–221).

Hendron and Patton (1985, pp. 51–58) plotted the correlations among reservoir level, precipitation, and rate of movement. They discussed Müller's contention that the movement was greatest on first wetting, pointing out, "The erroneous assumption which led to the conclusions . . . was that all other factors were remaining constant and the reservoir level was the main variable controlling the stability of the slide. In fact, rainfall was significant and was not remaining constant" (Hendron and Patton 1985, p. 54).

Periods of high rainfall preceded all of the major slide movements, but obviously the reservoir level often also rose at the same time. At those times when the reservoir was at the same level, however, the difference in movement correlated with the amount of rainfall. It proved possible to plot a failure envelope of combinations of reservoir elevation and precipitation, with points plotted above the envelope indicating instability of the soil mass.

Patton reviewed Edoardo Semenza's reports, finding that Edoardo had reported this clay layer as mylonite. Edoardo Semenza had made four boreholes, one of which penetrated to the base of the slide. This borehole found water pressures equivalent to 70–90 m (230–295 ft) above the reservoir. Unfortunately, the tunnels that had been installed to reduce water pressure were too high up on the mountain. The heavy rains the first week in October had infiltrated the mountain and increased pressures behind the clay layer. When the water level in the reservoir had been lowered just before the landslide, it had reduced the pressure holding back the slide and might in fact have triggered it. The nature of the clay might also have accelerated the disaster; its kinetic coefficient of friction was much smaller than its static coefficient of friction. Tests later showed that the shear strength decreased as much as 60% when slip exceeded 100 mm (4 in.) per minute. In fact, the fat clay acted as a lubricant (Wearne 2000, pp. 221–223). The test results that identified the loss of shear strength had been carried out by Tika and Hutchinson (Genevois and Ghirotti 2005).

Patton and co-author Hendron started from Edoardo Semenza's findings, with the following results:

- The existence of the old landslide was confirmed.

- The clay layer was up to 100 mm (4 in.) thick, with a residual friction angle between 8° and 10°.
- They found the probable existence of two aquifers, one above and one below the clay layer.

The lower, confined aquifer was fed mostly by precipitation on Mt. Toc (Semenza and Ghirotti 2000). Figure 7-4 shows a plan view of the mass that slid to the north. To observers on the opposite side of the valley, it appeared to be M-shaped.

Using aerial photographs available at the time, Hendron and Patton (1985, p. 31) identified a pattern of apparent kettles or sinkholes, along with shallow slide areas. These are shown in Fig. 7-5. These features could have been used to identify a possible landslide area for further ground-based investigation.

These sinkholes readily allowed rainwater or snowmelt to infiltrate into Mt. Toc, and to reach the base of the slide as shown in Fig. 7-6. With increasing precipitation, pressure would build behind the clay layer. During his surveys in 1959 and 1960, Edoardo Semenza had observed moist areas and springs below the eventual slide plane, which was consistent with the hydrogeology shown in Fig. 7-6 (Hendron and Patton 1985, pp. 34–35).

Soil samples were tested at laboratories in the United States, Canada, and Italy. The grain size distribution was 51% clay, 36% silt, 7% sand, and 6% gravel. Earlier studies had reported 52–70% clay. Atterberg limit tests placed the clay in two groups. The first was a CL/ML/MH (low plasticity clay/low plasticity silt/high plasticity silt) with liquid limits from 33 to 60 and plasticity indices from 9 to 27. The second was a CH (high plasticity clay) with liquid limits from 57 to 91 and plasticity indices from 30 to 61. Clay mineral analyses indicated that up to 80% of each sample was clay minerals. Overall, "such clay minerals have an expanding lattice, are associated with low shear angles, and exhibit swelling properties when stresses are reduced and water is present" (Hendron and Patton 1985, pp. 37–39).

Hendron and Patton (1985, pp. 59–65) included the reservoir level, precipitation findings, and the soil properties in a revised three-dimensional stability analysis. "The shear strength along the base of the slide was assumed to be related more to the residual shear strength of the multiple layers of clay found along the basal surface of sliding than to the higher shear strengths of the rock-to-rock contacts," which had been used in previous stability analyses (Hendron and Patton 1985, p. 59). Laboratory tests suggested a residual angle of shearing resistance ϕ_r of 5°–16°, with no cohesion. Such a low angle had puzzled previous investigators because it meant that the slide mass should have never been able to stay in place at all. The

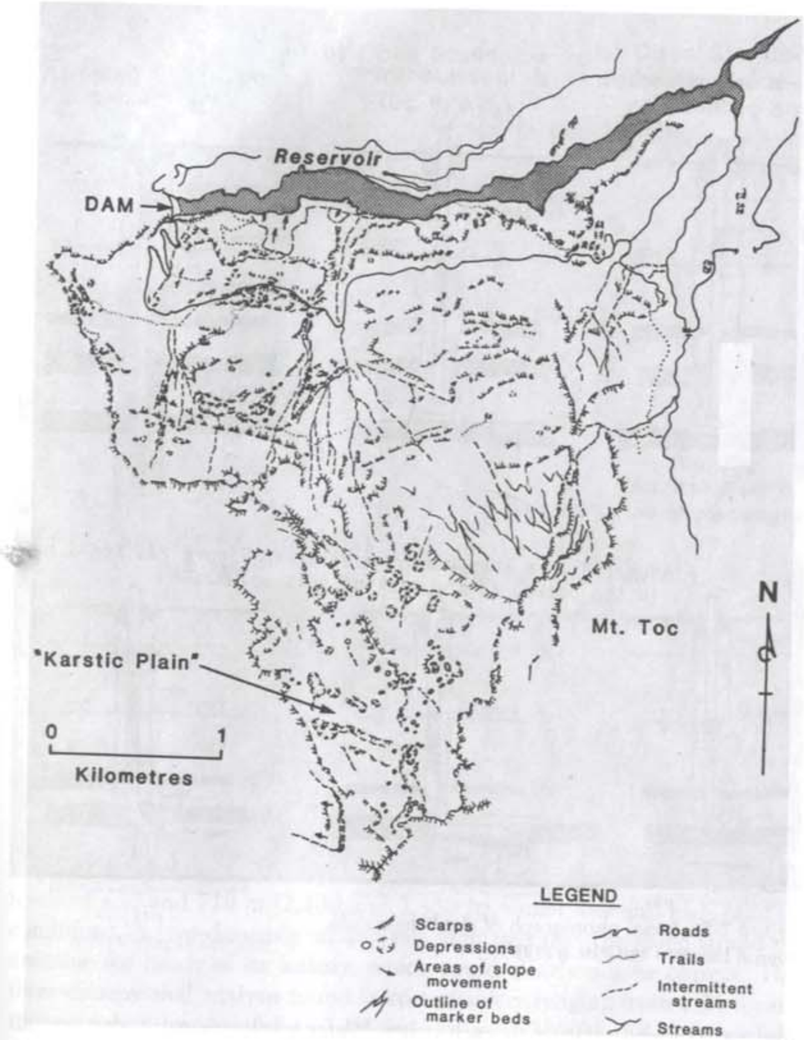

Figure 7-4. Plan view of mass before slide.
Source: Hendron and Patton (1985).

residual angle of shearing resistance ϕ_r was adjusted to a mean value of 10°–12°, based on the number of rock-to-rock contacts and the irregular surface. The angle of shearing resistance within the mass (β) was estimated at 30°–40°. This shearing resistance was mobilized when adjacent slices of

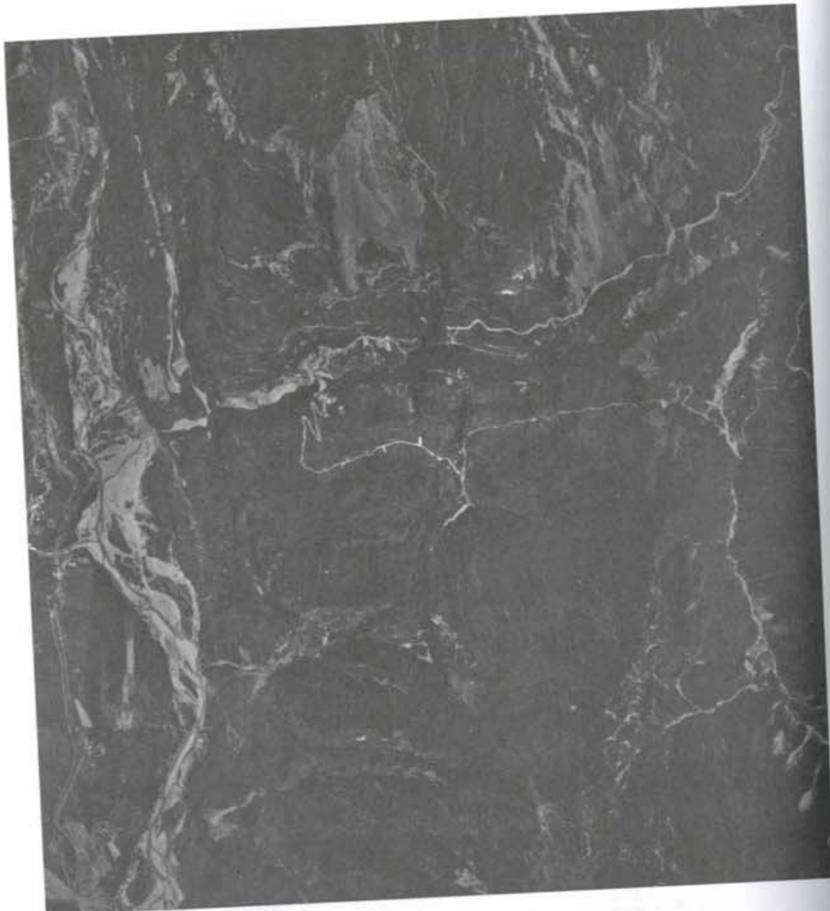

Figure 7-5. Aerial photograph of site before slide.
Source: Hendron and Patton (1985).

the mass moved relative to each other. Water pressures were also considered, with the reservoir pressure against the front of the slide and higher pressures, up to 90 m (295 ft), at the back.

Hendron and Patton (1985, pp. 66–79) noted that because the failure had occurred, it was necessary for stability analyses to demonstrate a factor of safety near 1.0 under the failure conditions. It was also necessary to demonstrate factors of safety near 1.0 at the times when significant movement was observed, as well as somewhat greater for the periods where

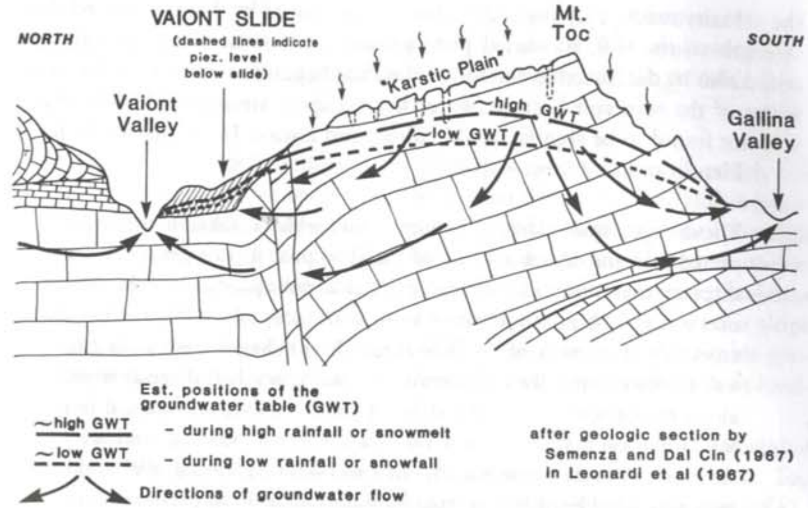

Figure 7-6. Infiltration of water through the mountain.

movements were insignificant. The periods of time when the factor of safety should be near 1.0 are the prehistoric landslide, the major movement of October 1960 when the cracks formed, and October 9, 1963. The important input parameters for the model are the soil shear strength, which is relatively constant under all conditions, and the pore pressure, which varies with different reservoir levels and rainfall conditions. The angle of shearing resistance within the mass was also important. Two- and three-dimensional slope stability analyses were performed for no reservoir and for reservoir levels of 650 and 710 m (2,130 and 2,330 ft), under low and high rainfall conditions. A two-dimensional analysis indicated that the slope would be unstable for much of its history, which could clearly not be correct. The three-dimensional analysis found factors of safety ranging from 1.21 for no reservoir with low rainfall to 1.00 for 710 m (2,330 ft) and high rainfall, the conditions under which the slide occurred. The second lowest factor of safety, 1.08, occurred at 650 m (2,130 ft) with high rainfall. The final soil parameters used were ϕ_r of 12° and β of 40° between slices and 36° along the eastern surfaces of the slide.

Once the slide began to move, of course, the problem became more complicated. Because the mass of the slide and the driving force remained the same, a considerable reduction in the resisting force was necessary to explain the velocity reached by the slide. This velocity could be due to reduced shear strength of the soil, increased pore pressure, or both. Shearing resistance could also be decreased as the toe of the slide began to ride into and over the water of the reservoir because water has no shear strength. This last factor was not found to be significant (Hendron and Patton 1985, pp. 80–82).

Hendron and Patton stated,

> The strength losses along the sliding surface which resulted in the unexpected high maximum velocity of the slide probably originated from three mechanisms: (a) a displacement induced reduction in the friction angle, β, between adjacent vertical surfaces of the sliding mass, especially at the back of the slide at the abrupt change from a steep to a flat failure plane; (b) a reduction of peak to residual shear strength along the eastern side of the slide where the sliding surface did not follow the bedding planes but sheared across the bedding; and (c) a reduction in shear strength along the basal sliding plane parallel to the bedding caused by heat-generated increases in the water pressure along this plane. (1985, p. 83)

In their analysis, they found that the combined action of all three phenomena could explain the velocity reached by the slide mass if the reduction in the clay's shear strength was about 50% (Hendron and Patton 1985, p. 90). The explanation for the high velocity of the slide was that the shear resistance of the clay layer decreased substantially once movement started. This change could have occurred due to either frictional heat or an inherent property of the material. Because it would have taken time for the heat to build up, the alternate explanation of the loss of frictional strength with shear strain rate is most likely (Semenza and Ghirotti 2000).

Lessons Learned

The Vaiont Dam case study shows the need for a thorough geotechnical investigation for a construction project, particularly one as important as a massive dam. It is necessary in particular to locate any thin clay seams that represent potential weak failure planes. Genevois and Ghirotti state,

> The catastrophic 1963 landslide failure has demonstrated to professionals and researchers in the fields of civil engineering and engineering geology the importance of performing detailed geologic investigations of the rim of narrow steep-walled valleys, which are planned as the reservoir for large dams. The failure mechanism of a large landslide mass may be very complex and difficult to evaluate and even leading experts may fail to reach correct conclusions if they do not fully understand all factors affecting the mechanism and the evolution of the landslide. (2005, p. 43)

At the time, geotechnical investigations were much less thorough and relied on more primitive tools than those available today. The full implications of the confined aquifer were not recognized, and the speed of the slide was much more than expected. The Vaiont landslide has played an important role in the subsequent development of engineering geology (Semenza and Ghirotti 2000). In fact, at the time of the Vaiont project, reservoir slope stability analysis was usually not included in the design. The available geological studies, carried out decades before, did not identify the ancient landslide (Genevois and Ghirotti 2005).

Interestingly, the local population seemed to be aware of the unstable nature of the slope. Edoardo Semenza pointed out that the name Mt. Toc means "crazy" in the local dialect (Hendron and Patton 1985, pp. 64–65).

The low strength and lubricating properties of high plasticity fat clays are important. Another important property of clay material is impermeability: Because it is difficult for water to pass through, it is easy for high pressures to build up under or behind clay layers.

Slope stability and the factors that affect the weight of the soil mass, as well as the frictional shear resistance along the failure plane, are important. Overall, water tends to increase the soil mass and reduce the shear resistance of many soils. In hindsight, it might have been possible to reduce pore pressures, drain the slope, and reduce the risk of a landslide, but the two drains installed were too high up the slope and did not penetrate behind the clay layer.

The engineers responsible for the dam's operations also failed in their duty to inform the public about the danger of a dam failure. Although the landslide itself was sudden, there had been plenty of concern and warning beforehand. The fact that the reservoir was being lowered shows that there was an appreciation of the risk. Even if the anticipated slide were 19 million m^3 (670 million ft^3), that was more than enough to warrant an evacuation order.

Later that same year, the Baldwin Hills Reservoir near Los Angeles, California, failed. Fortunately, the recent memory of the Vaiont disaster led the engineers to order an evacuation, perhaps saving hundreds of lives. On the other hand, the Malpasset Dam failure less than four years before should have been adequate notice for the engineers operating the Vaiont Dam. The Malpasset case study is discussed in Chapter 8.

An October 24, 1963, editorial published in *Engineering News Record* made some comments on the Vaiont Dam landslide:

- The main lesson is obvious—get out of the way if a landslide threatens a reservoir. An evacuation should have been ordered at the first signs of trouble, which were roughly 10 days before the slope stability failure. The potential, and in the end actual, loss of life greatly outweighed any inconvenience from a possible false alarm.
- The engineers operating the dam were confident that the slide would only be about a tenth of what actually occurred, but they were wrong. They were also confident that the dam would hold, and it was known that arch dams had considerable reserve strength. Given the uncertainty on both elements, not evacuating the downstream population was a considerable gamble.
- This was the first time in history that a reservoir had been completely filled by a massive landslide (Ross 1984, pp. 138–139).

As lessons learned, Genevois and Ghirotti state, "By its nature, any specific landslide is essentially unpredictable, and the focus is on the recognition of landslide prone areas . . ." (2005, p. 50). Hendron and Patton (1985, pp. 96–97) cautioned that cursory studies of important events such as the Vaiont Slide could be misleading. "Previous studies of the Vaiont Slide vary from useful factual accounts to misleading fiction. . . . The most misleading accounts in the literature have generally been given by those who have not visited the site or who are not familiar with the geology."

Essential Reading

The key document on this case study is *The Vaiont Slide: A Geotechnical Analysis Based on New Geological Observations of the Failure Surface, Volume I, Main Text* (Hendron and Patton 1985). It may be difficult to find except on loan from some large university engineering libraries.

The Vaiont Dam case study is covered in pp. 206–225 of *Collapse: When Buildings Fall Down* by Wearne (2000) as well as pp. 127 and 130–139 of Ross (1984). Wearne's chapter contains accounts from engineers and survivors of the disaster. *Engineering News Record* reported on the case in the October 17, 1963, and December 7, 1967, issues and published an editorial in the October 24, 1963, issue.

A similar mechanism caused the 2005 Bluebird Canyon landslide near Los Angeles, which is featured in the History Channel's Modern Marvels *Engineering Disasters 17* videotape and DVD.

The Transcona and Fargo Grain Elevators

The Transcona Grain Elevator collapsed in 1913, and the Fargo Grain Elevator collapsed in 1955, with almost identical failure mechanisms. They were similar structures built on similar types of soil. The two cases provided important confirmations of soil-bearing capacity calculations.

The Transcona Grain Elevator

This collapse was investigated four decades after the event by Ralph B. Peck, a geotechnical engineer and professor of civil engineering at the University of Illinois, who was also a native of Transcona's neighbor Winnipeg (Morley 1996, p. 27). Construction of the elevator started in 1911. It consisted of a work house and a bin house. The work house was 21 × 29 m (70 × 96 ft) in plan, and 55 m (180 ft) high, with a raft foundation 3.7 m (12 ft) below the surface. The bin house had 13 bins, each approximately 28 m (92 ft) high and 4.3 m (14 ft) in diameter. The bins rested on a reinforced concrete raft foundation, 23.5 m (77 ft) wide and 59.5 m (195 ft) long, also at a depth of 3.7 m (12 ft) below the surrounding soil. The underlying soil was a layer of stiff blue clay, roughly 6–11 m (20–35 ft) thick (Peck and Bryant 1953).

Small plate load tests were performed before construction. These tests indicated that the soil should be able to bear a pressure of 383–479 kPa (4–5 ton/ft^2). The total pressure with the warehouse filled with grain would be no more than 316 kPa (3.3 ton/ft^2) (Morley 1996, p. 26).

The structure was finished in September 1913 and began to be filled with grain as uniformly as possible. On October 18, 1913, the bins were about 88% full, and settlement was observed, increasing within an hour to a uniform 300 mm (1 ft). Over the next 24 hours, the structure tilted to the west by almost 27°. Surprisingly, the bin house remained intact through the rotation. Wash borings were taken immediately after the failure, and they determined the thickness of the layer of clay as well as several underlying layers (Peck and Bryant 1953).

> An eyewitness account of the failure states that on October 18, 1913, the bin house began to move almost imperceptibly, but after an hour, 0.31 m [1 ft] of vertical settlement had occurred. It then began to tilt to the west, and when it stopped moving the next day, it was resting at an angle of 27° from the vertical. The east side had risen 1.52 m [5 ft] above the original ground level, and the western side was about 8.84 m