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CASE HISTORIES OF EARTHEN DAM FAILURES

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ABSTRACT

Historical study of dams conceived in earlier times is essential. To continue advancing, the engineering profession must periodically review past problems and the lessons that they taught. Candid sharing of information on failures as well as successes is needed. In fact, some of the most valuable learning has come from projects where errors have been clear in retrospect. In recent years, dam safety has drawn increasing attention from the public. This is because floods resulting from breaching of dams can lead to devastating disasters with tremendous loss of life and property, especially in densely populated areas. Past dam failure disasters showed that the loss of life in the event of a dam failure is directly related to the warning time available to evacuate the population at risk downstream of the dam. Earth and rock fill dams are widely used throughout the world, and most of the dam failures involve such dams. To speak about failures of dams without a brief account of these happenings in the dam world is not possible. Therefore, it is essential to go through the case histories of such dam failures to understand the causes of failures of the dams failed in the past. The main causes of failures of such dams are attributed to overtopping, internal erosion and piping. There are excellent sources and case studies are available in the literature related to failure of the earthen dams due to overtopping, internal erosion and piping. The purpose of this paper is to highlight the most promising causes of the earthen dam failures and present the case histories of the dams failed in the world due to these causes. The case histories reported in this paper are chosen not for the entity of the damage occurred, but are representative of the body of knowledge that has been accumulated in the interest of the future safety of dams.

1. INTRODUCTION

Case histories in geotechnical engineering serve a number of useful purposes, one of which is to provide real data against which designers can test their predictions of behavior. There is ample evidence to indicate that, despite the many advances made in geotechnical engineering and engineering science in the past the designer's ability to predict the behavior of designed structures accurately has not increased. The reasons for this apparent lack of improvement are numerous, and perhaps it will always be as difficult to make accurate geotechnical predictions as it is to make predictions of human behavior.

In considering predictions related to geotechnical engineering, Lambe (1973) classified the predictions as shown in Table 1.

Our professional literature contains the results of more type C_1 predictions than of any other type. Autopsies can of course be very helpful in contributing to our knowledge. However, one must be suspicious when an author uses type C_1 predictions to 'prove' that any prediction technique is correct.

Table 1 Classification of prediction

Prediction type	When prediction made	Results at time prediction made
A	Before event	-----
B	During event	Not known
B_1	During event	Known
C	After event	Not known
C_1	After event	Known

In an attempt to present the type C_1 predictions, this paper considers a number of case histories related to embankment dam failures. Failure of a dam can result in a major disaster with devastating losses of both human life and property. The phenomenon is time-dependent, multiphase (water-soil interaction), and non-homogeneous (different materials, various degrees of soil compaction, and so on). Hydraulics, hydrology, sediment transport mechanism, and structural and geotechnical aspects are all involved in dam failures. Erosion of an earth-dam can be primed by low or weak points on the crest or on the downstream face, by piping or overtopping.

Progressive erosion then widens and deepens the breach, increasing outflow and erosion rate.

Internal erosion and piping has historically resulted in about 0.5% (1 in 200) earthen dams failing, and 1.5% (1 in 60) experiencing a piping incident. Of these failures and accidents, about half are in the embankment, 40% in the foundations, and 10% from the embankment to foundation (Foster et al. 1998, 2000a, b). Fewer incidents of piping in the foundation, and particularly from embankment to foundation, progress to failure, than for piping in the embankment. About two thirds the failures occur on first filling or in the first five years of the operation.

A number of studies have been devoted to investigating dam failures. Research on failure case histories and the resulting evolution of safety philosophy and practice is, and continues to be, a very dynamic process. Any advance in it provides answers to some pending questions, while others remain open and new ones are coming up. It cannot be possible to speak about failures of dams without a brief account of these happenings in the dam world.

In this paper an attempt has been made to discuss the main causes of failure of earthen dams - overtopping, internal erosion and piping and present the case histories of the earthen dams failed in the world due to these causes. The case histories reported in this paper are chosen not for the entity of the damage occurred, but for their representative characteristics.

2. MAIN CAUSES OF EMBANKMENT DAMS FAILURE

From the above discussion it is apparent that the main causes of failures of embankment dams are closely related to the erosion of embankment materials caused by either overtopping or seepage erosion/piping. Ralston (1987) discussed the mechanism of embankment erosion from overtopping. For non-cohesive embankments, materials are removed from the embankment in layers by tractive stresses. The erosion process from overtopping begins at a point where the tractive shear stress exceeds a critical resistance that keeps the material in place. For cohesive embankments, breaching takes place by head cutting. Usually, a head cut initiates near the down-stream toe of the dam, and then advances upstream until the crest of the dam is breached. The basic erosion mechanisms and erosion rate as pointed out by Singh (1996) are different for granular and cohesive embankments. For granular embankments, surface slips take place quickly due to the seepage existing on the downstream slope; and hence granular materials are removed rapidly layer by layer. For cohesive embankments, no seepage exists on the slope because of the low permeability. Instead, erosion often begins at the embankment toe and advances upstream, undercutting the slope and in turn causing the removal of large chunks of materials due to tensile or shear failure of the soil on the over-steepened slope.

Other than overtopping, internal erosion and piping are another common mode of failures of embankment dams. Piping phenomenon as defined by ASTM (2002) is the progressive removal of soil particles from a mass by a percolating water, leading to the development of channels. According to McCook (2004), seepage erosion occurs when the water flowing through cracks or defect erodes the soil from the walls of the crack or defect. Internal erosion and piping can be divided into four phases: initiation and continuation of erosion, progression to form a pipe and formation of a breach (Fell et al., 2003). In general, the seepage erosion/piping failure initiates when the erosion/piping resistant forces are smaller than the erosion/piping driving forces, resulting in the removal of soil particles through large voids or existing discontinuities in soil. After a large amount of embankment materials has been washed away by seeping flow, a free path named “pipe” is formed through the dam. Then, the erosion advances quite rapidly until the portion of the materials above the pipe becomes unstable and collapses. After the collapse, the subsequent erosion proceeds in the same fashion as in the case of overtopping (Xu and Zhang, 2009).

3. FAILURE CAUSED BY OVERTOPPING

Overtopping is by far one of the most frequent causes of dam failures, in particular for embankment dams. When overtopping hits embankment dams, the effects can turn into disasters. According to the international committee on large dams (ICOLD, 1995), and the work of Foster et al. (2000), one –third or more of the total identified failures was caused by dam overtopping. Overtopping of a dam is generally the consequence of an extreme flood event and is often a precursor of partial or complete dam failure. The analysis of case histories of this cause of dam failure reveals the inadequacy of formerly used hydrological methods to estimate extreme floods and the specifications for the selection of the spillway design conditions. Recently the advances on hydrology and on climatic processes have allowed obtaining better estimations of extreme flood events with a reduction of overtopping occurrence. Hence, hydrological reliable data are essential for dam safety and criteria of minimum risk have to be assumed in the evaluation of the design flood.

In this section, three case histories of earthen dam failures due to overtopping are briefly presented. Failures of these dams are related to the undersize of outlets and spillways, flood gates operation and human errors.

3.1 Belci Dam Failure

The Belci dam, a clay core earth-fill structure provided by an upstream concrete facing, was built in 1962 on Tazlaur River, near Slobozia in Romania (Diacon et al., 1992; Vogel, 1993). This dam was 18.5 m high, 432 m long earthen structure with a storage capacity of 12.7 Mm^3. The hydrological measurements for the estimation and pre-calculation of design floods for this dam had only been collected from a gauging station 10 km upstream from the dam site for ten years prior to construction. During its 29 years of operation, floods occurred on Tazlaur River with peak values much more than those estimated at Belci dam. On 7 July 1970 a peak inflow of 980 m3/sec caused overtopping of the dam and part of the left wing was eroded. After further floods on 29 May 1971 and in August 1979 with peak values of $890m^3$/sec and $855m^3$/sec respectively, a new calculation for the design floods was commissioned. However the spillway capacity was never changed because at the same time the risk classification of the dam was reduced.

On 28 July 1991 heavy rainfall occurred, which caused the failure of main telephone lines and power supply. Due to failure of the main telephone lines no prediction of flood warnings could be sent from the upstream catchment areas to the dam site. The bottom outlet could not be opened more than 40cm due to power failure at the site. Emergency power was also unavailable and the gates could not be opened manually as timber and debris were blocking the bottom outlet. At 02.15 am on 29 July the dam began to overtop and the reservoir was emptied by 07.15 am. Twenty five people were killed by the flood wave caused by the dam break and 119 houses were destroyed.

The peak inflow of 1200 m^3/sec measured by a water gauge located at downstream was lower than the later estimated 1-in-100-year flood of 1515 m^3/sec. The initial dam break occurred at the same point where the dam had been affected by erosion in 1970. The final size of the breach was about 112 m long and 15 m deep (fig.1). Post-failure measurements of the intact dam crest near this initial break showed that the repair works had produced the effect of a natural overflow section. A cable trench that had been dug along the dam crest also had a negative and accelerating influence with regards to the process and dynamics of the breach formation. The dam has not been reconstructed.

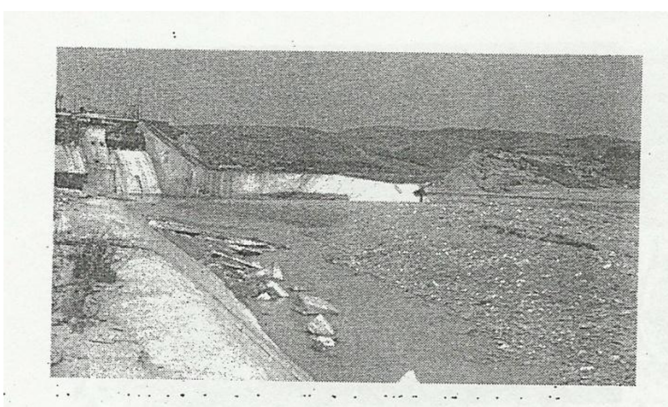

Fig. 1. *The Belci dam after failure.*

3.2 Tous Dam Collapse

Tous dam was a 70 m high rockfill dam with a central clay core located near Valencia, Spain, failed due to overtopping. It was designed and built as a flood defense structure being also used for regulation and irrigation. Its construction was started in 1958 following a project of a concrete dam 80 m tall. During foundation works geotechnical conditions revealed a problem and construction was stopped in 1964. It was continued in 1974 with a modified project, in which the central part had been changed to loose material design with clay core and finally finished in 1978. It was earth-gravity dam 70 m tall with 400 m crest length. The dam was provided with radial gates to regulate the spillway whose capacity was 7000 m^3/s; the bottom outlet had a capacity of $250m^3$/s. During October 19 and specially 20, 1982, heavy rain took place in the Jùcar basin close to Tous dam. Heaviest rain was recorded in the Cofrentes area, about 25 km north-west of Tous dam. Total rainfall in Cofrentes exceeded 550 mm with 285 mm falling in only 3 hours. The estimated inflow was 5000 m^3/s and the gates of the spillway were to be opened. Unfortunately, the electric network was out of order due to the weather conditions; moreover, of the two emergency diesel generators, one was under repair and the other could not be started. Efforts to raise the gates manually were fruitless.

The overtopping started at 17.00 pm; the water overcame the dam reaching about 1.10 m above the crest at 19.15 pm. About 16 h after recognizing the impossibility of overtopping the flood gates, the dam was overtopped and washed out after 1 h by erosion of a greater part of the shoulders and of the central rock-fill. After such an extraordinary flood, in the downstream basin 8 people lost their lives and about 100,000 people had to be evacuated. The damages were estimated to reach 400 M$, even if part of these damages were likely to be caused by the floods before the arrival of the break wave (fig. 2).

A new Tous dam was built on the same site and part of the clayey core material, which had shown a relatively high resistance to water flow, was reused for constructing the new dam.

Fig. 2. *The Tous dam after failure.*

3.3 Taum Sauk Dam Failure

The Taum Sauk dam (upper reservoir) suffered failure on 14 December, 2005, located in Missouri State (USA), is part of a pumped Storage Hydroelectric Power Plant which was constructed from 1960-1962 and began operation in 1963. The lower reservoir was formed by constructing a 60 feet high concrete gravity dam along the East Fork of the Black River 3 miles upstream of Lesterville, MO. The upper reservoir was sited on Proffit Mountain, approximately 800 feet above the lower reservoir and connected by a 7,000 feet long tunnel. The majority of the upper reservoir's rockfill embankment appears to have been constructed through simple end dumping of the excavated material. High rates of settlement were experienced at the upper reservoir during the first four and a half years of operation and it continued up to the time of failure in December 2005, with differential settlements approaching two feet along the crest of the reservoir's parapet wall. Since its construction, the reservoir suffered minor leaks that were reduced when in 2004 a geo-synthetic liner was installed on the upstream facing of the dam.

A variety of design/construction flaws, instrumentation error, and human errors contributed to a catastrophic failure of the upper reservoir on Dec 14, 2005. Malfunctioning and improperly programmed/placed sensors failed to indicate that the reservoir was full and didn't shut down the facility's pumps until water had been overflowing for 5-6 minutes. This overflow undermined the parapet wall and scoured the underlying embankment, leading to a complete failure within 5-6 minutes due to overtopping. Figure 3 shows the breach through embankment of the dam. The peak flow from this event is estimated at 289,000 cfs.

Fig. 3. *Breach through Taum Sauk embankment*

The Federal Energy Regulatory Commission found guilty owner's of the dam for his decision to continue operating the dam despite knowing the malfunctioning of the sensors and fined $15 million; the largest fine ever accessed by the agency.

4. FAILURE CAUSED BY INTERNAL EROSION IN THE DAM BODY AND FOUNDATION

Internal erosion and piping through a dam body or its foundation is one of the most important factors which define the safety structural condition and can represent a serious source of troubles. Piping can occur in the embankment, through the foundation and from the embankment into the foundation as a progression of internal erosion caused by seepage. In the case of piping failure, the incidence of piping through the embankment is two times higher than piping through the foundation and twenty times higher than piping from the embankment into the foundation (Foster et al., 2000). Further, it was noticed that half of all piping failures through the embankment are associated with the presence of conduits. The different modes of piping associated with conduits are piping into the conduit, along and above the conduit or out of the conduit (Fell et al., 2005). Other than conduit the internal erosion in the dam body can be caused by settlement cracks or even passages created by animals. Any leakage does not have to be underestimated and has to be carefully detected since quick erosion may increase initial minor defects and can become potentially dangerous. In this section the case history of Teton dam and Baldwin Hills dam failures due to erosion in the dam body and foundation respectively are briefly presented.

4.1 Teton Dam Failure

Teton Dam, a 93 m (305 ft.) high with a crest length of 975 m (3200 ft) earth fill dam across the Teton River in Madison County, southeast Idaho, failed completely during first filling at 11:57 AM on June 5, 1976. The water surface was 9 m below the crest of the dam. Failure was initiated at 7.30 AM by a large leak near the right (northwest) abutment of the dam, about 39 m (130 ft) below the crest. The structure was breached 4.30 h later as a result of internal erosion (Fig. 4), causing the loss of 11 lives and extensive flooding in the farmland and towns below the dam. Peak flow at the time of breaching was estimated at 42,500 m^3/s.

The dam, designed by the U.S. Bureau of Reclamation, failed just as it was being completed and filled for the first time. This failure of a modern dam so soon after construction was a shock to the engineering community. It prompted one of the most intensive investigations of any dam failure. A panel of experts investigated that the failure of dam was related to erosion and piping phenomena which occurred in the key trench fill on the right abutment possibly caused by seepage through cracks.

The expert's panel supposed that the seepage occurred due to either deficiencies of grouting in the sealing of rock joints or differential settlements in the key trench fill itself, or a combination of both causes. Further investigations carried out on the remains of the left embankment detected the presence of 'wet seams' (horizontal lenses varied in thickness from 75 to 200 mm) in some of the construction layers. Such wet seams possibly caused by the unsuccessful attempts to mix dry and wet soils during the construction of core layers, thus creating low-density zones undetected during the earth work controls. The seepage through the key trench fill could have been the seat of intense flow leading to erosion and piping of the core.

Among the deficiencies of the design, one particularly relevant aspect was the lack of an appropriate defensive technical solution able to cope with the possible different modes of failure. Moreover, the needs of an effective control during the different construction phases have to be once more emphasized.

Fig. 4. *The Teton dam after failure.*

Investigations by commissions and boards together with recriminations, typical of most disasters, followed the failure of Teton Dam. Lessons were learned, but no attempt has been made to rebuild the dam. Its remnant sits today in silent testimony that "Nature bats last."

4.2 Baldwin Hills Dam Failure

The Baldwin Hills reservoir was a basin having four sides, carved and constructed on the top of the highest hill in the north-west-south-west chain of ridges in Los Angeles Country, known as the Newport-Inglewood (ICOLD, 1974). The reservoir, approximately rhomboid in shape, consisted of compacted earth dykes on three sides, whereas the fourth and north side was closed by the main dam, 71 m high with a crest length of 198 m, designed as a homogeneous earth fill. The construction of the reservoir started in January 1947 and was placed in service in 1951 with a purpose of providing water to the northwest part of the city of Los Angeles. The reservoir had a storage capacity of 1.1 Mm^3 and was served by inlet and outlet conduits in tunnels through east side. An impervious compacted clay blanket was used to cover all excavated slopes and constructed embankments. The blanket was 10 feet thick on the reservoir floor, tapering up the slopes to a lesser thickness. Under the blanket there was a four inch thick porous concrete drainage layer which was placed over an asphalt seal coating.

After 12 years of operation, on December 14, 1963, at about 11:15 A.M., an unprecedented flow of water was heard in the spillway pipe at Baldwin Hills Dam. The water came from drains under the reservoir lining. At approximately 1:00 P. M., muddy leakage was discovered downstream from the east abutment of the dam, which formed the north side of the reservoir. At 2:20 P. M. lowering of the reservoir water level revealed a 3-ft-wide break in the reservoir's inner lining. A futile attempt was made to plug the hole with sandbags. Water broke violently through the downstream face of the dam. By 5:00 P.M., the reservoir had emptied, revealing a crack in the lining extending across the reservoir bottom in line with the breach in the dam (Fig. 5).

Mr. R. B. Jansen, Chairman Engineering Board of Inquiry Baldwin Hills Reservoir Failure, immediately after failure of the dam constituted a panel of experts to investigate the cause of failure of the dam. Expert’s panel in his investigation report stated that a gradual deterioration of the foundation took place during the life of structure and culminated with sudden failure on December 14, 1963. The porous concrete drain was damaged by early small movements at the fault, and leakage water found its way into the fault. These earth movements were mainly caused by land subsidence, locally concentrated along the fault which was a weak plane. During the life of the reservoir, erosion took place in the fault under the undamaged blanket and partially damaged drain. The narrow width of the fault permitted the porous concrete drain to span openings that were developing under the drain. These occurrences were gradual and progressive. The perviousness of the fault permitted the water to disappear into the hill without emerging on the downstream abutment. Movement occurred at the fault on December 14, rupturing the impervious blanket and admitting full reservoir pressure to the fault and to the drainage system for the first time. The full reservoir pressure in the fault forced an outlet to the surface at a point low down on the east abutment of the main dam. Flow developed in the pervious and erodible fault zone and foundation rock. The flow and erosion increased rapidly, a cavernous opening piped through the abutment, the overlying foundation and embankment collapsed into this opening and the reservoir drained quickly and completely.

In summary, the reservoir and its immediate environs were subjected to many adverse forces, including horizontal and vertical displacement due to subsidence; local breaking of the weak foundation; some erosion at the faults and rebound effects due to oil field re-pressurization, reservoir loading and unloading in 1951 and 1957, and the final inrush of water into the Fault I-II zone at time of failure.

From the time it was placed into service in 1951, Baldwin Hills Reservoir had been regarded as a model of engineering excellence and a source of pride to its builder and owner, the Los Angeles Department of Water and Power. In spite of careful design, construction and constant surveillance, the reservoir failed, and lessons are to be learned from the failure. Facing the same site conditions now, and equipped with the knowledge of what went wrong at Baldwin Hills, designers could make several improvements. The obvious first step would be to avoid rigid drains so close to the water face and to the unstable and erodible foundation. Drains should be amply sized and provided with access, where possible, to facilitate maintenance, and earth linings should have appreciable plasticity. Erodible embankment and foundation elements must have adequate filter protection. Foundations in erodible

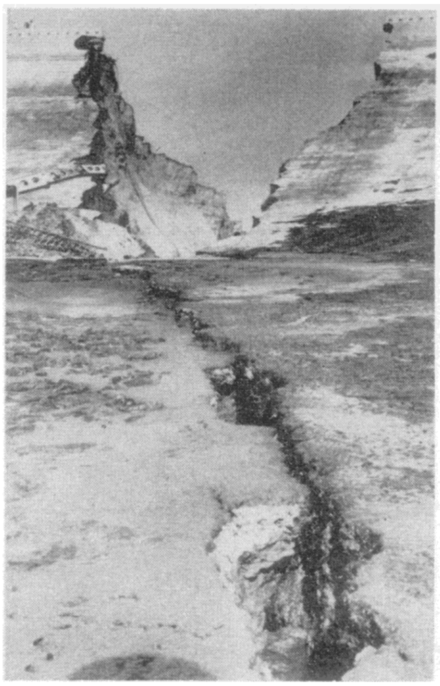

Fig. 5. *Breached Baldwin Hills dam Embankment.*

rocks must be thoroughly explored to disclose any pre-existing cavities or other defects. Finally, the use of heavy construction equipment must be carefully controlled to avoid the damage of critical reservoir feature on soft foundations.

5. CONCLUSION

The case histories reported in this paper refer to failures that happened due to overtopping and internal erosion and piping and they have been chosen not for the entity of the damage occurred, but are representative of the body of knowledge that has been accumulated in the interest of the future safety of dams. These case histories are a useful lesson for owners, designers and builders.

In the last fifty years a tremendous progress has been made in the development of dam design, construction and operation. Due to greatly improved technology as well as to the lessons learned by the careful analysis of incidents and failures, the number of dam failures has been considerably reduced. Most of these lessons have given rise to the improvement of safety criteria and have been taken into account within national legislations devoted to dam safety and international recommendations that represent a reference to the whole dam engineering community.

Throughout the world, legislation and safety criteria for dams vary quit significantly. For public safety along the valley downstream of dams as well as for the protection of economical and environmental resources, the majority of the contemporary safety legislation and technical guidelines promote and support dam-break flood risk management is practical and important issue.

Despite the increasing safety of dams due to improved engineering knowledge and better construction quality, a full non-risk guarantee is not possible and an accident can occur, triggered by natural hazards, human actions or just because the dam is losing strength capacity due to its age.

To mitigate the dam-break risk enforcement of safety control measures at dam site, implementation of emergency planning and preparedness measures in the downstream valley, early warning system, rescue and relief measures are essential. Emergency planning and effective warning systems are now mandatory issues in modern dam safety regulations. However, these measures need to be implemented with support of local authorities and adequate public information and participation.

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