Chapter 1

Dam failures

C. Deangeli[1], G.P. Giani[2], B. Chiaia[1] & A.P. Fantilli[1]
[1]*Politecnico di Torino, Turin, Italy*
[2]*State University of Milan, Milan, Italy*

Abstract

Dams may either be human-built or result from natural phenomena, such as landslides or glacial deposition. The majority of dams are human structures generally constructed of earthfill/rockfill or concrete. Man-made dams are obstacles to naturally flowing water and are built for many purposes, such as water storage for potable water supply, livestock water supply, irrigation, hydro-electricity, prevention of flooding, raising the water level to provide a head which can be turbined in an electricity-generating power station at the dam toe, or to divert the river flow into a canal, tunnel or pipeline, forming an artificial lake for navigation, leisure, activities, etc.

The last century saw a rapid increase in large dam building. By 1949, about 5000 large dams had been constructed worldwide, three-quarters of them in industrialized countries. By the end of the 20th century, there were over 45,000 large dams in over 140 countries (fig. 3). Construction of large dams peaked in the 1970s in Europe and North America. Today most activity in these regions is focused on the management of existing dams, including rehabilitation, renovation and optimizing the operation of dams for multiple functions. An estimated 1700 large dams have been under construction in other parts of the world in the last few years. Of this total, 40% are reportedly being built in India (Word Commission on Dams, 2000).

This chapter describes the different dam typologies constituted by different materials (earthfill, rockfill and concrete) and the related types of failure in relation to the material characteristics and foundations (soil and rock) and loading conditions during the different stages of the dam life (from construction to operation phases).

1 Dam classification

Dams may either be human-built or result from natural phenomena, such as landslides or glacial deposition. The majority of dams are human structures generally constructed of earthfill/rockfill or concrete.

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Man-made dams are obstacles to naturally flowing water and are built for many purposes including:

- water storage for potable water supply, livestock water supply, irrigation, hydroelectricity, prevention of flooding, maintaining the water level in the highest reaches of canals, etc.;
- raising the water level to provide a head which can be turbined in an electricity-generating power station at the dam toe, or to divert the river flow into a canal, tunnel or pipeline;
- forming an artificial lake for navigation, leisure, activities, etc.;
- preventing water from the downstream side moving inland to protect farmland against seawater encroachment, maintain a reserve of fresh water in an estuary, create polders for land reclamation, contain mine tailings, etc.

Numerous criteria can be used to group dams into classes: intended purpose, features of the structure, dimensional parameters, hazard potential, etc.

A classification based on the intended purpose can include, for example, the following three main categories: storage dams (municipal water supply, recreation, hydroelectric power generation, etc.), diversion dams (supplying irrigation canals and transferring water to a storage reservoir for municipal or industrial use, etc.) and detention dams (to minimize the impact of flooding and restrict the flow rate of a particular channel). Large dams frequently serve more than one of these purposes and may combine features of the three main categories.

Traditional dam classifications are based on different building features such as type of material or the design solution for the dam structure. On the basis of these criteria, dams can be grouped into two main categories (International Commission on Large Dams, ICOLD), each of which is further subdivided:

- Embankment dams (fig. 1): They are built of earth and/or rockfill and resist the water pressure by their weight. If the material is not inherently watertight, they are faced with an impervious material or have a watertight core.
- Concrete dams (fig. 2):
  - Gravity dams: They have a roughly triangular cross-section and also resist water pressure by their weight. These are the most widespread type of concrete dams, accounting for two-thirds of the total.
  - Arch dams: They transmit most of the water load into the valley sides or large concrete thrust blocks.
  - Buttress dams: They have the water load transmitted to triangular buttresses parallel to the direction of river flow.
  - Multiple arch dams: They consist of a number of small arches bearing on buttresses.

Barrages are a special case, consisting of a line of large gates which, when closed, transmit the water load to flanking piers. Barrages are built on wide, slow-moving rivers.

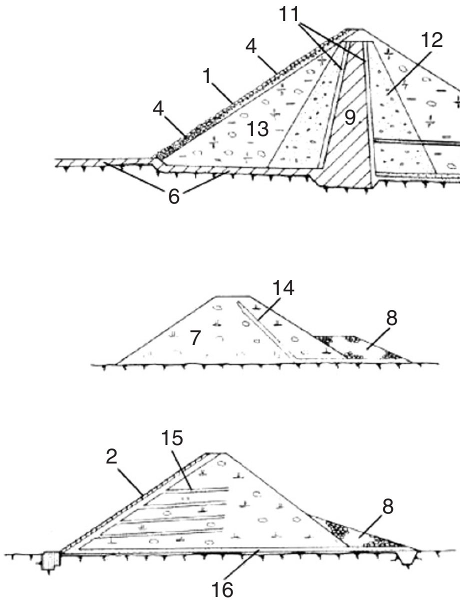


Figure 1: Types of embankment dams: (1) slope; (2) facing; (3) diaphragm; (4) slope protection; (5) berm; (6) upstream blanket; (7) shoulder or shell; (8) toe weight; (9) core; (10) core wall; (11) filter; (12) transition zone; (13) pervious zone; (14) chimney drain; (15) drainage layers; (16) blanket drain; (17) toe drain (ICOLD).

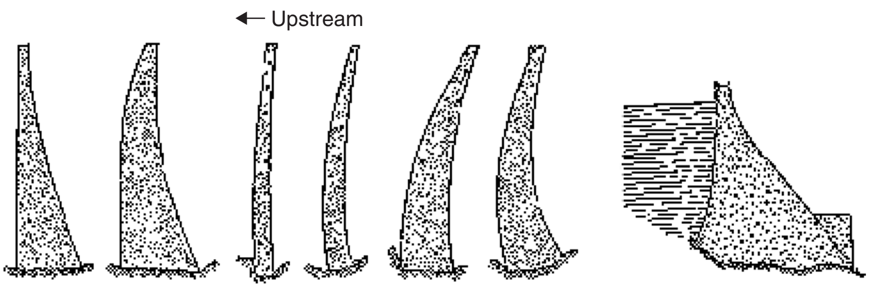


Figure 2: Concrete dams.

Dam design is determined by considering various factors associated with topography and geology of the dam site, the hydrological characteristics of the catchments, the regime of the river, the mechanical properties of the foundation soil and the available construction materials and the climate.

Jappelli [1] observed that today classification criteria based on building features are found to be insufficient or defective because of intermediate solutions that are now available. In fact, on one hand, earth can be reinforced with different materials and acquires qualities resembling concrete; on the other, concrete can be placed with methods used for earth embankment construction. One can also find composite dams, consisting partly of concrete and partly of earth or rock materials.

Dam classification can be based on the dimensional parameters of the construction (size of the dam), such as dam height, storage capacity, etc.

Several definitions are available of large dams, each serving a different purpose and objective, and, as such, are based on different criteria for evaluation.

The ICOLD defines a large dam as one with a height of 15 m or more from the lowest general foundation to the crest. However, even dams between 10 and 15 m in height could be classified as large dams if they satisfy at least any one of the following criteria:

- the crest length is not less than 500 m;
- the capacity of the reservoir formed by the dam is not less than one million cubic metres;
- its spillway is proportionate to a flood not less than 2000 m^3/s;
- the geotechnical conditions at the site are particularly difficult;
- the dam is of unusual design.

The last century saw a rapid increase in large dam building. By 1949, about 5000 large dams had been constructed worldwide, three-quarters of them in industrialized countries. By the end of the 20th century, there were over 45,000 large dams in over 140 countries (fig. 3). Construction of large dams peaked in the 1970s in Europe and North America. Today, most activity in these regions is focused on the management of existing dams, including rehabilitation, renovation and optimizing

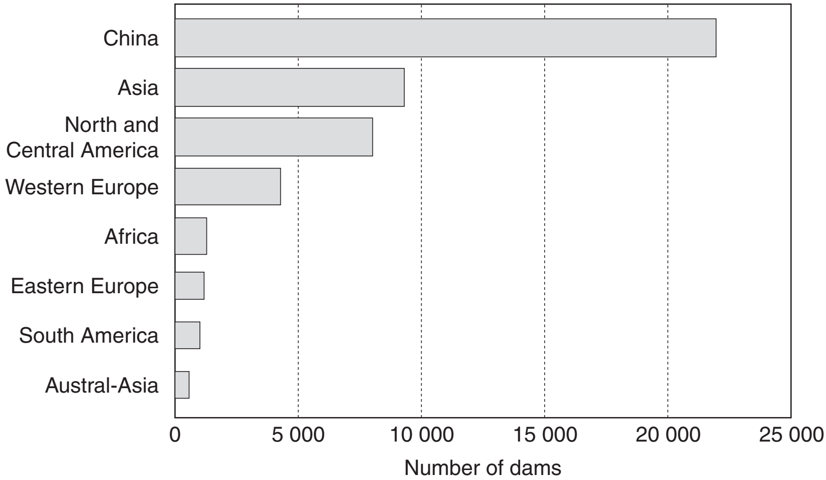


Figure 3: Regional distribution of large dams at the end of the 20th century (after [2]).

the operation of dams for multiple functions. An estimated 1700 large dams have been under construction in other parts of the world in the last few years. Of this total, 40% are reportedly being built in India [2].

Several countries adopt dam classification based on the downstream hazard. The majority of classifications do not consider the probability of failure of the dam, but the assessment of losses that dam collapse can cause downstream (Spain, Norway, Sweden, Finland, Portugal, United Kingdom, Netherland, Czech Republic, Slovakia Republic, China, New Zealand, Australia, Canada, United States, etc.).

The National Inventory of Dams, NID, which is maintained and published by the United States Army Corps of Engineers, is a database of approximately 79,000 dams in the United States (2005). The inclusion criteria are based on the hazard potential class of dams and on dam height and storage volume. A dam is included in the NID of the United States if:

1. it is a high or significant-hazard potential class dam or,
2. it is a low-hazard potential class dam that exceeds about 7.6 m (25 feet) in height and 18,500 m^3 (15 acre-feet) in storage or,
3. it is a low-hazard potential class dam that exceeds about 62,000 m^3 (50 acre-feet) in storage and is about 1.8 m (6-feet) height.

There are different definitions of hazard potential class in the policy of states and agencies (USA). For example, the definition, as accepted by the Interagency Committee on Dam Safety, of hazard potential class is as follows:

- Low hazard potential: Dams assigned the low-hazard potential classification are those where failure or misoperation results in no probable loss of human life and low economic and/or environmental losses. Losses are principally limited to the owner's property.
- Significant hazard potential: Dams assigned the significant hazard potential classification are those dams where failure or misoperation results in no probable loss of human life but can cause economic loss, environmental damage, disruption of lifeline facilities or other concerns. Significant hazard potential classification dams are often located in predominantly rural or agricultural areas but could be located in areas with population and significant infrastructures.
- High hazard potential: Dams assigned the high-hazard potential classification are those where failure or misoperation will probably cause loss of human life.

In the following text, dam failures are examined by different approaches typical of different engineering topics. Embankment dam stability analysis is examined by soil mechanics methods; rock foundations and abutments of concrete dams are studied by rock mechanics techniques and concrete dams are faced with structural mechanics approaches.

2 Embankment dams

Embankment dams are the oldest and most widespread type of dam, accounting for 83% of the world total. They presently represent the large majority of newly built dams. Today, embankment dams exist in excess of 300 m height with volumes of many millions of cubic metres of fill. Thousands of embankment dams exceeding 20 ms in height have been constructed throughout the world. Currently, China is the leader in embankment dam construction.

This large diffusion is due to three main reasons:

- Materials available within short haul distances are used.
- The embankment dam can accommodate a variety of foundation conditions.
- Often, the embankment dam is least costly when compared to other dam types.

Embankment dams are traditionally classified into two main categories by types of soil mainly used as construction materials, such as earthfill dams and rockfill dams.

Homogeneous earth dams have a dam section consisting almost entirely of one type of material (fig. 4(a)). In homogeneous earthfill dams, the dam body should exhibit resistance against both load transmitted by the reservoir and seepage forces with provided drainage facilities.

At most sites both pervious and impervious materials can be obtained. Under these circumstances, the dam is made up of a relatively impervious central zone called the core and outer zones that provide the structure with the required stability [3]. Such dams are called zoned earth dams.

The term rockfill dam refers to a dam in which the major portion of the pressure exerted by the impounded water is transmitted onto the foundation through a rockfill [3]. The fill material consists of fragments of sound rock, either obtained by blasting or available as natural boulder deposits.

At present, a great variety of construction materials is being used in combination with the rockfill and, consequently, the cross-sections of rockfill dams range between that of a rockfill dyke with the upstream slope covered with a membrane and that of a modern zoned earth dam. Rockfill dams can be classified into three categories based on the configurations of dam sections: rockfill dams with central cores, rockfill dams with inclined cores and membrane-faced rockfill dams, as shown in figs. 4(b), 4(c), and 4(d).

A filter zone should be provided in any type of rockfill dam to prevent loss of soil particles by erosion due to seepage flow through embankment. Rockfill dams with earth cores, which are basically similar to zoned earth dams, represent most of the highest embankment dams, because a good-quality rockfill provides free drainage and high shear strength [4]. The inclined core is adopted instead of the center core, for instance, in cases where the dam foundation has a steep inclination along the river, where a blanket zone is provided in the pervious foundation to be connected with the impervious core zone, and where different construction processes are available for the placement of core and rockfill materials. The membrane is commonly a reinforced concrete facing. Some dams are provided with composite reinforced

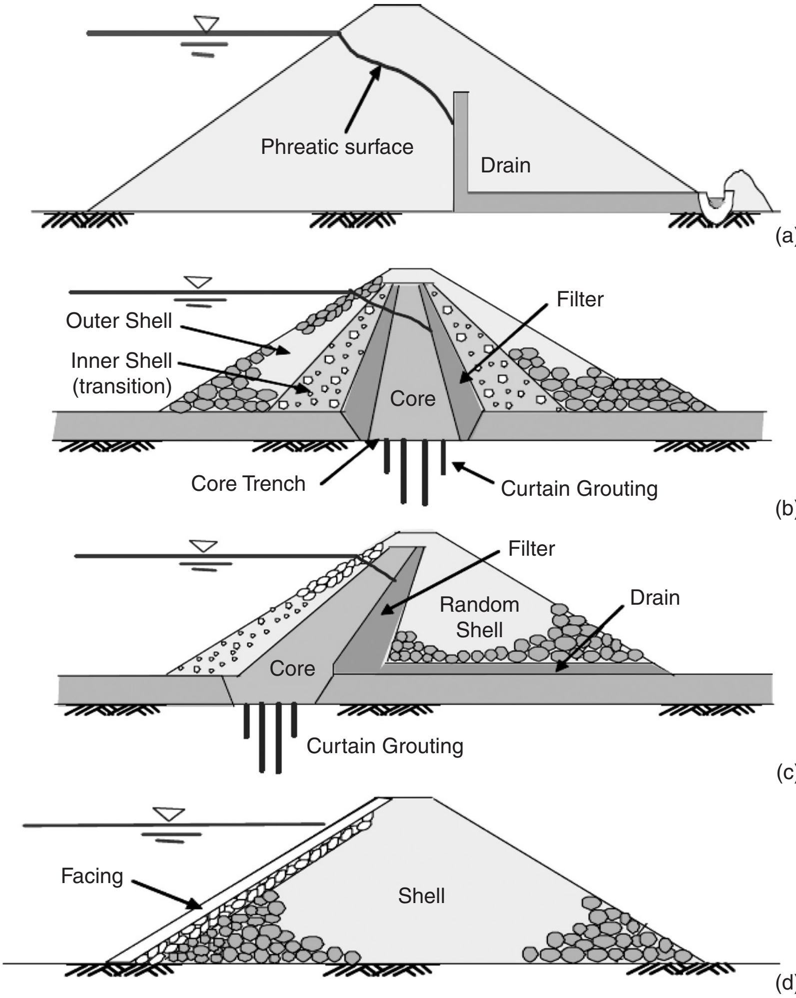


Figure 4: (a) Homogeneous earth dam; (b) rockfill dam with a centrally located core; (c) rockfill dam with an inclined core; (d) rockfill dam with a facing (after [5]).

concrete decks containing layers of asphaltic concrete or a drainage layer, or with facings made entirely of asphaltic concrete. This type of section is suited to rock foundations at sites where suitable earth core material is not available in adequate quantity or where continuously rainy weather makes soil placement difficult [4].

2.1 Loading conditions and related failures in embankment dams

Earth and rockfill dams must sustain very different loading conditions that arise during construction and subsequent operations. The acting loads include the self-weight, the varying reservoir water level, the uplift pressures in the foundation, the seepage flow pressures and eventually earthquake-driven loads. Total failures of dams have been reported during each of the stages of the dam life.

The behaviour of a dam can be inferred by the variation of displacements, total stresses, piezometric heads, leakages, etc. This variation depends on boundary mechanical and hydraulic conditions that develop during the construction and subsequent operations. The variation of these parameters is ruled by the self-weight, in the construction phase, and by water level and its oscillation in the reservoir, in the operation phases. Pore pressure changes are the response of the dam bulk to several processes, such as load increase, consolidation, and unsteady and steady seepage.

Load conditions during construction are induced by the progressive placement of compacted layers of material. The construction of an embankment dam is always associated with and followed by a differential settlement of its crest and slopes. Under unfavourable conditions they can be associated with the formation of open cracks across the impervious section of the dam. After the dam has been completed, the crest continues to settle at a decreasing rate. If the dam rests on sediments the settlements of the crest and slopes is increased by the compression of the foundation materials produced by the weight of the dam and of the impounded water at a later stage [3].

In this phase, the pore pressure distributions are principally controlled by static factors such as the load due to the self-weight and stiffness distributions. During construction, the impervious zone, which can eventually constitute the entire section of the dam, is placed at a specific moisture content for compaction to a specified density. The soil is partially saturated and pore pressure is negative. The development of positive pore pressures depends on the compressibility of the soil constituting the dam section, the rate of construction and the compaction moisture content.

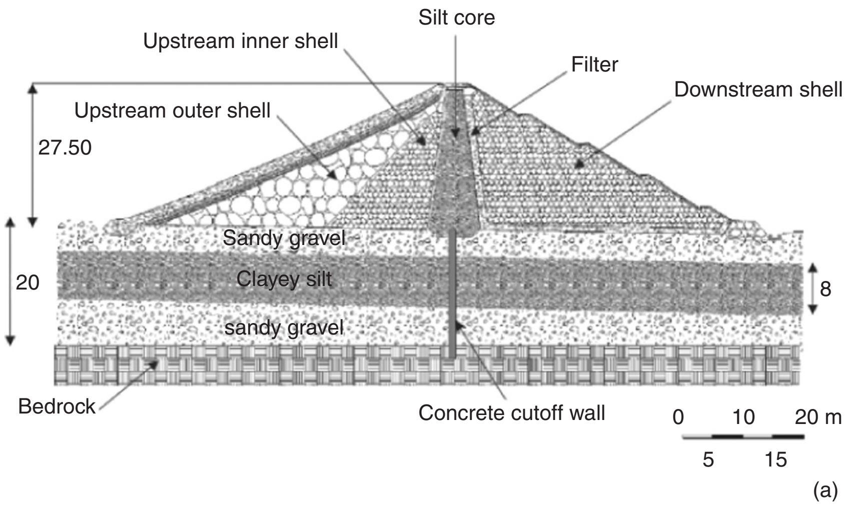
In a later stage of construction, the mean stress increment determines a reduction in void ratio: if the initial saturation is relatively high (80–90%) the soil may become saturated and the pore pressure is positive (fig. 5). In some cases, the high values of pore pressures can affect the stability of the upstream and downstream slopes.

Some dissipation of positive pore pressure can occur before the completion of the embankment, but is in the range between the end of construction and first filling that the pore pressure evolution is mainly controlled by consolidation processes (fig. 5).

During the initial filling of the reservoir, the embankment is subjected to the water loading, which implies different effects on the materials and boundaries. The seepage starts to develop in unsaturated materials.

In rockfill dams with earth cores or in zoned earth dams, the loading scheme affected mainly the upstream shell with the core loaded as if there were a thin impermeable membrane on its upstream face. The load of water also affects the upstream foundation.

The phenomenon of wetting induces reduction of strength and stiffness of the fill material, leading to saturation collapses.



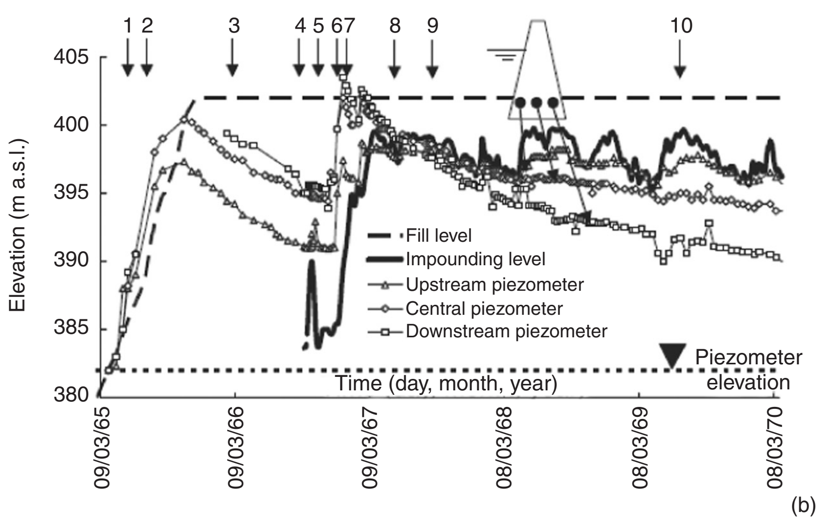


Figure 5: (a) Polverina dam (Italy) main cross-section. (b) Piezometric heads measured across the core of the Polverina dam during construction and the first stages of operation (after [6]).

As a result, the embankment experiences additional settlement, horizontal displacement and changes in stress field, demonstrated sometimes by cracking.

While the first filling loading scheme affects mainly the upstream shell the establishment of a steady seepage flow pattern within the core initiates another loading scheme [7]. In reality there is no clear distinction between them: the first merges into the second. Generally, the steady seepage stage is characterized by the development of pore pressure within the part of the core which is below the

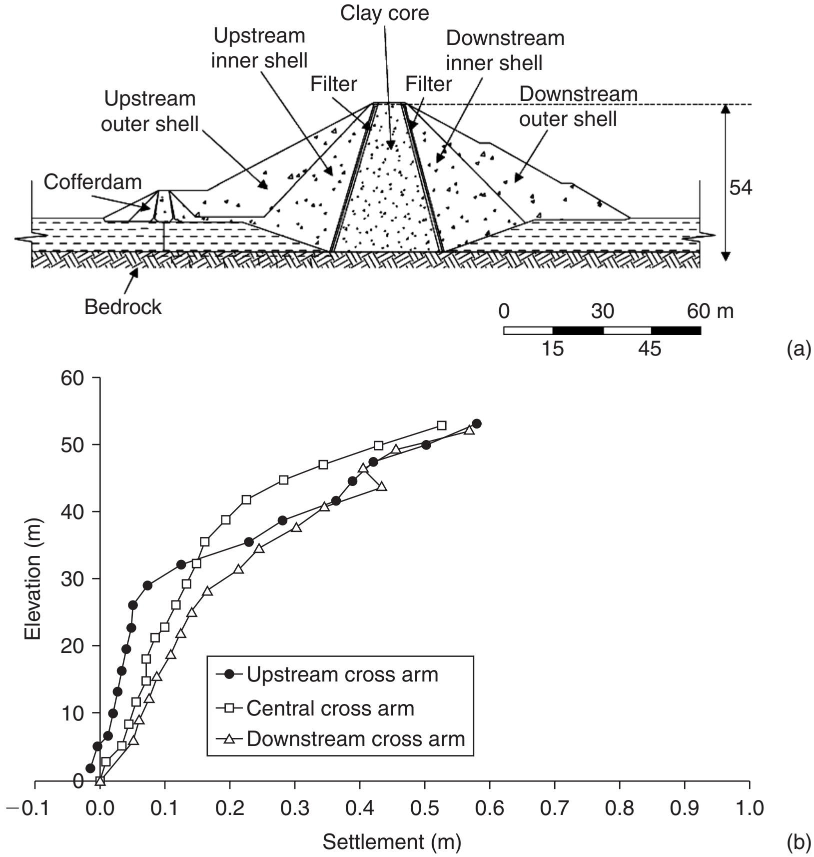


Figure 6: (a) Main cross-section of Beliche dam (Portugal). (b) Settlement profiles measured in the core of the Beliche dam during the first 5 years of operation, not including settlements developed during construction (after [6]).

phreatic line. The distribution of pore pressures is governed by a corresponding flow net. A change of pore pressure caused by seepage results in a change of effective stress thus causing additional deformations (fig. 6). Some movements during the steady seepage period can be attributed to the viscous behaviour of the soil or to a process of consolidation. It is however difficult to separate clearly first filling and steady seepage movements, as there is no clear division between the two stages because seepage begins as the reservoir fills.

At this stage, the knowledge of the stress distribution within the core is important, because under particular conditions of the state of stress the formation of fractures or the propagation of pre-existing fractures can occur. Cracks may evolve in zones where the minor total principal stress at any point within the soil is equal to the pore water pressure at the same level. In zoned embankment dams,

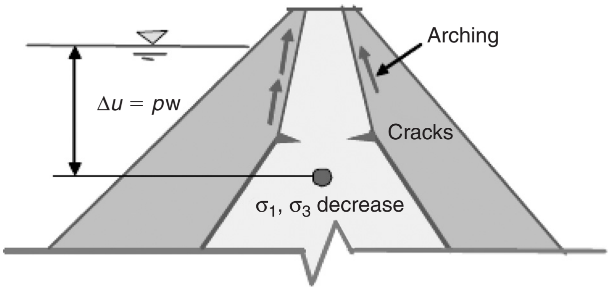


Figure 7: Arch effect in zoned embankment dams (after [5]).

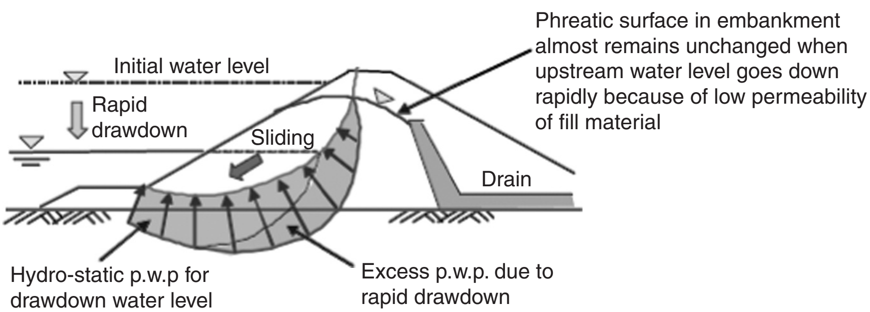


Figure 8: Excess pore water pressure due to rapid drawdown (after [5]).

the contrast of stiffness of the different materials may induce an inhomogeneous distribution of vertical stress. This phenomenon is called arch effect and implies a reduction of the state of stress in the core and an increase in the shells (fig. 7).

After steady seepage has been established, if the reservoir level is lowered suddenly or so rapidly that the drainage of the upstream portion of the dam, as well as of the natural slopes adjacent to a reservoir, cannot occur, the condition of drawdown results. In this condition, the soil inside the embankment remains saturated, while the stabilizing effect of the water on the upstream face is no longer available (fig. 8). Seepage takes place from the saturated embankment towards both slopes. For the downstream slope, this condition is less adverse than the steady seepage condition. The drawdown condition is critical for the upstream slope.

Earthquakes result in an additional loading on the dam embankment materials. The acceleration produced by earthquakes alters the distribution of forces in slopes in a manner equivalent to a temporary steepening of the slope. Shear stress can exceed temporarily the shear strength of the soils, leading to a local yielding. Rapid repeated stress fluctuation (cyclic loading) can induce changes in pore fluid pressure distributions that reduce soil strength or can induce a decay of the mechanical properties of some type of soils. These actions can lead to sliding of the dam slopes or to liquefaction of the upstream slope under particular value of state parameters of the soil. Furthermore, earthquakes can generate positive excess pore pressure in loose granular foundation soils that can experience liquefaction. The most common effects caused by earthquakes are settlements and cracking.

2.2 Mechanisms of failure

ICOLD [7] defines failure as a collapse or movement of part of a dam or its foundation, so that the dam cannot retain water. In general, a failure results in the release of large quantities of water, imposing risks on the people or property downstream.

The statistics of dam failures performed by ICOLD [8] takes into account dams 15 m high or more or with a storage volume of at least 1 million of cubic metres. The results of this work can be summarized as follows (data from China and the URSS were not considered): the most common type of failure in earth and rockfill dams is overtopping (31% as a primary cause, 18% as a secondary cause), followed by internal erosion of the body (15% as a primary cause, 13% as a secondary cause), and in the foundation (12% as a primary cause, 5% as a secondary cause).

Foster *et al.* [9] used ICOLD data and information on the dams which had experienced incidents (either a failure or an accident requiring major repair) to develop historic performance data for dams constructed up to 1986. In this context, they summarized the failure statistics of embankment dams: 35.9% of all dam failures can be attributed to overtopping, 5.5% to slides, 1.6% to earthquake liquefaction and 46.1% to some form of piping.

Failures leading to catastrophic consequences occur typically at first filling and full reservoir stage, although overtopping during construction has caused, in some cases (for example Panshet dam in India, Sempor dam in Indonesia and S. Tomas dam in Philippines), several victims.

During construction, shear stress on potential failure surfaces increases. Pore pressure also increases, since soil already in place is loaded as subsequent lifts are placed. A state of limit equilibrium can occur if the shear strength along a plane is reduced by pore pressure increase equaling the shear stress required for equilibrium, resulting in sliding failures for both the upstream and downstream slopes (fig. 9).

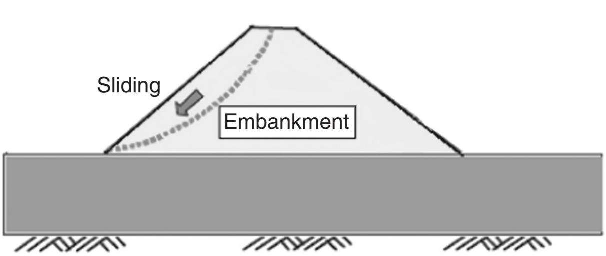


Figure 9: Sliding of dam slope at the end of construction (modified from [5]).

Filling of the reservoir causes the shear stresses within the upstream slope to decrease because of the favourable effect of water pressure against the slope, while the average shear stresses under the downstream slope remain unchanged or increase slightly [10].

Water stored behind a dam always seeks to escape or flow along the path of least resistance [11]. This route may be through the dam, beneath it or around it. Seepage can lead to dam failure by one or a combination of the following mechanisms:

1. Subsurface erosion
2. Heave (or blow-out)
3. Concentrate leak/hydraulic fracturing
4. Suffusion
5. Slope failure (including sloughing).

The reported statistics showed that the most frequent mode of failure is piping.

In engineering practice, the term piping is used to indicate different types of failure related to the seepage of water. The process of piping is always accompanied by a phenomenon of internal erosion, which is induced by water seepage.

Piping by subsurface erosion, as described by Terzaghi and Peck [3], occurs when water removes soil particles and transports them towards unprotected exit, developing unseen channels or pipes through a dam or its foundation. Failure occurs as soon as the upstream or intake end of the eroded hole approaches the bottom of the reservoir. The basic condition for this type of failure to occur is that the material overlying the eroded soil must possess at least a trace of cohesion, sufficient to form a roof over the erosion tunnel.

Heave is a typical subject in soil mechanics problems. The seepage pressure is produced by the friction between the percolating water and the walls of the voids and can be described as a “drag” [3]. If the water flows in an upward direction, the friction between the water and the walls tends to lift the sand grains. If the hydraulic gradient i is steadily increased, but is less than the critical hydraulic gradient i_c the discharge increases in accordance with the Darcy’s law, in direct proportion to i, and the value of the coefficient of permeability remains constant. This fact indicates that mutual position of sand grains remains practically unaltered. However when i becomes equal to i_c, the discharge increases suddenly, involving a corresponding increase of the coefficient of permeability. This process is accompanied by a violent and a visible agitation of the sand particle, which is commonly referred to as a boiling sand (fig. 10). Sand starts to boil at a hydraulic gradient greater than the critical value i_c. The extension of this local phenomenon is called heave and involves a prism of soil subjected to a total excess pore pressure on the base of the prism that equals the effective weight of the soil (fig. 11). The progression of this mechanism is called piping by heave.

This kind of failure occurrence in embankment dams can be prevented by constructing a weighting berm or a pressure relief well (loaded filter above the area in which the seepage emerges from the ground).

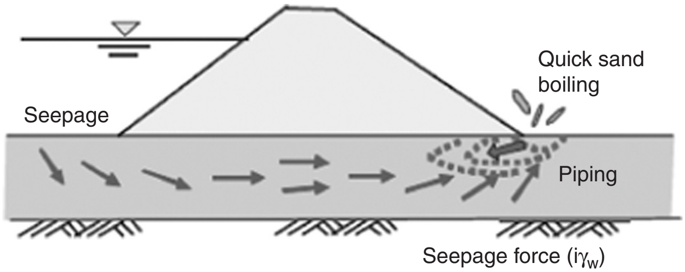


Figure 10: Boiling sand (after [5]).

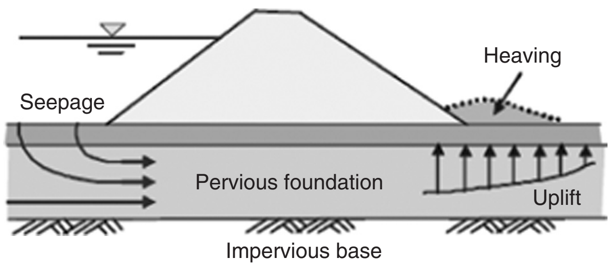


Figure 11: Heave mechanism (after [5]).

Embankment dams are subjected to deformations such as compression, extension, and shear distortion. Normally, embankment dams are constructed in zones of different materials that exhibit different compressibilities leading to internal redistribution of stresses.

In a central core earth and rockfill dam, internal stress redistribution might take place in both directions, i.e., by transverse arching of the core between the upstream and downstream shells (fig. 12) and longitudinally between dam sections of different height (fig. 13). A critical zone for potential leaks is over the abutments, where longitudinal stretching of the dam and hydraulic fracturing are most probable.

Deformations in a dam might generate cracks or "soft zones" where internal erosion can be initiated. Fill materials have in general low tensile strength, so cracking can result from even very small tensile stresses.

There are often various types of uneven settlements, which is the main cause for cracking. Differential settlement cracks can expand and develop further by the hydraulic fracturing process. If differential settlements have led to "soft zones" rather than cracks, hydraulic fracturing might later on form cracks in such "soft zones" [12].

Mattsson *et al.* [12] describe hydraulic fracturing as the process when water pressure splits a soil with a low permeability, which might happen if the minor total principal stress at any point within the soil becomes equal to the pore water

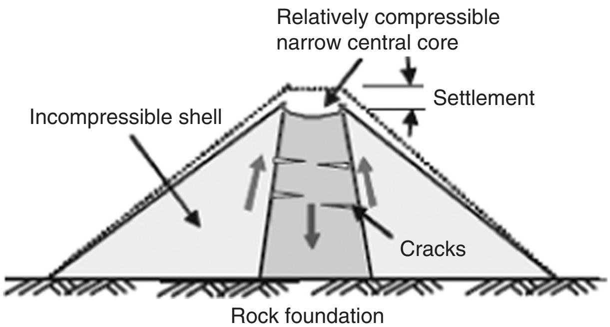


Figure 12: Transverse arching (after [5]).

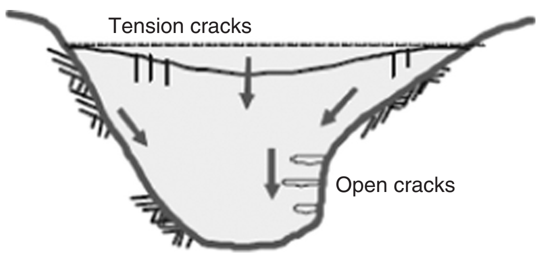


Figure 13: Longitudinal cracks (after [5]).

pressure at the same level (hence the minor effective principal stress reduces to zero). Fracturing may proceed continuously following roughly the plane of the minor principal stress. For hydraulic fracturing to take place, the adjacent material must be sufficiently impervious so that water can wedge its way along opening cracks more rapidly than it permeates the pores. Therefore, hydraulic fracturing occurs only in rather impervious materials, such as a dam core of clay, and not in coarse pervious materials such as gravel and stone [13].

Suffusion is the process where the fine particles of the soil wash out or erode through the voids formed by the coarser particles. This can be prevented if the soil has a well-graded particle size distribution with sufficiently small voids. Soils are called internally instable if suffusion takes place and internally stable if particles are not eroding under seepage flow [12].

Piping can occur in the embankment, through the foundation and from the embankment into the foundation (figs. 14, 15, 16 and 17) as a progression of internal erosion caused by one of the seepage-induced mechanisms described above.

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 [9].

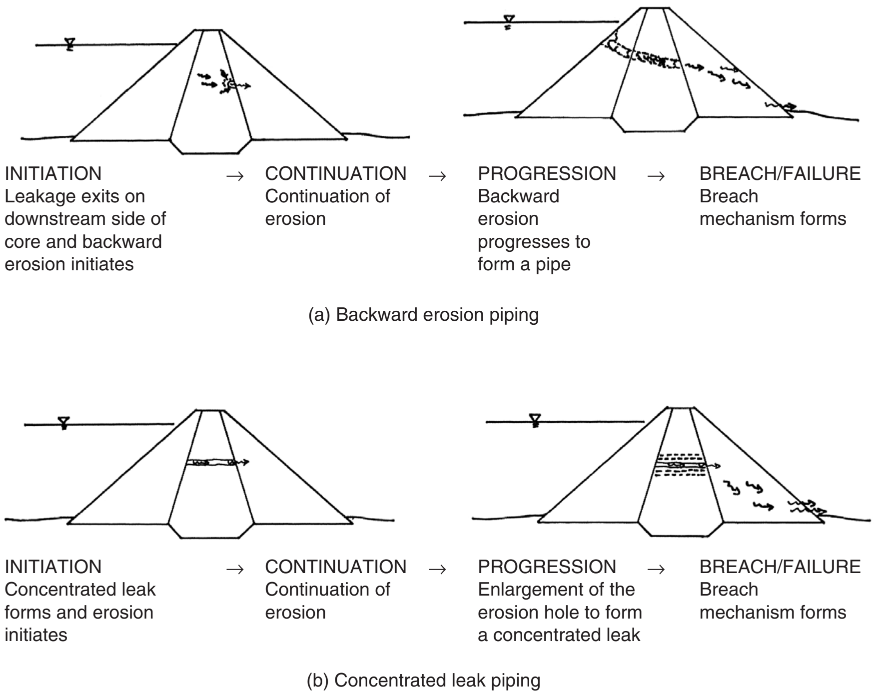


Figure 14: Conceptual model for development of failure by piping in the embankment: (a) backward erosion and (b) concentrated leak (after [15]).

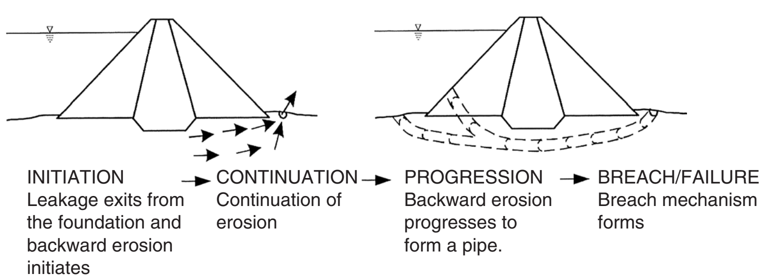


Figure 15: Conceptual model for development of failure by piping in the foundation (after [15]).

Further, it was noticed that about half of all piping failures through the embankment are associated with the presence of conduits (fig. 18). The different modes of piping associated with conduits are piping into the conduit, along and above the conduit or out of the conduit [14].

Slope failure induced by seepage forces requires the evaluation of pore pressure distribution in the dam and foundation. In fact when considering internal

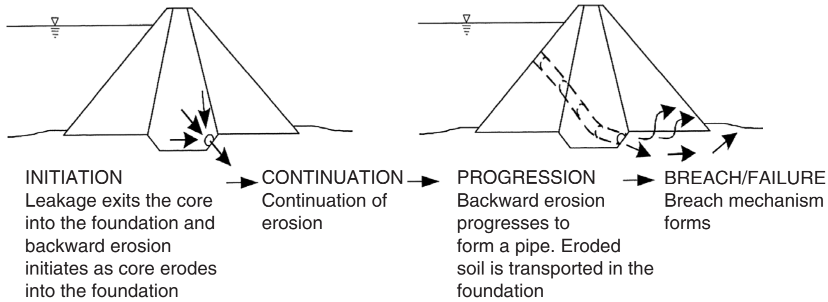


Figure 16: Conceptual model for development of failure by piping from the embankment into the foundation (after [15]).

Figure 17: Failure by piping, Tunbridge dam, Tasmania, Australia (available from Geoengineer.org website).

erosion and piping and slope instability the confidence with which seepage pore pressure can be assessed is critical [14].

Figure 19 shows the seepage flow net in the case of an earth fill with a horizontal drain in the case of isotropic hydraulic behaviour of the soil (horizontal-to-vertical permeability ratio is 1). Seepage pore pressures are controlled by the permeability of the soil in the dam and foundation. If the horizontal drain is a filter adequately designed, internal erosion and piping is controlled. Pore pressures in the downstream slope should be low, with a resulting high safety factor against sliding.

Fell *et al.* [14] observed that the permeability can be affected by defects generated during construction of the earth fill layers, resulting in zones of poor compaction within

Figure 18: Failure by piping along conduits (after [16]).

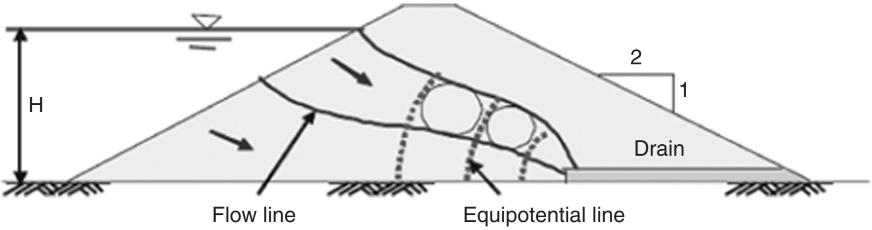


Figure 19: Seepage through an isotropic embankment (after [5]).

the single layer and zones cracked by desiccation. The permeability can be also affected by cracking in the dam induced by differential settlement or earthquake. These effects influence the horizontal-to-vertical permeability ratio which can lead to high pore pressure in the dam. Figure 20 shows the seepage flow net in the case of horizontal-to-vertical permeability ratio greater than 1. The degree of stability of the downstream slope is lowered by higher pore pressures.

If seepage line emerges on the downstream face, the dam toe softens due to saturation below the point of emergence. In addition, soil particles are subjected

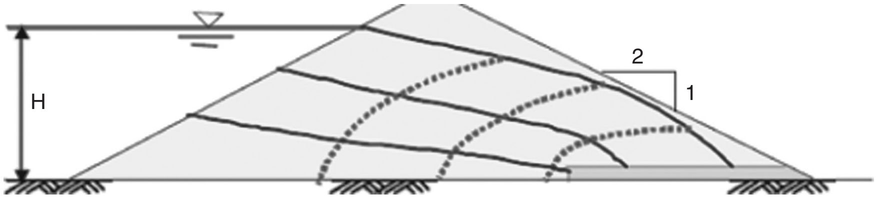


Figure 20: Seepage through anisotropic embankment (after [5]).

to a drag force in the direction of flow. The horizontal component of this force will tend to dislodge the soil particles if the resistance of the soil is exceeded. Break-up of the downstream slope induced by the dislodgement of particles under excessive gradient is called sloughing. Washing out of the soil is a progressive action and leads to a progressive break-up of the downstream slope through a series of "micro-slides" until the remaining section is no longer stable under the applied reservoir pressure and fails by sliding [4].

The control of pore pressures and hence internal erosion and piping can be performed by providing filters/drains for the full height of the dam, such as with an earthfill with horizontal and vertical or inclined downstream drains or a central core earth and rockfill dam [14].

According to the International Committee on Large Dams [8], and the work of Foster *et al.* [9] one-third or more of the total identified failures was caused by dam overtopping.

Overtopping of a dam (fig. 21) 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.

When overtopping of the crest occurs, water flows along the dam downstream face itself, endangering its own integrity by both erosion and creation of uplift pressures in the embankment. If persistent in time, it can lead to breaching of the embankment [17].

Generally embankments are not designed to resist the erosive action of water flow over the crest and overtopping failure cannot be considered a defect of embankment dams. de Almeida Manso [17] presented the results of a study regarding the stability of concrete macro-roughness linings for surface protection of overflow earthfill dams, and summarized the processes induced by overtopping of a dam as reported in the following.

Once water starts flowing over the crest two kinds of flow develop: an infiltrating flow through the dam's body and a free-surface flow along the dam's face. Depending on the embankment constitution, on the duration of overflow and on the magnitude of the overtopping event, the relation between these two types of

Figure 21: Dam overtopping (available from Association of State Dam Safety Officials website).

flows changes considerably resulting in very different failure evolutions. Magnitude is defined as the head over the crest or the head rise rate.

During normal operation of the reservoir a water saturation line will be established across the embankment. A drainage prism in the downstream toe of the dam, preventing the saturation line to reach the downstream face surface, collects the seepage flow. The establishment of a steady-state seepage flow corresponding to a given reservoir level is a long-term process when compared with the short duration of most overtopping events, that are instead in the order of hours.

Depending on the magnitude and on the duration of overtopping, different situations can occur (fig. 22):

1. For low flows the entire overflow might be infiltrated. If the event is of short duration there will be hardly any rise in the saturation line.
2. When the overflow increases more rapidly than the infiltrating flow, water will run along the dam slope. Free-surface flow will progressively erode the slope and the dam toe.
3. For an event of long duration, infiltration might occur during time enough to raise the saturation line, leading to internal erosion and shallow surface sliding. Water will spring out along the downstream face of the dam, *independently* of the magnitude of free-surface flow. The destabilizing uplift pressures generated in the embankment by this seepage flow will unbalance the equilibrium of some parts of the embankments that will subsequently slide.

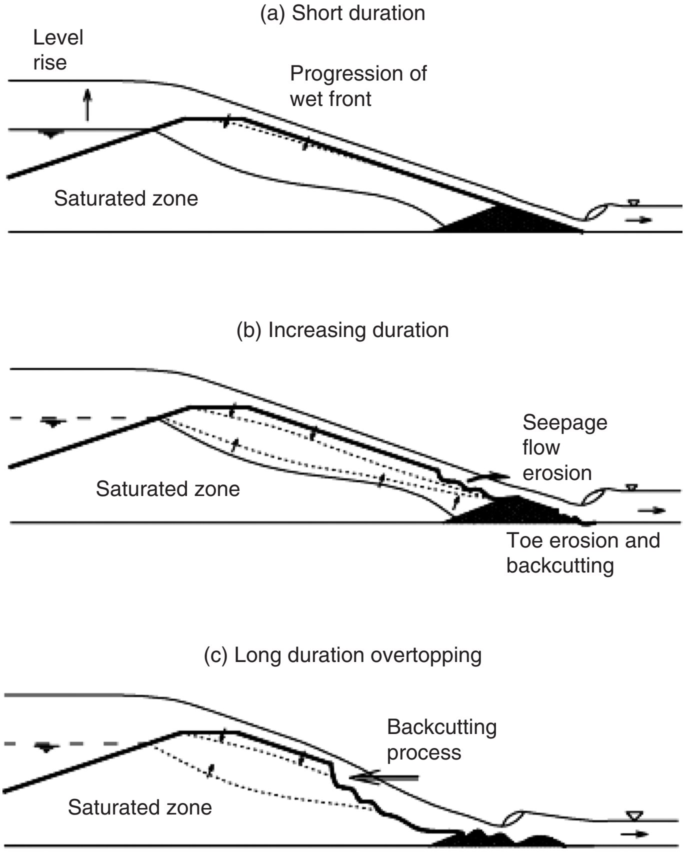


Figure 22: Schematic representation of the failure mode of an homogeneous earth fill dam subject to overtopping (with hydraulic jump at the toe) (after [17]).

Often, the critical zone in terms of stability is the downstream toe, where free-surface flow reaches its maximum velocity and thus its maximum erosion potential. Further, the seepage flows resulting from the intersection of the wet front with the elevated saturation line will also emerge in this region.

Starting at the toe, erosion will progress upstream, by deepening the free-surface flow channel, also generating bank slides; it will eventually end up completely breaching the embankment. This process is often called back-cutting or retrogressive erosion and is accelerated if there is a hydraulic jump over the toe.

Although few cases of total failures of embankment dams induced by earthquakes have been reported, quite a number of cases in which significant damage occurred are known [4]. Moderate-to-severe damage mainly in the form of cracking, sliding of the downstream and upstream slopes, internal erosion and piping

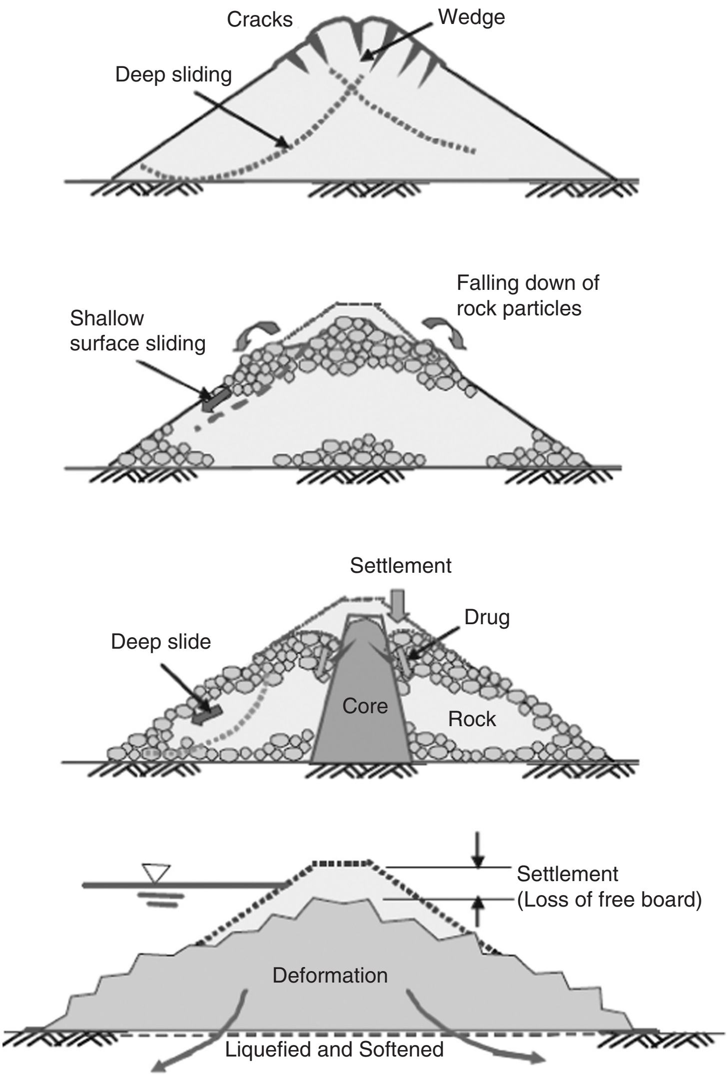


Figure 23: Earthquake-induced failures in embankment dams (after [5]).

which may develop in cracks took place in many cases (fig. 23). Total failure is most attributable to liquefaction in embankment and its foundation (fig. 23). Failure resulting from overtopping may be frequently caused by:

- loss of freeboard due to settlement of the dam and foundation;
- seiches or oscillation of the reservoir water level and
- waves caused by failure of natural slopes adjacent to the reservoir.

During the past decades, the advancements in theoretical soil and rock mechanics, soil testing techniques and seepage assessments have allowed to face with adequate safety margins the design of embankment dams. However, presently the occurrence of dam failures is not always the result of extreme loading conditions. Rather, dams fail due to unforeseen or unrecognized conditions that result in piping failures, overtopping at less than the design inflow, foundation failures, errors in operation or other “non-standard” failure modes (Bulletin of the workshop on “Learning from International Dam Incidents and Failures” March 24–25, 2009 – Los Angeles).

3 Concrete dams

From the structural point of view, concrete dams can be divided into the following construction typologies [18]:

- *Gravity dams*, which include the so-called *hollow gravity dams* (e.g. *buttress dams*, *solid-head buttress dams*, etc.)
- *Arch gravity dams* or *thick-arch dams*
- *Thin-arch dams*, made with *single-curvature arches* (simple arch) or with *double-curvature arches* (dome-shaped)
- *Buttress dams with multiple arches*
- *Buttress dams with slabs*
- *Composite dams* (i.e. made with two of the previous typologies, such as *gravity* and *multiple-arch dams*).

The structural safety of dams has to be measured through both global and local analyses. As for a classical retaining wall, the following ultimate limit states of external instability should be considered for the entire structure:

- *Loss of overall stability* is likely to occur in an area which in itself is of marginal stability (e.g. a steeply sloping site or a slope with a high groundwater level) or where a weak subsoil (e.g. a very soft clay layer) is present underneath the dam. Limit equilibrium methods may be used to check the overall stability.
- *Sliding failure* involves outward translation of the retaining wall due to shearing failure along its base or along a soil surface below the base.
- *Overturning failure* involves rotation of the wall about its toe.
- *Bearing capacity failure* occurs when the ground bearing pressure (i.e. the intensity of loading imposed on the ground by the wall) exceeds the load-carrying capacity of the foundation.

The calculation models used for checking against these types of failures assume that the dam acts as a monolith at limit state.

From a local point of view, the stem of a concrete dam should be designed to resist the effects due to pressures acting on the dam. Therefore, it is necessary to define, for a given set of loads, stresses and strains in each cross-section of the dam body.

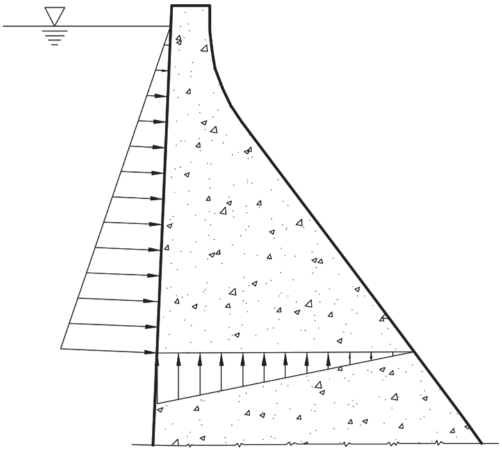


Figure 24: The structural behaviour of gravity dams.

Analytical approaches and numerical models are generally used to evaluate both states. They are developed for a single construction typology or can be applied to each dam body, such as in the case of Finite Element Models (FEM).

As a first approximation, the strength of *gravity dams* (including *hollow gravity dams* with and without *buttress*) is that of the vertical cross-sections (fig. 24), similarly to a gravity-retaining wall [19].

Conversely, the bearing capacity *of arch gravity dams* is evaluated by reducing the dam to the arches defined by the horizontal cross-sections (fig. 25(a)). However, in this case the structural contribution is usually enriched by considering the multi-axial state of stress acting in the dams. As depicted in fig. 25(b), the dam body can be considered, both geometrically and statically, as a portion of an axial symmetric structure, for which, under particular loads, the complete state of stress (and strain) is analytically defined [20].

Modelling concrete dams by means of more complex static schemes is a necessity not only for arch gravity dams, but mainly for *double-curvature arches*. In these cases, the horizontal cross-sections are always the arches of fig. 25(a), whereas the vertical cross-sections are in the middle between a slender cantilever beam and the corbel of fig. 24. In many cases, due to complex shapes, body dams are modelled with three-dimensional finite element analysis (3D-FEM), in which the whole domain Ω is subdivided into three-dimensional finite elements Ω_k (fig. 26).

Without going into details, the basic steps of a 3D-FEM using the displacement method are the following [21]:

1. Idealize the structure as an assemblage of 3D elements that are interconnected at structural nodes.
2. Identify the unknown node displacements that completely define the displacement response of the structural idealization.

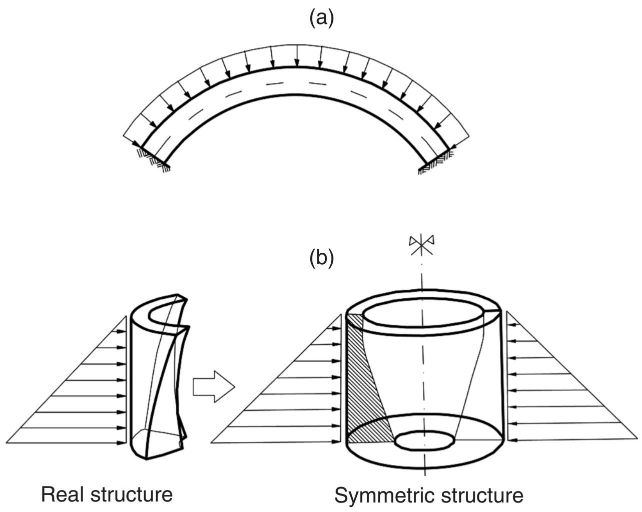


Figure 25: The structural behaviour of arch gravity dams.

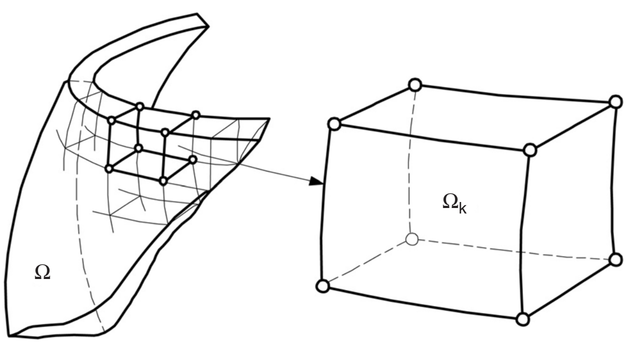


Figure 26: 3D finite element model for concrete dams.

3. Formulate force balance equations corresponding to the unknown node displacements and solve these equations.
4. With the finite element node displacements known, calculate the internal element stress distributions.
5. Interpret, based on the assumption used, the displacements and stresses predicted by the solution of the structural idealization.

In practical analysis and design, the most important steps of the complete analysis are the proper idealization of the problem (i.e., step 1) and the correct interpretation of the results, as in step 5. Depending on the complexity of the analysed structural system, considerable knowledge may be required in order to establish an appropriate idealization. More specifically in the case of concrete dams, 3D-FEM simulation of failure scenarios entails the use of nonlinear stress strain relationships for massive or unreinforced concrete. For these reasons, validation of the numerical prediction is usually made by means of reduced-scale experimental tests, conducted on mock-up dams up to the failure [22].

3.1 Concrete as dam material

In the developed countries, concrete is the most used material in gravity and arch gravity dams. It is generally unreinforced, except for the presence of an orthogonal reinforcement mesh near the surface. When used in massive structures like dams, concrete does not possess high strength. Conversely, a good homogeneity of mass concrete is strongly required.

At the beginning, dams consisted of vertical monolith blocks cast separately by using low-cohesion concrete (with the cohesion of humid soil), and compacted with immersed vibratory equipments. This was a slow construction procedure because a lot of time was needed for placing, compacting and consolidating the mass concrete. Improvements came from the construction of earth and rock-filled dams. In particular, horizontal continuous layers (of about 70 cm) were cast and extended to all the dams. In addition, the use of earth-moving equipments made the construction of concrete mass dams faster and therefore more cost-effective.

At the end of 1970s, the traditional vibratory equipments were replaced by roller compaction, and the first successful applications of this new roller compacted concrete (RCC) were demonstrated.

According to Mehta and Monteiro [23], the following advantages can be achieved by using RCC:

– Costs: depending on the complexity of the structure, RCC costs 25–50% less than conventional concrete.
– Rapid construction: for large projects, RCC dams can be finished 1 to 2 years earlier, compared to regular mass concrete dams.
– Spillways: compared to embankment dams, which normally require that spillways are constructed in abutment, RCC dams offer the attractive and cost-effective alternative of constructing the spillway in the main structure of the dam.
– Diversion and cofferdam: the cost of river diversion and damages caused by cofferdam overtopping are smaller for RCC dams than for embankment dams.

With respect to conventional concrete dam construction, additional advantages come from the worldwide experiences of using RCC:

– Cement consumption is lower because of the layer placement method.
– Pipe cooling is unnecessary because of the low temperature rise.
– Cost of transporting, placement and compaction of concrete is lower because concrete can be hauled by dump trucks, spread by bulldozers (fig. 27) and compacted by vibratory rollers (fig. 28).

Figure 27: Transportation and placement of RCC.

Figure 28: Compaction of RCC.

– Rates of equipment and labor utilization are high because of the higher speeds of concrete placement.

3.1.1 RCC

According to ACI 207.5R [24], roller compact concrete is a concrete compacted by roller compaction. It differs for conventional concrete principally in its consistency requirements. For an effective consolidation, the RCC mixture must be dry enough (whereas traditional concretes have plastic or humid consistencies) to support a roller while being compacted. However, it should be wet enough to permit adequate distribution of the binder mortar in concrete during mixing and vibratory compaction operation.

The consolidation by a roller does not require special cements. However, when RCC is used in massive concrete dams, the use of cements with lower heat generation is strongly required. Low cement contents (lower than 100 kg/m^3) are generally adopted. Sometimes, depending on the impermeability requirements, cement contents can be extremely low (30–40 kg/m^3). In such situations, mineral admixtures are extensively used in RCC mixtures. For instance, the use of large amounts (170 kg/m^3) of fly ash, slag and natural pozzolan reduces the adiabatic temperature rise of concrete and improves its durability.

The size of coarse aggregate has a significant influence on the degree of compaction in small layers. This influence is less marked in relatively thicker layers especially when large vibratory rollers are employed. In both cases, aggregates greater than 76 mm in diameter are seldom used in RCC because they can cause problems in spreading and compacting the layer. Usually the aggregate size is limited to 38 mm. Aggregate gradation is also a key aspect. In fact, the use of 8–10% in weight of material finer than 75 μm produces a more cohesive mixture by reducing the volume of voids.

Although ACI 207.5R [24] identifies five methods of RCC mixture proportioning, only two approaches are used for proportioning the components of RCC [23]. In the first, based on the principles of soil compaction, the optimum water content is chosen in order to obtain the maximum dry density of the mixture (whereas in ordinary concrete the higher concrete strengths are obtained by reducing the water–cement ratio). In this case, for RCC the best compaction gives the best strength. The second approach uses traditional concrete technology methods to produce RCC mixtures.

For RCC mixtures made according to the concrete technology approach, where the volume of the paste exceeds the volume of voids between the aggregates, the compressive strength depends on the water–cement ratio as predicted by Abram's rule. Conversely, for the RCC mixtures made according to the soil mechanics approach, where the cement paste may not fill the voids between aggregates, Abram's rule does not apply and strength is a function of the moisture content. Generally, the compressive strength of concrete varies from 8–10 MPa (with low cement content) to 20–40 MPa in the presence of mineral admixtures.

As for normal concrete, the ratio between tensile and compressive strength ranges from 7 to 13%. RCC containing very low cement content or marginal aggregate will have much lower tensile strength.

The shear stress developed between the layers of the RCC is a critical parameter in designing concrete dams because the permeability of the structure strongly depends on it. According to Coulomb's law, shear stresses are related to normal stresses, to cohesion (which varies from 0.5 to 4 MPa [23]) and to the angle of internal friction (whose tangent varies from 0.25 to 0.5 [23]).

The coefficient of permeability of RCC is a critical parameter for long-term performance of dams. Depending on the mixture proportioning and construction project, the coefficient of permeability ranges from 10^{-4} to 10^{-13} m/s. The upper bound limit of such range is that of low cement content, and dams made of this type of concrete require an impermeable membrane at the upstream. Conversely, the lower bound limit, obtained in RCC with high cement content and/or mineral admixtures, corresponds to that of traditional concrete.

As for mass concrete, the thermal stresses generated by heat of hydration are proportional to the elastic modulus of concrete. Therefore, RCC mixtures, which produce concrete with low elastic modulus, are very attractive to designers. As with regular concrete, the elastic modulus of RCC depends on the degree of hydration, volume and type of aggregate, and water–cement ratio. Poisson's ratio for RCC typically ranges from 0.15 to 0.20.

The long-term deformation of RCC depends on the amount and the type of aggregate, the water–cement ratio, the age of loading and the duration of loading. RCC with lower compressive strength and lower elastic modulus will normally show high creep, which is a critical factor in determining the stress relaxation when thermal strain is restrained. Lean concrete with large amounts of fines also shows high creep rates.

The adiabatic temperature rise of RCC is similar to conventional mass concrete mixtures and depends on the amount and type of cementitious material used in the mixture. The specific heat, conductivity and coefficient of thermal expansion are a function of the type and amount of aggregates used in the mixture.

The construction process of RCC generates porous zones between the lifts where water can percolate. It is important to point out that, besides optimizing the mixture proportions, it is equally crucial to provide proper compaction throughout the structure. If the moisture content in concrete goes beyond the critical saturation point, the performance of non-air entrained RCC to cycles of freezing and thawing will be poor. However, if the structure does not become saturated, the frost resistance of RCC is satisfactory. Air entrainment of RCC mixtures has not been very successful. Finally, the erosion resistance of concrete is a function of the strength of the concrete and the quality of the aggregate.

3.2 Structural damage of concrete dams

Independently of the relatively low risk of global collapse, structural damage of concrete dams causes enormous retrofit costs and infers durability dramatically. Most of the problems begin at the construction stage, due to relevant heat hydration of the cement, shrinkage and creep effects, and alkali aggregate reaction.

In this chapter, the various damage phenomena which can affect concrete dams are described. Chemical and physical attacks, as well as mechanical damage and onset of fractures, are considered.

3.2.1 Ageing of concrete

Ageing of dams may be characterized by:

- Transport mechanism
- Physical processes
- Chemical processes
- Mechanical phenomena.

They affect, with different emphasis, the concrete surface parts, as a consequence of chemical reactions between adverse substances penetrating the concrete and the various phases of the hydrated cement paste and, in some instances, of the aggregates. Such reactions may lead to damage either due to a dissolution of concrete compounds (leaching) or due to a volume increase (expansion) of the reaction products compared to the initial volume of the reaction partners.

In most instances, the onset of chemical attack requires a high moisture content of concrete and thus transport of moisture and of the aggressive medium to the place of reaction. Therefore, associated with chemical composition, concrete resistance to the entrance of aggressive media is the principal parameter controlling the resistance against chemical attack.

The severity of chemical attack depends primarily on the nature of the aggressive medium, its composition and whether it is solid, fluid or gas. Additional parameters are the rate of flow and the temperature of the adverse substance as well as the external pressure. Substances in natural water or soils which are adverse to concrete or masonry are in particular acids characterized by their pH-value, lime dissolving carbonic acid (CO_2), ammonium (NH_4^+), magnesium (Mg^{++}), as well as sulphate (SO_4^{--}) ions and, to some extent, soft water.

It is worth noting that severe damage occurs only if the aggressive media can penetrate the concrete by capillary suction, diffusion or mixed modes of transport at a sufficiently high rate. Thus, the most important precautionary measures to prevent damage and ageing due to chemical attack are the protection of the structure against water by suitable structural elements and production of concrete and mortar with high resistance against the entrance of aggressive media, i.e. a very low capillary porosity. Consequently, high-strength concretes, in most instances, show a significantly higher resistance against chemical attack than conventional concretes [25, 26].

In addition, suitable concrete-making materials have to be selected, e.g., sulphate attack can be avoided if sulphate-resistant cements with a low content of calcium aluminates are used. In extreme cases, however, additional protective measures such as coatings or overlay are necessary.

3.2.2 Early-age cracking

Durability of concrete structures is seriously compromised by cracking in early-age concretes, particularly in massive concrete structures like dams. Stress which arises in early-age concrete leading to cracking is mainly associated with three types of deformation (see Mihashi and Leite [27] for a review): autogenous shrinkage, induced by water absorption during hydration of cement particles; drying shrinkage, induced by evaporation of water along concrete curing and thermal shrinkage, owing to a poor dissipation of heat evolved by cement hydration and cooling of the hot concrete.

Autogenous shrinkage of concrete is defied as a reduction in the global volume of the cementitious material, caused by hydration of cement during and after the setting process. On the other hand, drying shrinkage is a volume deformation induced by evaporation of water from hardened concrete to the surrounding atmosphere. Considering the differences in shrinkage mechanism, high-strength concretes with low water–cement ratios (*w*/*c*) are prone to significant autogenous shrinkage, while ordinary concretes with high *w*/*c* are likely to incur in drying shrinkage. However, when water evaporates from the surface of early-age concrete along desiccation during the hydration reaction process, it is impossible to separate drying shrinkage from autogenous shrinkage that may simultaneously occur. Autogenous shrinkage and drying shrinkage share similar characteristics in that they are volume reductions induced by decrease of relative humidity (RH) during concrete hardening, yet they are very different in timing of stress development, as well as in mechanisms as mentioned before. While autogenous shrinkage may occur from few hours after setting has started, through several days until the hydration is completed, drying shrinkage may occur only after the surface is eventually exposed to environmental conditions, few days after setting starts, to the end of the curing process.

Another important difference is that autogenous shrinkage is a uniform deformation with no in-plane strain gradient unless the hydration heat distributes nonhomogenously inside the section or deformation is locally restrained. On the other hand, since drying shrinkage is induced by the loss of water inside concrete through the dam surface, resulting in nonhomogenous distribution of humidity within the member, strain gradients are always observed within the member sections. As the surface dries and attempts to shrink, it is restrained by the core volume. Consequently, cracking caused by drying shrinkage initiates from the surface area. Then, as the core volume dries, it undergoes shrinkage deformation which, when subjected to restraint, induces tensile stress leading to the development of cracks throughout the section. Note that initial cracking due to autogenous shrinkage may already exist. Around 5–10 h after hydration starts, at the time when the concrete microstructure starts to form, autogenous shrinkage deformation, if free, increases as hydration further progresses. Yet if it is restrained, significant tensile stresses may arise inducing cracking.

Thermal shrinkage differs from the former two in the fact that it is not related to moisture movements but to fluctuations in internal and external temperatures. Thermal shrinkage includes the effects of diurnal temperature changes, as well as the response of massive concrete structures to the heat generated during hydration. As cement hydrates in an exothermic reaction, a large quantity of heat is generated. Dissipation of such heat in large structures is relatively slow. The excessive rise of heat accelerates the hydration process and causes the concrete to set in an expansive condition. Then as the concrete cools, shrinkage occurs, which often leads to cracking.

In some cases, in concrete members subjected to loading during early age, creep strains gradually increase over time under sustained loads. Creep in concrete differs from the creep observed in other structural materials in the fact that it occurs at room temperature. When the concrete member is restrained, the reverse phenomenon may be observed as stresses gradually relax. Stress relaxation occurs not only in the presence of external loading, but also in the case of shrinkage stresses, adding further complexity to the phenomenon.

Early-age cracking mechanism involves the development or growth of strength, progress of autogenous and/or drying shrinkage, creep deformation and stress relaxation. Thus, when examining this mechanism, it is important to consider the strain component of each causal factor separately. In other words, the total strain may be defined by the equation proposed by CEB-FIP Model Code 90 [28]:

$$\varepsilon(t) = \varepsilon_{\text{elastic}}(t) + \varepsilon_{\text{creep}}(t) + \varepsilon_{\text{shrinkage}}(t) + \varepsilon_{\text{therm}}$$

The sum of $\varepsilon_{\text{elastic}}$ and $\varepsilon_{\text{creep}}$ represents the material mechanical response to stresses, i.e. stress-dependent strains, while $\varepsilon_{\text{shrinkage}}$ and $\varepsilon_{\text{thermal}}$ are stress-independent volumetric changes caused by moisture movements and thermal variations.

The accuracy of stress analysis of early-age concrete depends on how the required mechanical properties are described. As they rapidly change at early age, experimental campaigns of concrete at different ages are necessary. In particular, from uniaxial compression and three-point bending tests, the evolution of fracture toughness, both in tension and in compression, can be defined for early-age concrete. These intrinsic material functions serve as input for a thermo–chemo–mechanical analysis of the early stage of concrete dams. For instance, they were used by Lackner and Mang [29] to develop a 3D FEM analysis for the simulation of dams made of different RCC. If a normal RCC is used, no cracking was encountered during the analysis. In fact, the design of RCC dams aims at minimizing cracking, and the numerical predictions confirmed that. Conversely, in situations in which the cement content of RCC is increased and/or the environmental temperature decreases, early-age cracking can occur. It affects not only the upstream and the downstream faces, but also the central part of a dam.

3.2.3 Concrete swelling due to alkali-aggregate reaction (AAR)

There is a large number of concrete dams around the world which are affected by deterioration due to AAR [30]. The ARR is a chemical reaction between aggregates and the hydroxyl ions associated with alkalis in the pore fluid within concrete. It produces an irreversible anisotropic volumetric expansion of concrete and generally leads to loss of strength, excessive distortions and cracking. Three conditions must be simultaneously satisfied for the potentially harmful AAR concrete expansion to develop:

– The content of reactive aggregates must be in excess of a critical value.
– Sufficient alkali must be present.
– Sufficient moisture must be present.

The time evolution of the AAR concrete expansion process typically consists of:

– an initiation period where the concrete mass becomes saturated by the ARR reaction by-product, a silicate gel that absorbs moisture and then swells;
– a period of development of pressure and expansion in concrete;
– a period of rest upon depletion of reactive material.

The expansion rates of mass concrete affected by AAR have been typically found to vary between 0.02 and 0.2 mm per year in the development period. This leads to observed structural displacements varying from 0.1 to 5 mm per year in many dams, resulting in cracking and joint displacements of affected components.

Surface cracking is generally accelerated by the presence of wetting–drying and free–thaw cycles. The intensity and spatial distribution of ARR concrete expansion are significantly influenced by:

– Moisture content: approximately 75% RH within concrete is necessary to initiate significant expansion, which is assumed to vary linearly between RH = 75% and RH = 100%.
– Temperature: the reaction may stop below a limit value and accelerate at elevated temperatures. However, there is no indication that concrete in warm countries is more affected than concrete in colder countries. Specific data must be estimated on a case-by-case basis.
– The presence of applied or induced compressive stresses and other restraint/confinement mechanisms such as the presence of rebars.

To take into account all these factors, a thermo–chemo–mechanical analysis is therefore necessary to investigate the structure and decide upon repairs. There are many models (see Comi *et al.* [31] for a review) capable of simulating the swelling and the deterioration of both local stiffness and concrete strength due to AAR. However, in each numerical analysis, the objective is to distribute within the structure the displacements recorded by monitoring systems. In other words, according the steps illustrated in fig. 29, the structural analysis (i.e. the box n.1, labeled "AAR SIMULATION MODEL") must be calibrated against the displacements recorded from the monitoring system in rebar and concrete (i.e. the box n.11).

3.2.4 Erosion of concrete

The problem of erosion and abrasion of concrete dams and, in general, of concrete in hydraulic structures is well known and deeply investigated. According to ACI committee 210 [32], erosion is defined as the progressive disintegration of a solid by cavitation, abrasion or chemical action.

Cavitation bubbles will grow and travel with the flowing water to an area where the pressure field will cause collapse. Cavitation damage can begin at that point. When a cavitation bubble collapses or implodes close to or against a solid surface, an extremely high pressure is generated, which acts on an infinitesimal area of the surface for a very short time period. The erosion progresses rapidly and, after an initial period of exposure, slightly roughens the surface with tiny craters or pits. Possible explanations are as follows:

– The material immediately beneath the surface is more vulnerable to attack.
– The cavitation impacts are focused by the geometry of the pits themselves.
– The structure of the material has been weakened by repeated loading (fatigue).

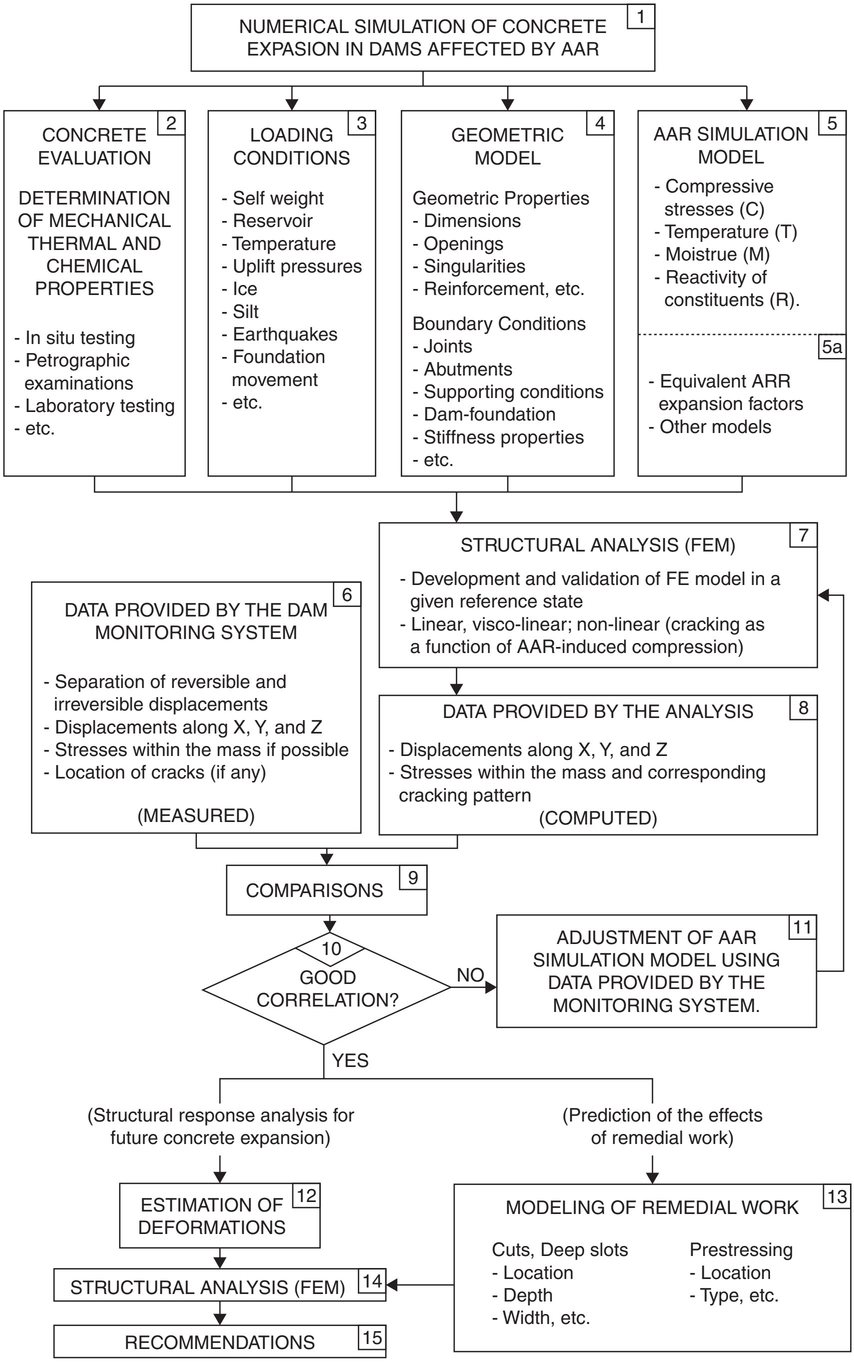


Figure 29: Numerical simulation of concrete expansion in dams [30].

There is a tendency for erosion to follow the mortar matrix and undermine the aggregate. Microfissures in the surface and between the mortar and coarse aggregates are believed to contribute to cavitation damage. Compression waves in the water that fills such interstices may produce tensile stresses which cause microcracks to propagate. Subsequent compression waves can then loosen pieces of the material. Once erosion has begun, the rate of erosion may be expected to increase because protruding pieces of aggregate become new generators of vapor cavities.

Once cavitation damage has substantially altered the flow regime, other mechanisms begin to act on the surface. These other mechanisms include high water velocities striking the irregular surface and mechanical failure due to vibrating reinforcing steel. Significant amounts of material may be removed by these added forces, thereby accelerating failure of the structure (fig. 30).

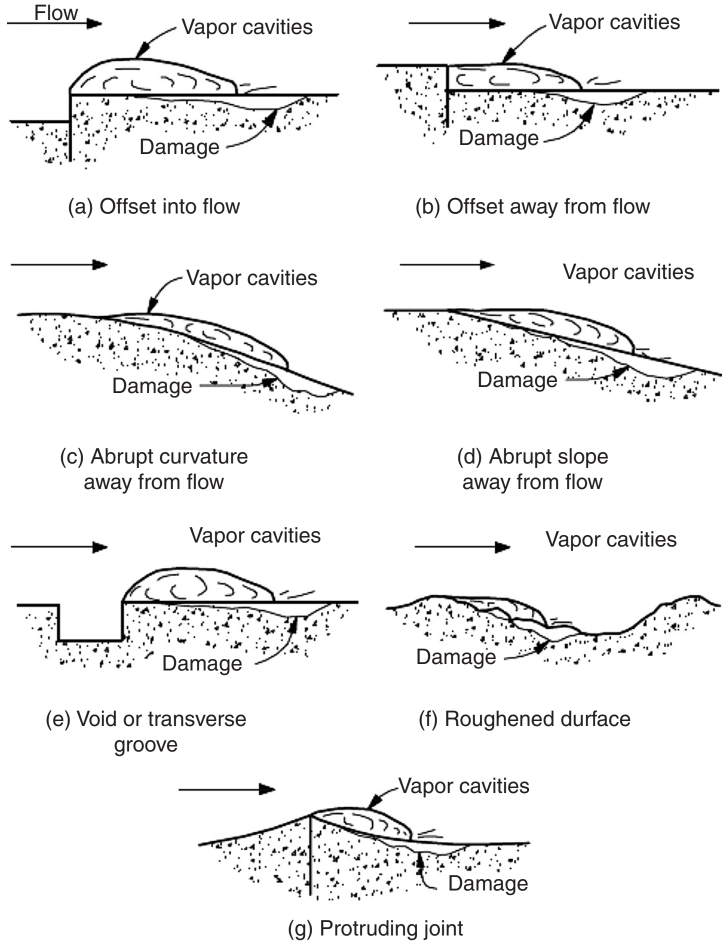


Figure 30: Cavitation situations at surface irregularities [32].

Abrasion erosion damage results from the abrasive effects of water-borne silt, sand, gravel, rocks and other debris being circulated over a concrete surface during operation of a dam.

Abrasion erosion is readily recognized by the smooth, worn-appearing concrete surface, which is distinguished from the small holes and pits formed by cavitation erosion. The rate of erosion is dependent on a number of factors including the size, shape, quantity and hardness of particles being transported, the velocity of the water and the quality of the concrete. While high-quality concrete is capable of resisting high water velocities for many years with little or no damage, the concrete cannot withstand the abrasive action of debris grinding or repeatedly impacting on its surface. In such cases, abrasion erosion ranging in depth from a few centimetres to several decimetres can result depending on the flow conditions. Moreover, decaying vegetation is the most frequent source of acidity in natural waters. Also deterioration of concrete in sewer systems resulting from bacterial action must be taken into account.

Ordinarily, if properly designed, constructed, used and maintained, concrete dams will work for years without problems. However, in particular conditions, erosion does occur in dams.

Cavitation erosion can be controlled through adequate design of the structure. When the value of the cavitation index is known, a designer can calculate velocity and pressure combinations that will avoid trouble. To produce a safe design, the target is to ensure that the actual operating pressures and velocities will produce a value of the cavitation index greater than the value at which damage begins. A good way to avoid cavitation erosion is to keep the pressure high and the velocity low. For example, deeply submerged baffle piers in a stilling basin downstream from a low spillway are unlikely to be damaged by cavitation because both of these conditions are satisfied.

A second, equally effective procedure to avoid cavitation is to use boundary shapes and tolerances characterized by low values of the cavitation index for incipient damage.

A third choice, often not avoidable, is to expect cavities to form at predetermined locations. In this case, the designer may: (a) supply air to the flow or (b) use damage-resistant materials such as stainless steel, fibrous concrete or polymer concrete systems. Using damage-resistant materials will not eliminate damage, but may extend the useful life of the dam surface.

Engineers who design dams, especially in particular geophysical conditions, are required to carefully consider all the unfavourable situations (e.g. presence of rapid flows during storm days, earthquakes, etc.) which can potentially produce erosions. Surface peel-offs, concrete disintegration, and sometimes rebar exposures are the main effects produced by erosion in several dams all over the world.

This explains the reason why finding an effective technology to control erosion damages for hydraulic systems like dams is of absolute necessity. Beside hydraulic solutions, the mechanical properties of concrete do play a significant role in enhancing the performance of concrete dams against erosion. As is well known, the construction of concrete dams involves a large quantity of concrete, and therefore a large quantity of heat is liberated while it hardens. In many cases, temperature distribution generates thermal gradients and tensile stresses within concrete masses. If such stresses exceed tensile strength, microcracking develops. The presence of microcracks, which can favour the potential erosion, depends on the environmental temperature. For instance, a sudden air temperature drop may cause surface cracks particularly in the early stage of cracking. Conversely, high temperatures induce rapid evaporation of mix-water and result in incomplete hydration and low strength.

All these aspects are intimately related to the cement type and to the mechanical properties of the hardened cement paste. Thus, some researchers [33] led their attention to the benefits of using low cement content together with appropriate amounts of fly ash and slag powder, commonly known as a high-performance concrete. The obvious advantage of using high-performance concrete with less cement content (LcHPC) is the low rate of hydration heat liberation that can significantly reduce the thermal gradient. Thus, it behaves like roller-compact concrete, but LcHPC, minimizing microcrack formation, can develop higher density and less porosity. The dense constituents strengthen the binding forces between cement paste and aggregates and enhance the ability to resist erosion attacks. A full-scale field test showed that the abraded depth of a dam with a top layer of LcHPC was approximately 25–35% that of a dam made of normal concrete.

3.3 Failures of concrete dams

At the end of 1980s, an ad hoc committee of ICOLD (ICOLD) presented a statistic of dam failures [34]. Firstly, they classified the possible scenarios, which can regard concrete dams (including foundations), earth and rockfill dams (including foundations), masonry dams (including foundations), appurtenant works and reservoirs.

In the field of concrete dams, failures can be due to:

- inadequate design;
- foundation (inadequacy of site investigations, deformation and lad subsidence, shear strength, percolation, internal erosion, tensile stresses at the upstream toe, leak of drainage system);
- concrete permeability;
- unforeseen actions or action of exceptional magnitude (uplift, overtopping of abutment or of main section) and
- structural behaviour of gravity and buttress dams (onset of tensile stresses).

Of a total of 142 dam failures, only 16 were related to concrete dams (11%). Most of them were mainly caused by foundation problems, whereas only two failures were ascribed to inadequate design and materials. The failures caused by the structural behaviour of the dam body regarded only five dams and generally occurred during the first filling operation.

3.3.1 The collapse of Gleno dam

The failure of a power dam at Gleno (Italy) is an emblematic case of disastrous break in a multiple-arch dam. It occurred on 1 December 1923 and the consequent flood caused about 500 casualties. The Gleno dam, built immediately after the First World War and completed during the spring of 1923, was a reinforced concrete structure of multiple-arch type, resting on a gravity base of stone masonry (fig. 31).

It was 43 m high above the stream and 263 m long on top, and had a capacity of 3,900,000 cubic metre. According to the original plan, a gravity dam was to be built. Subsequently, the company which held the franchise applied for a change in its franchise to permit the construction of a multiple-arch dam on the ground. While authorization for the change was pending, the company proceeded with construction, ignoring government orders to stop construction until the new plans had been approved. The dam was built on a solid masonry base, filling the lower part of the valley, and consisted of a superimposed concrete multiple-arch structure. The curved part of the superimposed structures mainly extended over the masonry base, whereas the straight flanking portions were founded directly on the rock of the valley. The buttresses of the structure at the junction of the straight and curved portion were made of extra thickness. The multiple-arch superstructure consisted of semicylindrical arches, 8 m in span between the centres of the buttresses and inclined 53° to the horizontal. Of the 25 arches, nine formed the central curved portion, four the straight portion of the left bank and twelve the straight portion of the right bank. Among these, nine arches fell, together with their buttresses. They were eight of the arches of the curved portion and the first arch of the tangent section on the left bank. The heavy buttress at the point of tangency also went out. The masonry base, to a large extent, remained in place (fig. 32).

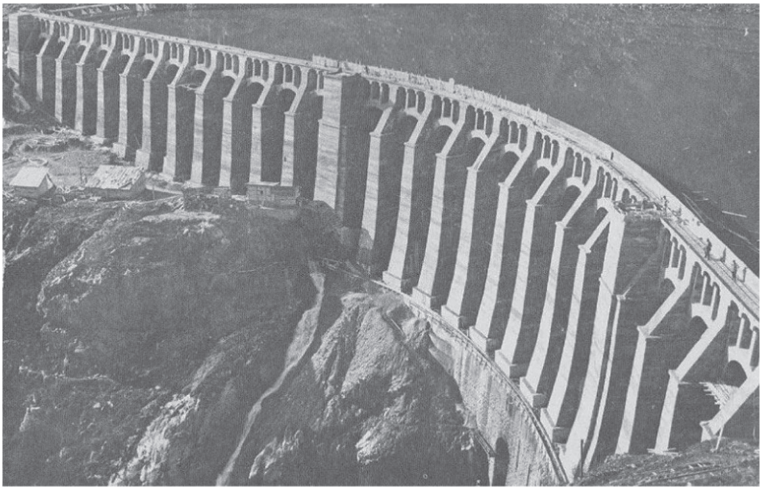


Figure 31: Gleno dam immediately after the filling [35].

As reported by De Martini [36], the cause of the failure was well defined by the engineers who first observed the Gleno structure: the work had been badly executed. The reinforced concrete part of the dam on the sides was placed directly on the rock surface, without being trenched into the rock. The masonry of the gravity base was made of lime mortar, whereas the specifications required cement mortar. The lime was burned near the site by the builders and transported to the dam by cableway. The gravel aggregate used in the concrete was not washed, and the concrete in the structure was porous. The reinforcement of the buttresses had been used during the war for protecting against hand grenades. During construction, some of the arches leaked, and part of the frame timber remained embedded in the concrete cast. Hand-mixed concrete was used for the arches, and the usual precaution of vibrating it in the forms was omitted. Summarizing, De Martini listed the following faults in the work:

– Failure to cut footings in the rock for the buttresses of the dam
– Use of improper materials and lack of inspection
– Poor mixing and lack of inspection of the concrete
– Use of unwashed aggregate
– Lack of inspection in pouring the concrete
– Failure to ram the concrete in the forms
– Generally incompetent direction and supervision.

After the collapse of Gleno dam, the Italian government undertook a series of actions, and, in particular, the first code requirements about dams were issued immediately after the tragedy [37]. In fact, this failure gave the significant warning that concrete dams are hazardous structures, and can be made safe only

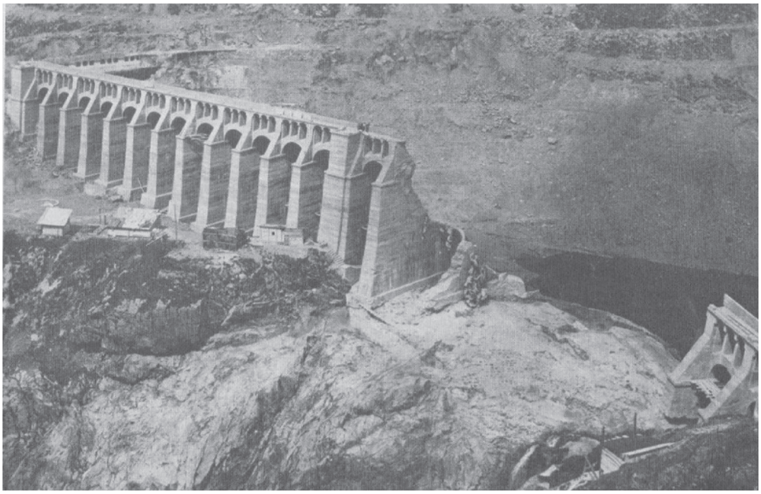


Figure 32: Gleno dam immediately after the failure [35].

through skilled engineering and deep control of both design and construction works. As stated by Engineering news-record after the failure, "*convulsions of nature have never destroyed a dam; negligence and errors of their builders have destroyed many.*"

4 Stability of rock foundations of dams

This paragraph deals with the methods available to assess the stability conditions of rock foundations of dams.

A dam [38] radically alters the natural antecedent state of stress in the valley flanks and river channel; it adds weight, a more or less concentrated vertical load resulting in a complex compression and shear forces in the foundation. It transmits forces caused by loading during its operational life (water load, temperature effects, etc.). These forces produce compression, shear and often upstream tensile stresses. Water seeps into pores and rock discontinuities of the foundation; seepage forces can be considerably high and act in extremely unfavourable direction for stability of dams.

The main steps of stability analysis of such a complex structure made up of rock mass and concrete structure consists of the definition of the possible modes of failure and calculation of the equilibrium conditions for stability.

Geological and rock mechanical studies are required to evaluate the possible modes of failure and the variables of the problems.

Analytical or numerical methods are used to assess dam foundation stability conditions.

4.1 Geological and rock mechanics studies

Geological studies with field explorations help to determine the feasibility of a dam and decide the general layout of the works including dam type and position.

The first geological work [38] is to outline the regional and thereby the site structure, then the genesis and history of rocks and hence their stratigraphic, petrographic and tectonic descriptions which indicate the type of the problem to be expected. High-scale hydrographic and hydrogeological studies are required since a dam construction determines the presence of a new water reservoir which changes the original superficial and underground water flow.

The principal purpose of rock mechanics studies is the characterization of the rock mass hosting the dam in order to assess deformation and strength features and to evaluate the geometrical and physical properties of singular and systematic rock discontinuities.

At a preliminary stage of the design, air photogrammetric survey [39] is an irreplaceable method for the geological reconnaissance of rock type, major structures, faults, dikes and lythological contacts. Air photogrammetric survey at the scale of hundreds of metres can be also used, alternately to classical *in situ* scanline technique, to recognize domains where systematical joint set features such orientation and spacing.

The principal methods of a quantitative description of rock masses and their discontinuities are the survey of rock exposure, the drill core and drill hole observation, and the use of terrestrial photogrammetry.

The photogrammetric method is a practical alternative to rock exposure survey when the examined rock face is close to magnetic anomalies or if the rock face is unstable or inaccessible. Photogrammetric and laser scanner technique can also be used in conjunction with rock exposure survey to allow a better definition of geometrical discontinuity parameters such as orientation, spacing or persistence.

In drill core and drill hole method, the axes of each drilling represents a sample line of the rock mass.

Drill core can be useful for the determination of some rock mass quality indexes such as RQD and for the assessment of discontinuity parameters such as spacing or frequency. Specimens from drill core can be used for laboratory tests.

Rock exposure survey is the most common method used to assess the parameters required for the quantitative description of discontinuities. Rock exposures are objectively surveyed (randomly) when the sampling is carried out by setting fixed lines (scanlines) onto the rock mass surface.

The surveyor can carefully and systematically work along the scanline by measuring and recording the discontinuity features required for a quantitative description. The ten parameters selected to describe discontinuity and rock masses are [40] orientation, spacing, persistence, roughness, wall strength, aperture, filling, seepage, number of sets and block size.

Rock discontinuity shear behaviour can be assessed by laboratory direct shear tests. Uniaxial and triaxial laboratory tests allow the determination of the uniaxial compressive strength, intact rock friction angle and cohesion. Free scale shear strength properties can be evaluated by *in situ* tilt test. Push and pull test also allows to estimate *in situ* shear behaviour at a real scale.

In situ plate loading tests are used to evaluate the bulk modulus of rock masses. *In situ* pumping tests are used to estimate rock mass permeability, discontinuity conductivity and aperture.

Geophysical measurements are also used in the rock mass characterization. Seismic refraction is used to measure the thickness of weathered rock or soil cover. This method allows, at a minimum cost, the zoning of depth in terms of compressive wave velocities, which can be correlated to the degree of fracturation, and to have the basic information on a site part without drillholes or adits. Seismic logging of boreholes is carried out to correlate the shear wave velocity of the signal to the rock mass fracture density. Cross-hole seismic method can also be used to measure the seismic velocity between boreholes, between a borehole and the ground surface, and between adits, horizontally and vertically.

Rock masses can be considered as a medium constituted by rock matrix and discontinuities. Discontinuities are, from a mechanical point of view, weakness planes and, from a hydraulic point of view, preferential water flow paths that play a fundamental role in the rock mass permeability determination. Rock mass can be described as an equivalent continuous medium or as a discontinuous medium. When the discontinuities are smeared off the rock mass in such a way as to homogeneously weaken the rock matrix, the consequent medium is defined as an equivalent continuous medium. When the description of the medium is performed by defining the rock mass as a system of blocks, the medium is considered as discontinuous medium.

Rock mass classifications are used to characterize equivalent continuous medium: RMR [41] and *Q* [42] [43] classification methods are the most used in engineering practice. These classifications divide the rock mass according to different qualitative judgments, from very good to very scarce rock mass. These qualitative judgments are obtained by taking different discontinuity and rock matrix properties into account. The particular values determined for these properties correspond to rates which are given in the Classification tables. The final value of a rock mass index is given by adding all these ratings. The final result is the RMR or the *Q* quality index, according to the adopted classification method chosen. The qualitative judgment of the rock mass is determined by the numerical range to which the examined rock mass RMR or *Q* belongs.

Hoek and Brown [44] shear strength criterion is the most widely criterion used in engineering practice to estimate the shear strength of a rock mass.

Starting from the RMR index obtained by classifying the rock mass and considering the rock lithotype the strength parameters of every kind of rock masses can be determined.

In situ tests and RMR or *Q* correlations can be used to assess the deformation features of rock masses. Elasto-plastic behaviour is often used to simulate the stress–strain rock mass behaviour. Mohr–Coulomb or Hoek–Brown criteria are used to determine the limit between elastic and plastic behaviour. Elastic ideally plastic model is the most widely used to simulate the rock mass stress–strain behaviour as there is not often enough information to correctly simulate the post-peak or the plastic behaviour.

Numerical analysis of the interaction between rock masses, concrete structure and water flow can be carried out by using numerical methods such as finite element method (FEM) or boundary element method (BEM).

When the rock mass is considered a discontinuous medium, stability analysis are then carried out by using the limit equilibrium method or numerical methods capable of analyzing the mechanical behaviour of a system of blocks.

Since the rock mass is considered as a medium intersected by discontinuities, taking the ubiquitous geometrical features of joint set into account, limit equilibrium analysis is carried out by examining the static conditions of rock blocks formed by discontinuity intersections and facing on rock walls. In this case, only discontinuity shear strength properties are required for a stability analysis.

The mechanical analysis of a system of blocks are instead carried out by numerically simulating the geological history of the rock mass, the different dam construction phases and water reservoir filling or discharging phases. In this case deformation and strength features of rock matrix and rock joints are required.

Numerical flow analysis can be carried out and conductivity properties of joints are required as the time variation of the piezometric heads during the simulation period. Numerical methods used for this kind of analysis belong to the discrete element method class (DEM).

4.2 Stability computations

Stability analysis refers to portions of rock foundation which are located on the dam base or on the valley flank. The stresses induced by the dam weight and the water pressure resulting by the water reservoir seepage into rock discontinuities can trigger rock sliding and toppling.

Rock slides and topples may be subdivided, according to the number of units in the movements, into slides or topples of a single or few units and slides and topples of many units.

In the first case, limit equilibrium methods that examine static equilibrium equations are considered. In the second case, methods that examine the dynamic equilibrium equations are used according to the importance of relative displacements between the different block units involved in the phenomenon.

When a rock mass is considered as a continuous equivalent medium, a FEM or BEM model can be set up to discretize dam concrete structure and rock foundation and to carry out a stress–strain analysis.

4.2.1 Limit equilibrium methods

The first step of the analysis is to construct all the possible rock blocks formed by the surveyed discontinuities and the piezometric surface to only determine the convex removable blocks.

The second step is to determine the blocks that have the kinematical possibility to slide, topple or fall.

Kinematic tests of rock block slide, topple or fall in a dam analysis are different from classical kinematic tests of a slope stability problem, where only gravity is present, since the weight of the dam determines a concentrated load acting with an inclined direction on the rock blocks.

Londe [45] developed a method of three-dimensional analysis of tetrahedral rock blocks.

The solid is formed by the intersections of three discontinuity planes and two free surfaces (fig. 33). The acting forces are the weight of the blocks, the force which represents the action of the dam structure on the rock block and the uplift forces due to the water pressure acting on the discontinuity planes.

The kinematic test is carried out only after than the resultant of the external and body forces is computed.

The safety factor is computed with regard to possible sliding movement which can occur on three joint intersection lines or on the three joint face dip direction.

The safety factor is determined from the ratio between shear strength resistance force and the active force. Seismic events are usually simulated by means of

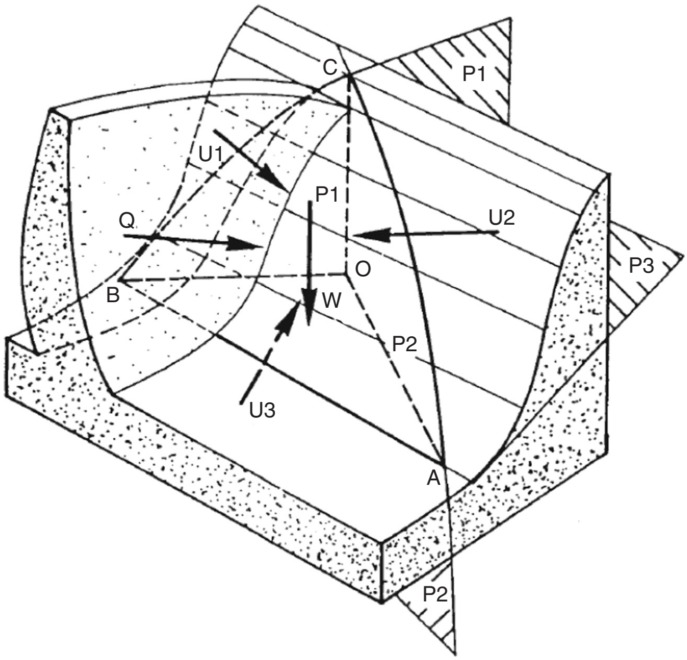


Figure 33: Stability of tetrahedral rock volume. OABC is the tetrahedron; P1, P2 and P3 are the rock discontinuity planes; *W* is the weight of the rock solid and *Q* is the dam thrust (after [46]).

a pseudostatic analysis where horizontal and vertical body forces are added in the statical scheme to take maximum seismic acceleration into account.

Limit equilibrium analysis can also be carried out by using the block theory [47]. The method allows to construct rock blocks made up of an unlimited number of joints. Block theory is a vector analysis method which provides the formulation for all the quantities relating to block morphology in a rock mass, including the volume of a block bounded by joints, the area of each of its faces, the position of its vertices and the position and attitudes of its faces and edges (fig. 34). The geometrical characteristics of the analysed blocks depend on the joint geometrical data gathered from the geomechanical survey: orientation for the block shape, persistence and spacing for the block volume.

4.2.2 Statistical and parametrical analysis

The geometrical and physical variables of the problem are regionalized variables with a spatial variation. Moreover, even though a large number of joint features have been surveyed and a large number of laboratory tests have also been carried out, we can have an acceptable value of the standard error of the mean values and of the statistical distribution of the variables of the problem.

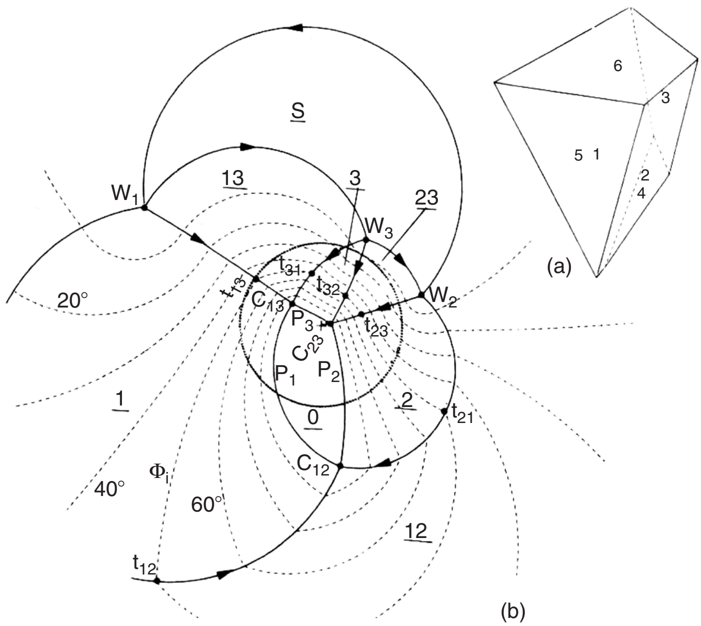


Figure 34: Equilibrium region of a rock joint pyramid (after [39]).

A deterministic approach is now widely recognized as inadequate and probabilistic analyses are carried out to assess the probability of collapse. The design problem becomes to choose what failure probability can be accepted for any particular case.

Several probabilistic methods are used in the engineering practice; some of those can be used for the different particular phase of the stability analysis. Geostatistics can be useful in the evaluation of the spatial variability of joint geometrical feature. Rosemblueth [48] method and Montecarlo simulation [39] can be used to take the joint strength feature variation into account. Fuzzy set method [49] can be applied when there are not enough data to set up reliable distribution of frequency of the variables of the problem.

Parametrical analyses are often used to evaluate the importance of the single variable of the problem on the final result.

4.2.3 Numerical methods

The interaction between the dam and rock foundation can be adequately simulated only with three-dimensional numerical models working in elasto-plastic field. Numerical methods discretize the modelled medium. While concrete can be considered as a continuous medium, the rock mass exhibits discontinuities at two different scales. First, there are randomly distributed flaws of the size of some millimetres in intact blocks of rock. The mechanical parameters of rocks decrease with increasing size. Unconfined stress tests [50] indicate that the resistance of different kind of tested rocks decreases with an increase in the specimen size, reaching a constant value at around 1.5 m edge length. An explanation of this phenomenon is that larger samples are likely to contain more flaws in "critical locations."

A second scale of discontinuities is that of joints, which are some metres in size and are systematically distributed over a rock mass. Rock volumes containing a number of existing discontinuities are also scale dependent. The size effect mechanism [51] is that larger volumes contain more blocks that provide greater freedom to develop failure mechanisms. Smaller volumes of rock must involve intact rock failure, thus increasing the overall strength, while larger volumes are more likely to contain through – going pathways comprising of existing joint, which supplies a weakening effect.

In a rock engineering work, the scale of the problem (e.g. excavation size or sample size) needs to be compared to the fracture network pattern (e.g. block size or discontinuity spacing) for strength and deformation properties to be estimated for a rock mass. Cunha [52] suggests that the rate of decrease of mechanical property values diminishes significantly above a certain scale, which is referred to as the representative elementary volume (REV).

When the problem scale reaches the REV, a rock mass is often considered treatable as a continuous material and FEM models can be used for numerical simulations.

Hoek and Brown [53] state that in order to apply their empirical strength criterion, the slope or underground excavation should be "large and the block size small in comparison," resulting in the appearance of a "heavily jointed" rock mass.

Barton [54] suggested that a rock mass can be considered continuous, a discontinuous or equivalent continuous according to their quality index Q. When a rock mass is heavily jointed and it's Q value is lower than 0.1, it can be considered as an equivalent continuous medium and FEM or finite difference method can be applied to numerical modelling. When a rock mass can be defined fair or good and the Q index is in the 0.1–100 range the medium can be considered as discontinuous and a DEM can be used for numerical modelling. When a rock mass can be defined as very good or extremely good and the Q index is greater than 100, it can be considered as a continuous medium and FEM or BEM can be applied to numerical modelling.

A scientific discussion happened in these last 10 years on the use of Hoek and Brown criterion to define rock mass mechanical properties for the cases in which the REVs are 20–30 times greater than the block size. A new classification index named GSI (geological strength index), depending on the RMR and the block size, was introduced for this purpose (fig. 35) [55]. The strength features of rock mass are determined in their turn from the evaluated GSI and a continuous equivalent approach can be used for numerical models.

Pierce *et al.* [56] developed a new method to generate numerical representations of jointed rock masses in a scheme termed synthetic rock mass (SRM), in order to study the effects of joint network geometry and scale on block size

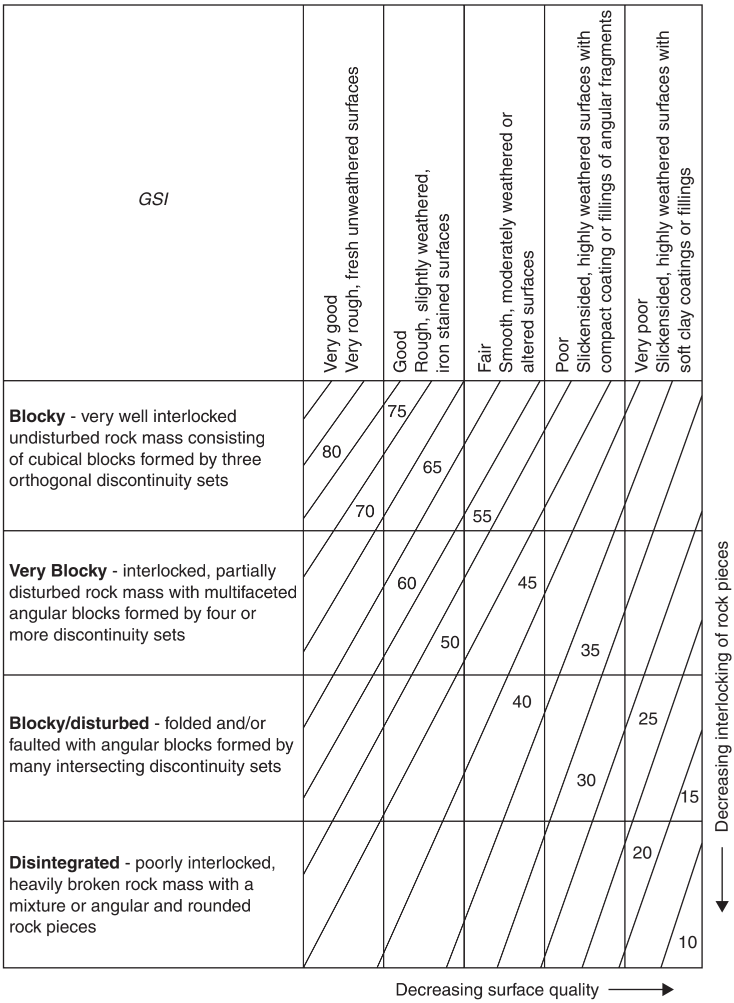


Figure 35: Quantification of the GSI system (after [55]).

and rock mass strength. Unlike previous approaches (Itasca, 2004), the SRM methodology allows for consideration of a complex non-persistent joint network as well as block breakage that includes the impact of incomplete joints on block strength.

3D computer codes were set up in these last 20 years for coupled hydrologic–mechanical analysis able to consider the modelled medium as a continuous equivalent or a system of blocs (3DEC).

Rutqvist *et al.* [57] proposed a modelling approach for analysis of coupled multiphase fluid flow, heat transfer and deformation in fractured porous rock.

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