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Introduction

1.1 Types of Embankment Dam

In dam engineering, many dam types, such as the arch dam, gravity dam, arch-gravity dam, barrage, and embankment dam, are used, but the embankment dam is the most important type. This is because the majority of dams around the world are embankment dams. Embankment dams are mainly made from compacted earth. There are two main types; rock-fill and earth-fill dams. Embankment dams rely on their weight to hold back the force of water, like gravity dams made from concrete.

Rock-fill dams are embankments of compacted free-draining granular earth with an impervious zone. The earth utilized often contains a high percentage of large particles, hence the term *rock-fill*. The impervious zone may be on the upstream face and made of masonry, concrete, plastic membrane, steel sheet piles, timber, or other materials. The impervious zone may also be within the embankment, in which case it is referred to as a *core*. In instances where clay is often utilized as the impervious material, the dam is referred to as a *composite dam*. To prevent internal erosion of clay into the rock-fill due to seepage forces, the core is separated using filters. The filters are specifically graded soils designed to prevent the migration of fine grain soil particles. When suitable material is at hand, transportation is minimized leading to cost savings during construction. Rock-fill dams are resistant to damage from earthquakes. However, inadequate quality control during construction can lead to poor compaction and sand in the embankment, which can lead to liquefaction of the rock-fill during an earthquake. Liquefaction potential can be reduced by keeping susceptible material from being saturated, and by providing adequate compaction during construction.

A concrete-face rock-fill dam has concrete slabs on its upstream face. This design offers the concrete slab as an impervious wall to prevent leakage and a structure that will resist uplift pressure. In addition, the concrete-face rock-fill dam design is flexible for topography, faster to construct, and less costly than earth-fill dams. The concrete-face rock-fill dam originated during the California Gold Rush in the 1860s when miners constructed rock-fill timber-face dams for sluice operations. The timber was later replaced by concrete as the design was applied to irrigation and power

schemes. As concrete-face rock-fill dam designs grew in height during the 1960s, the fill was compacted and the slab's horizontal and vertical joints were replaced with improved vertical joints. In the last few decades, the design has become popular. Currently, the tallest concrete-face rock-fill dam in the world is the 233 m tall Shuibuya Dam in China, which was completed in 2008.

Earth-fill dams, also called earthen, rolled-earth, or simply earth dams, are constructed as a simple embankment of well-compacted earth. A homogeneous rolled-earth dam is entirely constructed of one type of material but may contain a drain layer to collect seep water. A zoned-earth dam has distinct parts or zones of dissimilar material, typically a locally plentiful shell with a watertight clay core. Modern zoned-earth embankments employ filter and drain zones to collect and remove seep water and preserve the integrity of the downstream shell zone. An outdated method of zoned earth dam construction utilized a hydraulic fill to produce a watertight core. Rolled-earth dams may also employ a watertight facing or core in the manner of a rock-fill dam. An interesting type of temporary earth dam occasionally used in high latitudes is the frozen-core dam, in which a coolant is circulated through pipes inside the dam to maintain a watertight region of permafrost within it.

A third type of embankment dam is built with an asphalt concrete core. The majority of such dams are built with rock and/or gravel as the main filling material. Almost 100 dams of this design have now been built worldwide since the first such dam was completed in 1962. All asphalt-concrete core dams built so far have an excellent performance record. The type of asphalt used is a visco-elasto-plastic material that can adjust to the movements and deformations imposed on the embankment as a whole, and to the settlements in the foundations. The flexible properties of the asphalt make such dams especially suited to earthquake regions.

In this book, the rock-fill dam with a soil core is called the *earth-rock fill dam*. There are usually two types of soil core. One is the *vertical* core, and the other is the *sideling* core. The problem of hydraulic fracturing in the soil core of the earth-rock fill dam is focused on in this book.

According to statistics analysis from the Chinese National Committee on Large Dams (CHINCOLD) and the International Commission on Large Dams (ICOLD), by the end of 2005 the number of the dams higher than 100 m worldwide was 851, and the number in China was 130. By the end of 2008, the number of the dams higher than 100 m in China was up to 142. In the 142 dams above 100 m in China, the number of embankment dams was 69. And according to the *Bulletin of First National Census for Water* given by Ministry of Water Resources, P. R. China and National Bureau of Statistics, P. R. China (2013), by the end of 2011, the number of reservoirs in China totaled 98 002, with a combined storage capacity of 932.312 billion m³. Among these reservoirs, 97 246 were completed, with a total storage capacity of 810.410 billion m³, and 756 were under construction, with a total storage capacity of 121.902 billion m³.

Hydropower is a renewable energy source where power is derived from the energy of water moving from higher to lower elevations. It is a proven, mature, predictable, and price competitive technology. Hydropower has the best conversion efficiency

of all known energy sources (about 90% efficiency, water to wire). It also has the highest energy payback ratio. The total worldwide technically feasible potential for hydropower generation is 14 368 TWh per year with a corresponding estimated total capacity potential of 3838 GW (IJHD, 2005); five times the current installed capacity. Undeveloped capacity ranges from about 70% in Europe and North America to 95% in Africa indicating large opportunities for hydropower development worldwide. China, Canada, Brazil, and the US together account for over 46% of the production (TWh) of electricity in the world and are also the four largest in terms of installed capacity (GW) (IEA, 2008). According to the work of Tortajada (2008), in China the gross theoretical hydropower potential was 6083 TWh per year, the technically feasible hydropower potential 2474 TWh per year, the economically feasible hydropower potential 1753 TWh per year, and the planned hydro capacity 49–65 GW. About 75% of the existing 45 000 large dams in the world were built for the purpose of irrigation, flood control, navigation, and urban water supply schemes. Only 25% of large reservoirs are used for hydropower alone or in combination with other uses, as multi-purpose reservoirs.

1.2 Hydraulic Fracturing

In China, 17 earth-rock fill dams higher than 100 m have been constructed, and more than 24 are to be constructed. Most of them are located in Western China where water resources are very abundant. Among these high earth-rock fill dams, some are higher than 200 m, such as the Nuozhadu Dam (261.5 m in height) on the Lancang (Mekong) River in the Yunnan Province, the Shuangjiangkou Dam (322 m in height), and the Changhe Dam (240 m in height) on the Dadu River in the Sichuan Province in the southwest of China. It is well known that cracks frequently occur in the soil core of the earth-rock fill dam. The cracks are believed to be the result of stress arching action and/or hydraulic fracturing in the soil core (Zhu and Wang, 2004). Care must be taken to prevent such cracking and the engineers must decide whether the cracks are likely to extend and become serious, whether they are stable and can be backfilled, or will self-heal.

There have been a number of well-studied cases where dams have failed or been damaged by concentrated leaks for no apparent reason. In some of these experiences, investigators concluded that differential settlement cracks were probable causes, even though no cracks were seen on the surface. In these examples, it was not determined whether the cracks were open before the reservoir filled or whether they might have opened afterward.

In a number of the histories, a concentrated leak appeared abruptly at the downstream side of the dam after the reservoir was filled, perhaps several hours or several days later. This indicated that no large open cracks existed before the reservoir was raised. This is one piece of evidence for the conclusion that, under certain conditions, the reservoir water pressure acting on the upstream face of the dam can cause existing closed cracks to open or can create new ones.

As the embankment is deformed by differential settlement, the minor principal stress in the zone of potential cracking decreases and may approach zero, the major principal stress is less than the unconfined compressive strength. When the minor principal stress becomes zero (or even negative if the soil can withstand tensile stress), a crack is imminent and will open if further deformation occurs. Although the initial crack may be very narrow, perhaps not even visible, water from the reservoir can penetrate it. As a result, the stress acting on the inner planes of the crack changes abruptly from zero to a compressive stress. The compressive stress would approach the reservoir head if the crack did not extend completely through the core. The result is an increase in the width of the crack. The water from the reservoir may enter and open or spread an existing crack that had been previously closed, or the water pressure may form a new one. These situations were called *hydraulic fracturing* by Sherard (1973).

Hydraulic fracturing is a physical phenomenon in which the crack in rock or soil is induced or expanded by water pressure due to the rise of water level elevation (Independent Panel to Review Cause of Teton Dam Failure, 1976). The hydraulic fracturing is also defined as a weak link phenomenon in which fracturing will occur in the least resistant soil subjected to increased water pressure (Jaworski, Duncan, and Seed, 1981). Hydraulic fracturing can occur even in a theoretically homogeneous embankment, but the probability of its occurrence is much higher if the material is not homogeneous with respect to deformability and permeability (Sherard, 1973). Hydraulic fracturing in the soil core of the earth-rock fill dam is a very important and troublesome geotechnical technique related to dam safety. It may occur if “water wedging” action induced by water entering the crack located at upstream surface of the core is intensive enough. This is because the water wedging action may change the nominal stress intensity at the tip of the crack (Wang, Zhu and Zhang, 2005). The problem of hydraulic fracturing has received a lot of attention in many studies (e.g., Sherard, 1986; Lo and Kaniaru, 1990) since the failure of the Teton Dam of America in 1976 (Independent Panel to Review Cause of Teton Dam Failure, 1976), but is far from being solved completely.

In several unsolved problems related to the safety of earth-rock fill dams, hydraulic fracturing in the soil core of the earth-rock fill dam is one that has received much attention from designers and researchers. Hydraulic fracturing is generally considered a key cause of inducing leakage of the dam during first filling. The occurrence of hydraulic fracturing is also considered the main reason behind the damage of some dams (internal erosion).

Because of the importance and complexity of the problem, hydraulic fracturing has received a lot of attention in recent decades. Seeing hydraulic fracturing occur is very difficult, such that investigation into the problem is only based on the theoretical analysis and reasoning from tests and/or experimental data. In order to clarify the conditions and mechanisms inducing hydraulic fracturing, many investigations have been conducted. The investigations include field tests, laboratory experiments, and numerical simulations. The viewpoints on the conditions and mechanisms inducing hydraulic fracturing also differ from study to study. In laboratory conditions, the simulation of

hydraulic fracturing is very different to that in actual earth-rock fill dams. These reasons may result in difficulty of solving the problem of hydraulic fracturing, especially in the designation and construction of dams.

At one extreme, hydraulic fracturing is believed to have caused the complete failure of the Teton Dam, and the important erosional damage to the Balderhead Dam. At the other extreme, hydraulic fracturing may cause the opening of very narrow cracks, through which no appreciable concentrated leakage takes place and no damage occurs. Leaks are believed to have been caused by cracks that have developed abruptly after several years of satisfactory performance (e.g., Hyttejuvet Dam). The failure of the Teton Dam, the erosion of the Balderhead Dam and the leakage of the Hyttejuvet Dam are briefly described next.

1.3 Failure of the Teton Dam

Dam failures are generally catastrophic if the structure is breached or damaged significantly. The main causes of dam failures include mainly inadequate spillway capacity, piping through the embankment, foundation, or abutments, spillway design error, geological instability caused by changes to water levels during filling; or poor surveying, poor maintenance (especially of outlet pipes), extreme rainfall; and human, computer, or design error. The failure of the Teton Dam was caused by water leakage through the earthen wall, or from cracking induced by hydraulic fracturing that occurred in the soil core (Smalley, 1992).

The Teton Dam was a federally built earthen dam on the Teton River in southeastern Idaho, set between the Fremont and Madison Counties in the USA. When filled for the first time, the Teton Dam suffered a catastrophic failure on June 5, 1976. The collapse of the dam resulted in the deaths of 11 people and 13 000 cattle. The dam cost about US\$100 million to build, and the Federal government paid over US\$300 million in claims related to the dam failure. Total damage estimates have ranged up to US\$2 billion. The dam has not been rebuilt. The dam site is located in the eastern Snake River Plain, which is a broad tectonic depression on top of rhyolitic ash-flow tuff. The tuff, a volcanic rock dating to about 1.9 million years, sits on top of sedimentary rock. The area is very permeable, highly fissured, and unstable. Test boreholes, drilled by engineers and geologists employed by the Bureau of Reclamation, US Department of the Interior, showed that one side of the canyon was highly fissured, a condition unlikely to be remediated by the Bureau's favored method of "grouting" (injecting concrete into the substrates under high pressure).

The dam (see Figure 1.1) was completed in November 1975 and no seepage was noted on the dam itself before the date of the collapse. However, on 3 June 1976, workers found two small springs had opened up downstream. At the time of the collapse, spring runoff had almost filled the new reservoir to its capacity with a maximum depth of 73 m. Water began seeping from the dam on the Thursday before the collapse, an event not unexpected for an earthen dam. The only structure that had been initially prepared for releasing water was the emergency outlet works, which could only carry



Figure 1.1 View of the Teton Dam and its spillway. (Reproduced with permission of U.S. Department of the Interior)

$24 \text{ m}^3/\text{s}$. Although the reservoir was still rising over 1.2 m per day, the main outlet works and spillway gates were not yet in service. The spillway gates were cordoned off by steel walls while they were being painted.

On Saturday, 5 June 1976, at 7:30 a.m., a muddy leak appeared, suggesting sediment was in the water, but engineers did not believe there was a problem. By 9:30 a.m., the downstream face of the dam had developed a wet spot erupting water at $0.57\text{--}0.85 \text{ m}^3/\text{s}$, and embankment material began to wash out. Crews with bulldozers were sent to plug the leak, but were unsuccessful. Local media appeared at the site, and at 11:15 a.m. officials told the county sheriff's office to evacuate downstream residents. Work crews were forced to flee on foot as the widening gap, now over the size of a swimming pool, swallowed their equipment. The operators of two bulldozers caught in the eroding embankment were pulled to safety with ropes. At 11:55 a.m., the crest of the dam sagged and collapsed into the reservoir, and 2 minutes later, the remainder of the right-bank third of the main dam wall disintegrated (see Figure 1.2). Over $57\,000 \text{ m}^3/\text{s}$ of sediment filled water emptied through the breach into the remaining 9.7 km of the Teton River canyon, after which the flood spread out and shallowed on the Snake River Plain. By 8:00 p.m. that evening, the reservoir had completely emptied, although over two-thirds of the dam wall remained standing (see Figure 1.3).

Study of the dam's environment and structure (see Figures 1.4 and 1.5) placed blame on the collapse of the permeable loess soil used in the core and on fissured (cracked) rhyolite in the foundations of the dam that allowed water to seep underneath. The permeable loess was found to be cracked. It is postulated that the combination of these flaws allowed water to seep through the dam and led to internal erosion (called *pip*ing)



Figure 1.2 Teton Dam collapse on 5 June 1976. (Reproduced with permission of U.S. Department of the Interior)



Figure 1.3 View of the Teton Dam after collapse. (Reproduced with permission of U.S. Department of the Interior)

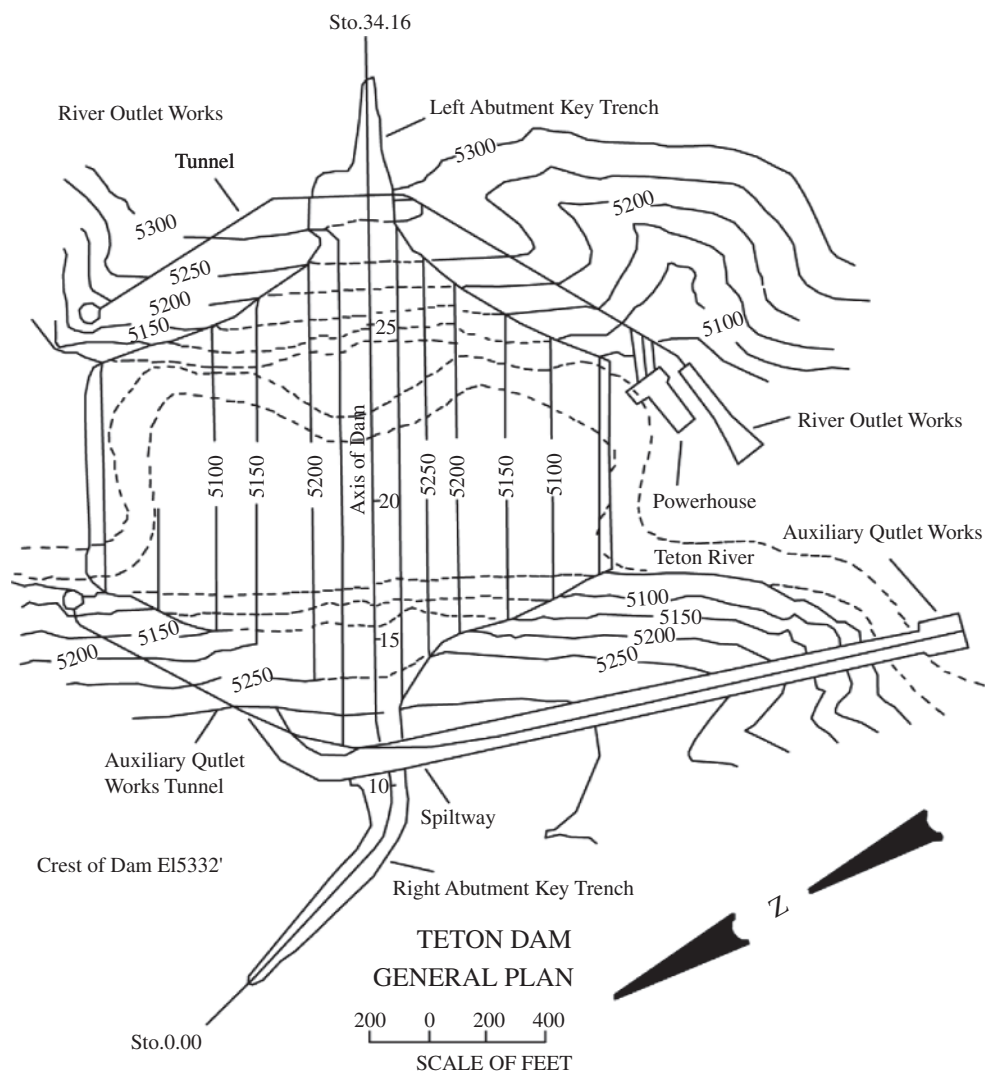


Figure 1.4 General plan of the Teton Dam. (From Smalley, 1992. Reproduced with permission of *Geology Today*)

that eventually caused the dam's collapse. The panel quickly identified piping as the most probable cause of the failure and then focused efforts on determining how the piping started. Two mechanisms were possible. The first was the flow of water under highly erodible and unprotected fill, through joints in unsealed rock beneath the grout cap, and development of an erosion tunnel. The second was "cracking caused by differential strains or hydraulic fracturing of the core material." The panel was unable to determine whether one or both of the mechanisms occurred. "The fundamental cause of failure may be regarded as a combination of geological factors and design decisions

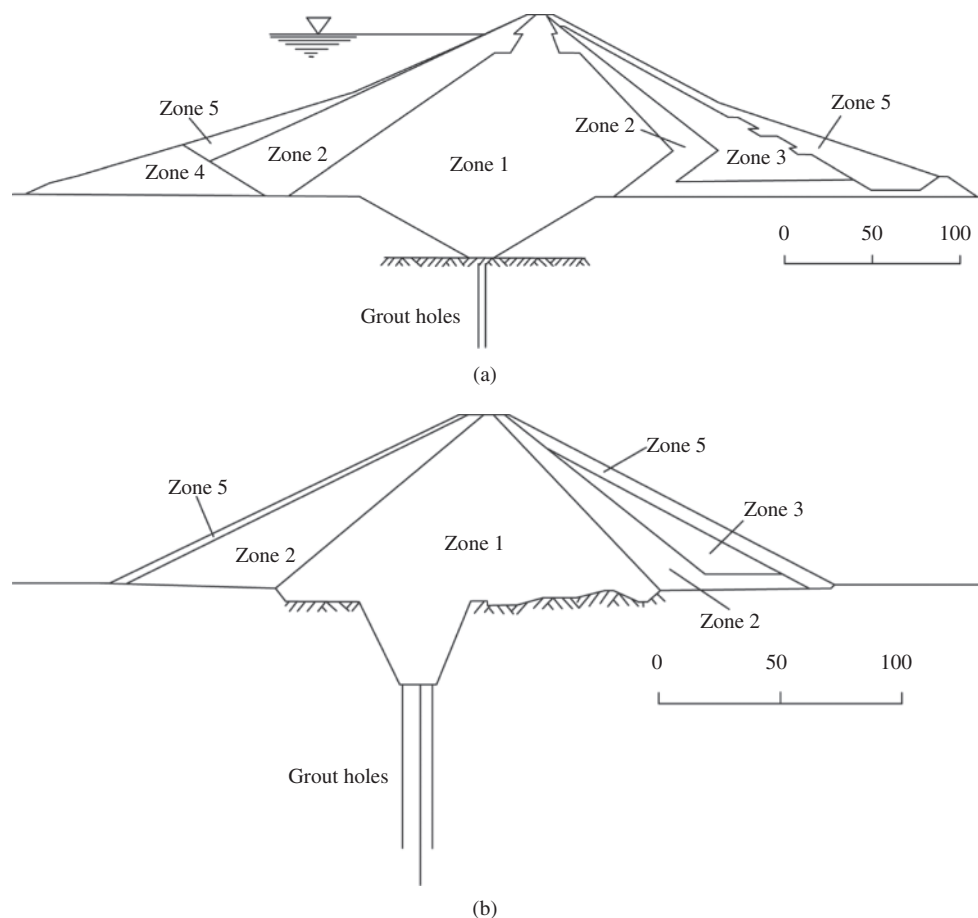


Figure 1.5 Schematic cross sections of the Teton Dam (where Zone 1 is loess core zone; Zone 2 is selected sand, gravel; Zone 3 is miscellaneous fill; Zone 4 is selected silts, sand, gravel, and cobbles; and Zone 5 is selected gravel and cobbles). (a) Center section of embankment and (b) typical abutment section (from Smalley, 1992. Reproduced with permission of *Geology Today*)

that, taken together, permitted the failure to develop.” A wide-ranging controversy erupted from the dam’s collapse. According to the Bureau of Reclamation, Reclamation engineers assess all Reclamation dams under strict criteria established by the Safety of Dams program. Each structure is periodically reviewed for resistance to seismic stability, internal faults, and physical deterioration.

1.4 Erosion Damage of the Balderhead Dam

The Balderhead Dam, constructed in Co. Durham, England in 1961–1965, is an earth-rock fill dam with a narrow vertical soil core (Figure 1.6) and 48 m at its

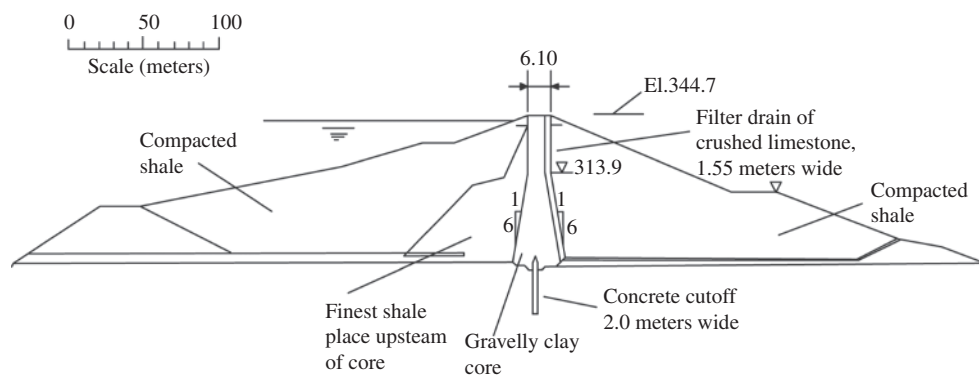


Figure 1.6 Cross section of the Balderhead Dam. (From Sherard, 1973. Reproduced with permission of John Wiley & Sons, Inc.)

maximum height. When completed it was the highest dam in the UK. In April 1967, after the reservoir had been full continuously for more than two years, a subsidence crater appeared near the upstream edge of the crest. Extensive investigations led to the conclusion that concentrated leaks had developed through horizontal differential settlement cracks in the core and that a considerable position of the core material had been damaged by progressive erosion.

The core of the dam was constructed using a gravelly clay compacted layer by layer, 15.2 cm in thickness, and the shells were constructed using the shale excavated with scrapers after ripping and compacted layer by layer, 22.9 cm in thickness. During the first construction season, the shale was compacted in 22.9 cm thick layers with four passes of a grid roller. This was subsequently changed and most of the volume of the shale shells was compacted in 76.2 cm thick wetted layers with a heavy, smooth-wheeled vibrating roller, which was shown in tests to give higher densities. The most weathered shale was placed in a zone directly upstream of the core. The vertical drain at the downstream edge of the core was constructed of less than 7.6 cm diameter crushed, hard limestone, and compacted in 22.9 cm thick layers with a vibrating roller.

The foundation was relatively incompressible. Over most of the length of the foundation was a shale. Portions of the shale length were overlain with a very stiff clay. The maximum thickness of the clay was about 18 m. The clay was left in place under the dam. A concrete cutoff wall with a 1.8 m width was sunk into the shale foundation. The cutoff wall was the full length of the dam in the center of the earth core. The width of the cutoff wall was 2 m (Figure 1.6) and the maximum depth of about 25 m (Figure 1.7).

Reservoir filling commenced in October 1964, and the water level rose relatively slowly to its maximum elevation by February 1966 and remained there until March 1967 (Figure 1.8). As the reservoir was rising in the fall of 1965, the total measured

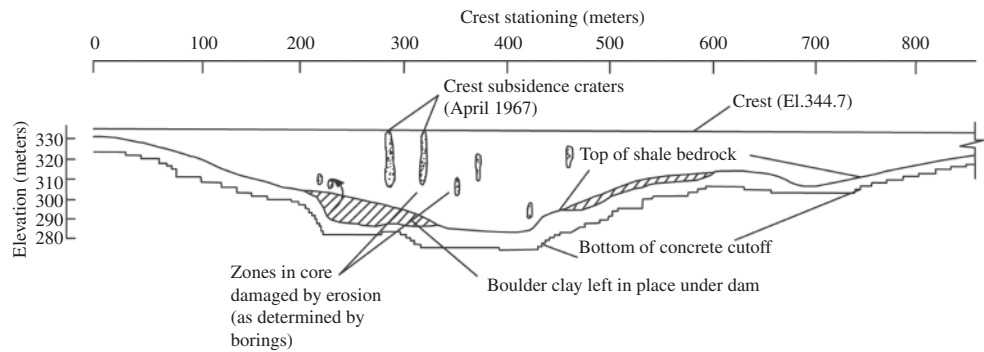


Figure 1.7 Longitudinal section and location of erosion damage zones in core of the Balderhead Dam. (From Sherard, 1973. Reproduced with permission of John Wiley & Sons, Inc.)

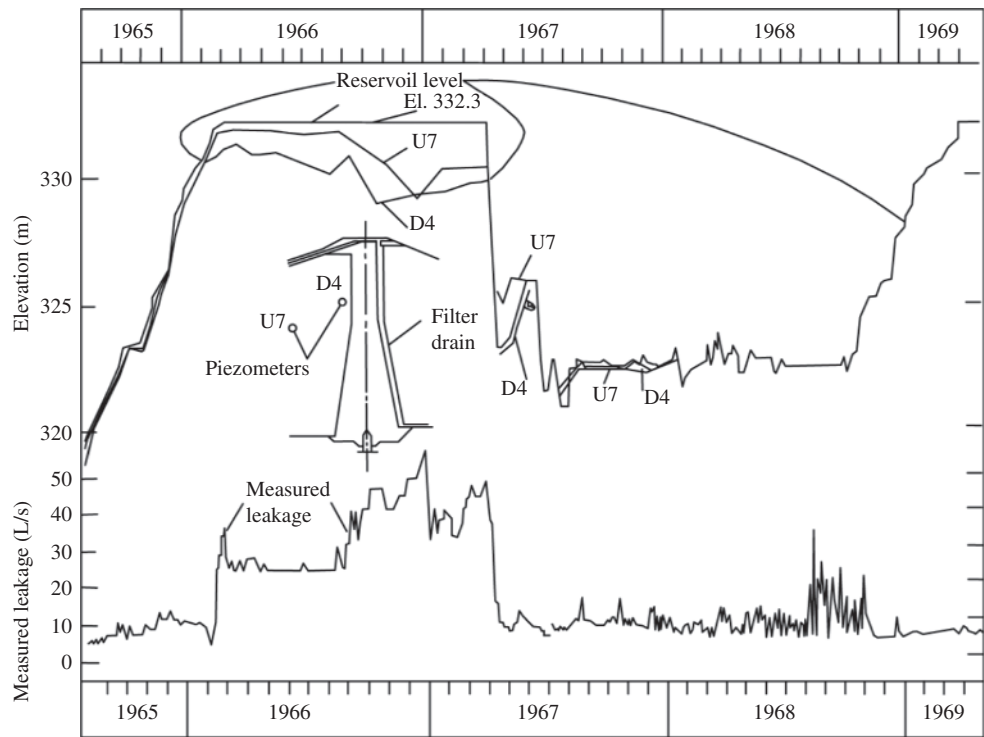


Figure 1.8 Reservoir level, measured leakage, upstream piezometer readings from the Balderhead Dam. (From Sherard, 1973. Reproduced with permission of John Wiley & Sons, Inc.)

seepage from the main under-drain for the full length of dam was about 10 l/s. In January 1966, as the reservoir reached its maximum level, the measured seepage dropped suddenly to 5 l/s for a few days and then abruptly increased again to about 35 l/s by the end of February 1966. Between March and August 1966, the measured leakage was roughly constant at 25 l/s. In August 1966, the measured leakage started to increase again, and to fluctuate considerably, reaching a maximum of about 55 l/s by the end of December 1966, when it abruptly fell to about 30 l/s. At the end of January 1967, a small depression was discovered on the downstream edge of the crest at Station 266. At the beginning of April 1967, a larger subsidence crater, about 3.0 m in diameter and 2.5 m in depth, developed on the upstream edge of the crest at Station 317 (Figure 1.7).

Following discovery of this crater, the reservoir was lowered about 9 m during the first half of April 1967. As the reservoir was lowered the measured leakage fell immediately from 45 to about 10 l/s. Another subsidence crater, similar to that at Station 317, was also developed on the upstream edge of the crest at Station 287 (Figure 1.7). During the reservoir drawdown, the crest in the central part of the dam settled an additional 10–15 cm. A year earlier, starting in the spring or early summer of 1966, when the reservoir was full some of the piezometers located in the upstream shell indicated a gradually reducing water pressure (piezometers D4 and U7 in Figure 1.8). It was apparent in retrospect that these piezometers were measuring a head loss in the semipervious upstream shell and were giving the first indication of concentrated leaks through the core.

Numerous zones of erosion damage were encountered in the core (Figure 1.7). The samples, which were obtained in these damaged zones, and consisted primarily of the coarser particles of the core material, showed that the leakage through the core had washed out the fines. Figure 1.9 shows the best interpretation from the borings of the extent of the damaged zone at Station 317.

The following hypotheses for the mechanics of the failure come from Sherard (1973) and other investigators: (i) The first leaks probably developed in near-horizontal cracks through the core, although the cracks were not in existence before the reservoir was filled. (ii) The cracks were opened by the pressure of the water acting on the upstream face of the core. This was possible because arching action had caused the total pressure on horizontal planes in the core to be low. (iii) The first cracks and leaks probably developed in the core at middle or lower height of the dam, and probably did not develop until the reservoir was nearly full. (iv) Since the upstream shell of the dam is not very pervious, the total volume and velocity of the leakage passing through the cracks was not high. And (v) the leakage gradually eroded the soil, carried the finer particles in the core into the downstream drain, deposited some of the fine material in the drain itself, and carried some of the finer particles out of the drain. The coarser particles of the core material were moved some distance by the leakage but remained in the core. Progressive collapse of the roof of the leakage channels gradually worked upward, and finally caused the subsidence craters on the crest and the extensive zones of erosion damage.

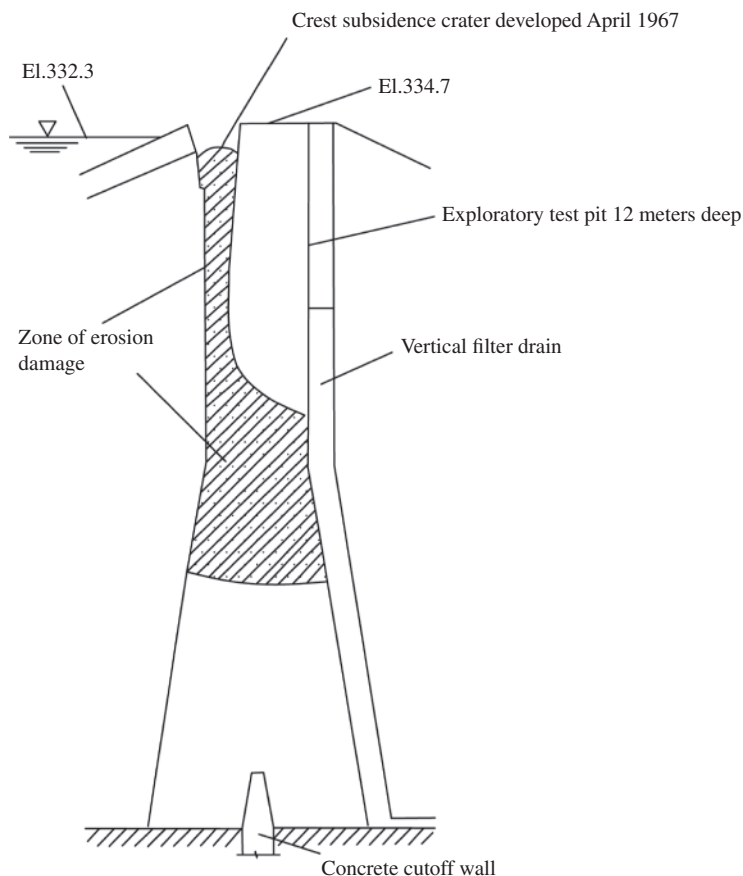


Figure 1.9 Estimated zone of erosion damage in dam core at Station 317 of the Balderhead Dam. (From Sherard, 1973. Reproduced with permission of John Wiley & Sons, Inc.)

1.5 Leakage of the Hyttejuvet Dam

The Hyttejuvet Dam, 90 m in height and 400 m in crest length, was constructed in 1964–1965 in Norway. It has a thin vertical-sided central earth core, two rock-fill shells, and two thick gravel transition zones (Figure 1.10). During the first reservoir filling in the summer of 1966, a concentrated leak of dirty water emerged abruptly at the downstream toe when the reservoir was nearly full. Investigations led to the conclusion that the leak developed through a horizontal differential settlement crack, and that the crack resulted from the arching action of stress in the core. The core was constructed using a clayey sand compacted in 25 cm thick layers. At the end of the 1964 construction season, when the dam was about mid-height, the design was changed and the core was made much thinner, as shown in Figure 1.10(a). The change was made because the construction pore pressures being measured in the core were higher than those anticipated.

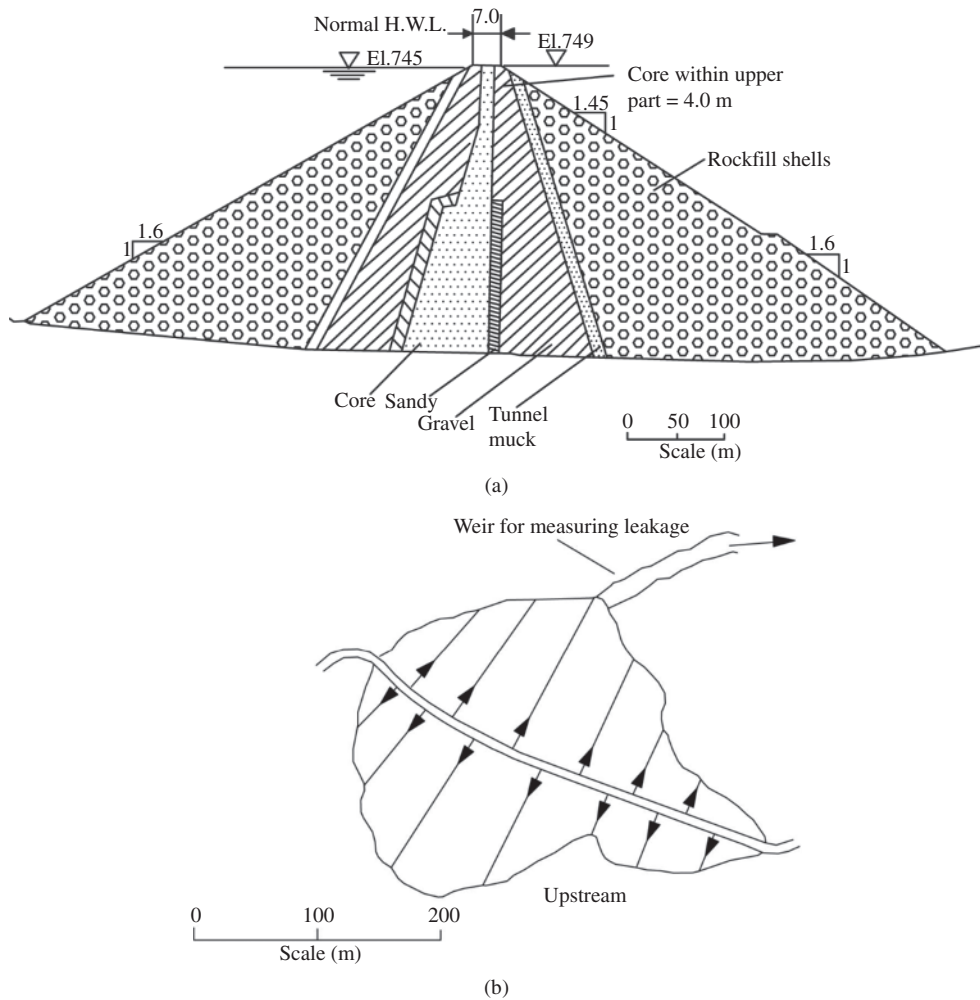


Figure 1.10 Hyttejuvet Dam. (a) Cross section and (b) general plan. (From Sherard, 1973. Reproduced with permission of John Wiley & Sons, Inc.)

The reservoir was filled rapidly for the first time starting in May 1966. As seen in Figure 1.11, very little leakage (less than 2 l/s) appeared at the downstream toe until the middle of August 1966, when the reservoir level had reached to within 7 m of the high water level (Elevation 737). As the reservoir was raised above this level in the last 15 days in August, the leakage increased abruptly, and reached a maximum of about 63 l/s by the end of August, when the reservoir had reached approximate Elevation 740. The leakage water, measures with a weir at the downstream toe (Figure 1.10b), was gray colored and contained approximately 0.1 g of fines per liter. At the beginning, it was not clear that the dirty flow was caused by piping of the core. This is because the water could have been picking up fines as it passed through the rock-fill

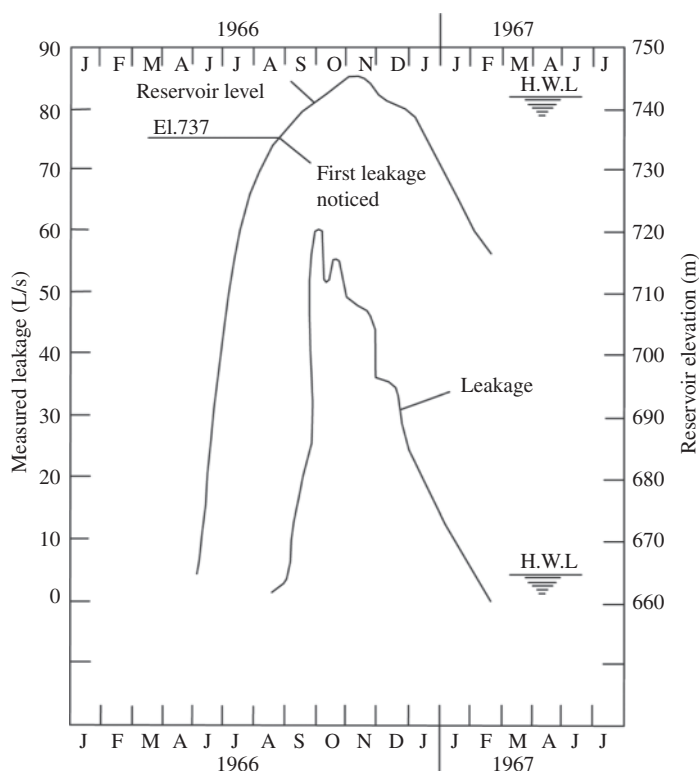


Figure 1.11 Reservoir level and measured leakage 1966–1967 of Hyttejuvet Dam. (From Sherard, 1973. Reproduced with permission of John Wiley & Sons, Inc.)

before emerging at the downstream toe. Later, primarily because of the persistence of the dirty leakage, the investigators concluded that most of the turbidity was caused by piping of the core. During September and early October 1966, the reservoir rose gradually another 5 m to the high-water level. During this time, the measured leakage decreased from about 62 to 45 l/s (Figure 1.11).

During the construction of the dam, in order to measure total vertical stress, a single earth pressure cell was installed in the center of the core 21 m below the dam crest, or 17 m below high-water level. As shown in Figure 1.12, at the end of construction, the pressure measured by the cell was around 17 tons/m² on 30 October 1965. The pressure decreased to about 14 tons/m² by June 1966 and increased again to about 23 tons/m² when the reservoir was full in mid-October 1966. These measurements made it seem probable that an appreciable portion of the total weight of the core had been transferred to the shells by arching action.

After the abrupt development of the leakage, a number of exploratory borings were made through the center of the crest at various points along the 350 m length of the dam. In most of these, water was lost from the holes into the core, especially in

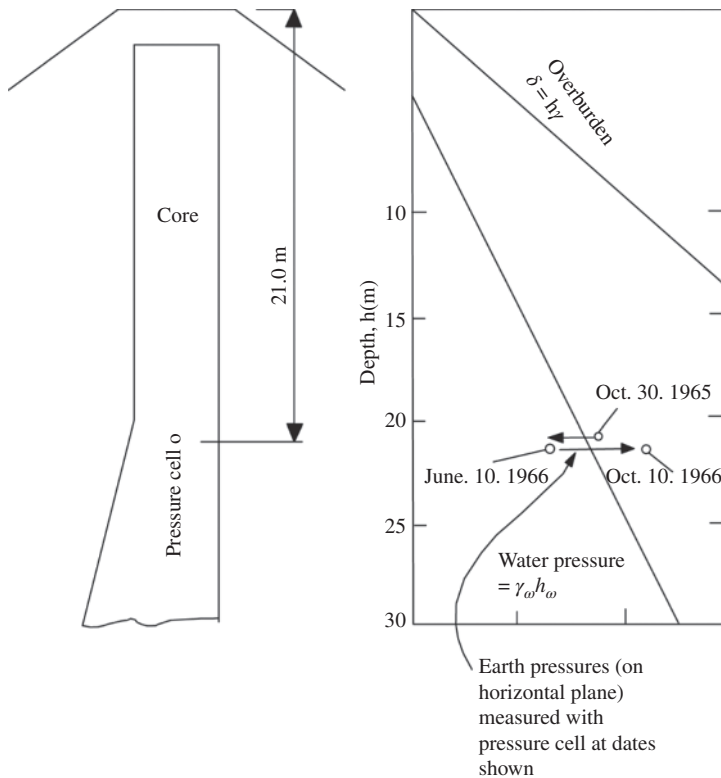


Figure 1.12 Measurement of earth pressure in core of Hyttejuvet Dam. (From Sherard, 1973. Reproduced with permission of John Wiley & Sons, Inc.)

the range of depth from 10–20 m below the crest (between Elevations 730 and 740). From the leakage observations, the low values of vertical total stress were measured by the pressure cells, and the fluid losses measured in the borings. The investigators concluded that there was a reasonable likelihood the leakage was passing through open horizontal cracks in the core. During the summer of 1967, the core was grouted from holes drilled to bedrock or to a maximum depth of 30 m. During the period of grouting in the summer of 1967, the reservoir was full or nearly full and the leakage gradually decreased. At full reservoir state in the summer of 1968, the rate of leakage was 15 l/s and the water was clear. In August 1970, as the reservoir was being lowered from the maximum level, the measured leakage abruptly increased to about 20 l/s and became visibly turbid again.

Although no test pits had been put down in the dam and no cracks were seen on the surface, the engineers investigating the problem (Kjaernsli and Torblaa, 1968; Wood, Kjaernsli, and Höeg, 1976; Sherard, 1973) believed that the most likely explanation for the trouble was that leakage had broken through a horizontal crack. The crack was made possible by arching action of the core between the upstream and

downstream gravel zones. It is not easy to explain why the crack developed. The total post-construction settlement of the crest was relatively low. It seems likely that the most important factor contributing to the cracking was the narrow width and vertical walls of the core. The relatively rapid rate of reservoir filling (about 65 m in 60 days in May and June 1966) was probably another important contributing factor. Furthermore, it may be significant that the core was constructed of a glacial moraine with a wide grain size distribution (extending from coarse gravels to clay-sized particles).

1.6 Self-Healing of Core Cracks

The cracks in the thin seepage barrier of embankment dams may be result from many factors, such as differential settlements, seismic activity, and hydraulic fracturing. Concentrated leaks through the cracks may erode the seepage barrier and lead to embankment dam failure (Dounias, Potts, and Vaughan, 1996; Wan and Fell, 2004; Rice and Duncan, 2010). More than 30% of failures of embankment dams may be attributed to progressive erosion in the seepage barrier (Foster, Fall, and Spannagle, 2000). Since there is no way to assure *a priori* that the core will not crack (in fact, evidence suggests cracking of the core is common due to construction deficiencies, differential settlement, or seismic activity), the downstream filter is usually designed to prevent progressive piping through the core in the event of a concentrated leak. It is so far along that the downstream or outflow filter is considered the primary line of defense.

Current practice involves designing the filter gradation using empirical criteria. The criteria being followed at present for the design of filters have all evolved from an experimental procedure called the “*no erosion filter test*” (Sherard and Dunnigan, 1989). This test involves simulating a crack at the core-filter interface of the dam by inducing flow in a 1-mm diameter hole of the base soil overlying a filter material. Many investigators have reported studies on filter designation, such as Vaughan and Soares (1982), Indraratna and Raut (2006), Indraratna, Raut, and Khabbaz (2007), and Fannin (2008).

If the filter seals with practically no erosion of the base material (a desirable condition), the result is “no erosion,” if the filter seals after “some” erosion of the base material, the result is “some erosion,” and if the filter conveys the eroded material continuously allowing unrestricted erosion, the result is “continuing erosion,” (an undesirable condition). Based on the qualitative results, criteria have been developed to express the “no erosion boundary,” which relates the particle size descriptors of the base and filter soils. Conditions that result in cessation of concentrated leakage and erosion are termed *self-healing* (Zhang and Wang, 2010; Wang *et al.*, 2013).

Self-healing in fractured fine-grained soils was reported by Eigenbrod (2003). The self-healing of concentrated leaks at core-filter interfaces in earthen dams was investigated by Reddi and Kakuturu (2004). And very recently, special attention has been paid to self-healing of core cracks in earthen dams by Kakuturu and Reddi (2006a, 2006b). However, certain conditions do not promote self-healing, but result

in progressive erosion of the core. The mechanism of the self-healing of core cracks in earth-rock fill dams is so far unclear for designers and investigators, further studies on the problem of the self-healing of core cracks is interesting and important.

1.7 Technical Route for Present Study

The conditions and mechanisms to induce hydraulic fracturing should be understood first. Analysis of stress state at the upstream face of the soil core in an earth-rock fill dam may be useful to understand the conditions and mechanisms. The feasibility of *fracture mechanics* (especially linear elastic fracture mechanics) for investigating the problem of hydraulic fracturing should also be assessed. After proving feasibility, the fracture behaviors and characteristics of core soil should be investigated in experimental and theoretical studies. Based on testing results and theories from fracture mechanics, a criterion for hydraulic fracturing could be suggested. The numerical simulation method for hydraulic fracturing is suggested based on presented criteria and the virtual crack extension method. The suggestion of the *numerical simulation* method is based on the *finite element method*, which is widely used for designation and analysis of earth-rock fill dams. Self-healing of the core cracks is very important to the safety of the dam. Factors affecting self-healing, such as the depth of crack, the grain size of base soils, and grain size of filter soils, should also be investigated. The detailed technical route of present study is given by:

1. Conditions and mechanisms inducing hydraulic fracturing.

Based on analyzing the states of the forces and stresses at the upstream face of the soil core of the earth-rock fill dam, the conditions and mechanisms inducing hydraulic fracturing are being investigated. The conditions include the material, mechanical, and filling conditions. The mechanisms behind hydraulic fracturing may be explained based on ideas in fracture mechanics; such that the feasibility of using fracture mechanics to investigate hydraulic fracturing is trialed first. The filling conditions of hydraulic fracturing may be determined by analyzing change in saturation degree in the core.

2. Fracture behaviors of core soil.

In order to investigate the fracture behaviors of the soil mass used to construct the core, a new testing method and testing instrument are suggested based on the standard three-point bending fracture testing method. The fracture behaviors of the core soil under mode I, mode II, and I–II mixed mode loading conditions are investigated through experimental studies. Based on the testing results, a fracture criterion on the testing soil is suggested. The fracture criterion can be used to determine the fracture failure of the testing soil under any of the modes of I, II, and I–II. This fracture criterion is the basis of the hydraulic fracturing criterion suggested later. The feasibility of using linear elastic fracture mechanics to study hydraulic fracturing is also shown by analyzing the fracture behaviors of the testing soil.

3. **Hydraulic fracturing criteria.**

The criteria for hydraulic fracturing are established based on several factors. The factors include the understanding of conditions and mechanisms of hydraulic fracturing, the theories and ideas in fracture mechanics, the states and features of the forces and stress at the upstream face of the core, the fracture behaviors of the core soil, and so on. The established criteria from hydraulic fracturing could explain easily the mechanisms that induce it. The criteria are used to study and solve the problem of hydraulic fracturing, especially to analyze the possibility of hydraulic fracturing occurring during the designation of an earth-rock fill dam. In order to verify the criteria, some theoretical analyses are necessary. The mechanisms of hydraulic fracturing in a cubic specimen and the upstream face of a core are theoretical analyzed based on the criteria.

4. **Numerical simulation methods for hydraulic fracturing.**

A new method to simulate hydraulic fracturing is suggested based on the conventional finite element method, the J integral in fracture mechanics, and the virtual crack extension method suggested by Hellen (1975). The method is used to determine the occurrence of hydraulic fracturing, and simulate the propagation of cracks under water pressure. The method can analyze hydraulic fracturing, and analyze the stress-deformation behaviors of the dam body during construction and filling at the same time. The re-establishing of the finite element mesh is not necessary in the new method. In order to verify the method, some comparisons between the results from the numerical method and those from theoretical formula are investigated.

5. **Factors affecting hydraulic fracturing.**

The influence of dam structure and materials on hydraulic fracturing is investigated by analyzing the stress arching action in the core. The stress arching action can be analyzed using the conventional method of three-dimensional finite element analysis. Many factors, such as crack depth, crack position, water level, and properties of the core, may affect the occurrence of hydraulic fracturing. The numerical simulation method suggested in the study is used to analyze the influence of the factors.

6. **Self-healing of core cracks.**

Many factors, such as the depth of crack, the characteristics of base soils and filter soils, may affect the self-healing of the crack in the seepage barrier of the embankment dam. In order to investigate the influence of the factors, the embankment dam is simplified to a five-layer structure, and a cylindrical sample with a five-layer structure is suggested.

7. **Application.**

As an example to analyze use of the suggested numerical simulation method, the stress-deformation behavior, and resistance to hydraulic fracturing of the Nuozhadu Dam, an earth-rock fill dam being contracted in Western China, are all investigated.

1.8 Summary

The characteristics of two main types of the embankment dams, rock-fill and earth-fill dams, are introduced in this chapter. The rock-fill dam with a soil core is called the *earth-rock fill dam* in this book. The problem of hydraulic fracturing in earth-rock fill dams is elaborated upon. Some typical examples related to hydraulic fracturing – the complete failure of the Teton Dam, the erosional damage of the Balderhead Dam, and the leakage of the Hyttejuvet Dam – are illustrated. Self-healing of core cracks, which may also be important to the safety of the dam, is described. The technical route of present study, which includes seven steps, is given.

References

- Dounias, G.T., Potts, D.M. and Vaughan, P.R. (1996) Analysis of progressive failure and cracking in old British dams. *Geotechnique*, **46**(4), 621–640.
- Eigenbrod, K.D. (2003) Self-healing in fractured fine-grained soils. *Canadian Geotechnical Journal*, **40**, 435–449.
- Fannin, J. (2008) Karl Terzaghi: from theory to practice in geotechnical filter design. *Journal of Geotechnical and Geoenvironmental Engineering, ASCE*, **134**(3), 267–276.
- Foster, M., Fell, R. and Spannagle, M. (2000) The statistics of embankment dam failures and accidents. *Canadian Geotechnical Journal*, **37**, 1000–1024.
- Hellen, T.K. (1975) On the method of virtual crack extensions. *International Journal for Numerical Methods in Engineering*, **9**, 187–207.
- Independent Panel to Review Cause of Teton Dam Failure (1976) Report to U.S. Department of the Interior and the State of Idaho on Failure of Teton Dam, U.S. Bureau of Reclamation, Denver, CO.
- Indraratna, B. and Raut, A.K. (2006) Enhanced criterion for base soil retention in embankment dam filters. *Journal of Geotechnical and Geoenvironmental Engineering, ASCE*, **132**(12), 1621–1627.
- Indraratna, B., Raut, A.K. and Khabbaz, H. (2007) Constriction-based retention criterion for granular filter design. *Journal of Geotechnical and Geoenvironmental Engineering, ASCE*, **133**(3), 266–276.
- International Energy Agency (IEA) (2008) *World Energy Outlook 2007*, IEA, Paris, p. 600.
- International Journal of Hydropower and Dams (IJHD) (2005) *World Atlas and Industry Guide*, IJHD, Wallington, Surrey, p. 383.
- Jaworski, G. W., Duncan, J. M., and Seed, H. B. (1981) Laboratory study of hydraulic fracturing. *Journal of the Geotechnical Engineering Division, ASCE*, **107**(GT6), 713–732.
- Kakuturu, S. and Reddi, L.N. (2006a) Evaluation of the parameters influencing self-healing in earth dams. *Journal of Geotechnical and Geoenvironmental Engineering, ASCE*, **132**(7), 879–889.
- Kakuturu, S. and Reddi, L.N. (2006b) Mechanistic model for self-healing of core cracks in earth dams. *Journal of Geotechnical and Geoenvironmental Engineering, ASCE*, **132**(7), 890–901.
- Kjaernsli, B. and Torblaa, I. (1968) *Leakage through Horizontal Cracks in the Core of Hyttejuvet Dam*, Publication 80, Norwegian Geotechnical Institute, Oslo, Norway, pp. 39–47.
- Lo, K.Y. and Kaniaru, K. (1990) Hydraulic fracture in earth and rock-fill dams. *Canadian Geotechnical Journal*, **27**, 496–506.
- Ministry of Water Resources, People's Republic China and National Bureau of Statistics, People's Republic China (2013) *Bulletin of First National Census for Water*, China Water Power Press, Beijing.
- Reddi, L.N. and Kakuturu, S.P. (2004) Self-healing of concentrated leaks at core-filter interfaces in earth dams. *Geotechnical Testing Journal*, **27**(1), 89–98.
- Rice, J.D. and Duncan, J.M. (2010) Findings of case histories on long-term performance of seepage barrier in dams. *Journal of Geotechnical and Geoenvironmental Engineering, ASCE*, **136**(1), 2–15.

- Sherard, J.L. (1973) in *Embankment Dam Engineering* (eds R.C. Hirschfeld and S.J. Poulos) Casgrande Volume, (eds), John Wiley & Sons, Inc, New York, pp. 271–353.
- Sherard, J.L. (1986) Hydraulic fracturing in embankment dams. *Journal of Geotechnical Engineering, ASCE*, **112**(10), 905–927.
- Sherard, J.L. and Dunnigan, L.P. (1989) Critical filters for impervious soils. *ASCE Journal of Geotechnical Engineering*, **115**(7), 927–947.
- Smalley, I. (1992) The Teton Dam: rhyolite foundation + loess core = disaster. *Geology Today*, **8**(1), 19–22.
- Tortajada, C. (2008) Water Management for an Increasingly Complex and Interrelated World. Hydropower and Dams, World Atlas.
- Vaughan, P.R. and Soares, H.F. (1982) Design of filters for clay cores of dams. *Journal of Geotechnical Engineering Division, ASCE*, **108**(1), 17–31.
- Wan, C.F. and Fell, R. (2004) Laboratory tests on the rate of piping erosion of soils in embankment dams. *Geotechnical Testing Journal*, **27**(3), 295–303.
- Wang, J.J., Zhang, H.P., Zhang, L. and Liang, Y. (2013) Experimental study on self-healing of crack in clay seepage barrier. *Engineering Geology*, **159**, 31–35.
- Wang, J. J., Zhu, J. G., and Zhang, H. (2005) Some ideas on study of hydraulic fracturing of core of earth-rock fill dam. *Chinese Journal of Rock Mechanics and Engineering*, **24**(S2), 5664–5668 (in Chinese).
- Wood, D. M., Kjaernsli, B., and Höeg, K. (1976) Thoughts concerning the unusual behaviour of Hyttejuvet Dam. 12th International Congress on Large Dams, Mexico City, Mexico, Vol. **2**, pp. 391–414.
- Zhang, L. and Wang, J. J. (2010) Core crack in earth-rock fill dam and mechanism of its self-healing. Proceedings of the First National Conference on Safety of High Dams in China, Nanjing, China, pp. 316–320 (in Chinese).
- Zhu, J. G. and Wang, J. J. (2004) Investigation to arcing action and hydraulic fracturing of core rock-fill dam. Proceedings of the 4th International Conference on Dam Engineering – New Developments in Dam Engineering, Nanjing, China, pp. 1171–1180.

