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CHAPTER V

Small concrete dams

By Paul ROYET (Cemagref)
With the participation of: Gérard DEGOUTTE (ENGREF), Max GIRARD (SOMIVAL), Michel LINO (ISL), Jean TEYSSIEUX (COYNE & BELIER), and Georges TRATAPEL (CNR).

In the introduction to this volume, we defined small dams as those under approximately 25 metres high. In this chapter, that height, which is for convenience only, is the height of the dam above the foundation. It is in fact essentially at the dam-foundation contact that overall dam stability is analysed.

No specific discussion is made of very small dams, i.e. less than 10 metres high above the foundation (for example weirs), although many of the recommendations given hereafter are applicable to such structures.

This chapter does not intend to be a treatise on small concrete dams, but rather to highlight the specific features of those dams in terms of choice of this alternative, design and construction techniques. In a word, it is intended to show how design of small dams may differ from design of large dams.

This chapter first looks at the criteria resulting in choice of a concrete dam, and then the selection of a subcategory: conventional concrete or roller compacted concrete (RCC) gravity dam, symmetrical hardfill dam or arch dam. Then design and dimensioning of conventional gravity dams is addressed, whether conventional or roller compacted concrete is chosen. The special features of hardfill dams are also given detailed examination, as are those of the barrage type dams routinely built on rivers developed for navigation.

Choice of a concrete dam

Why opt for a rigid dam structure?

In France, a rigid structure is rarely chosen in small-scale projects. Statistically, there are many more fill dams built than rigid structures.

What are the most usual reasons for choosing a rigid structure?

- the need to discharge high floods;
- requirements to fulfil complex functions (e.g. a gated structure to flush sediment and ensure a long service life for the reservoir, high discharge bottom outlet, etc.);
- unknown hydrological factors, as rigid dams are generally less vulnerable to overtopping than fill dams. At sites where flood values are highly uncertain, rigid designs often bring advantages, for example limiting the required temporary diversion works and providing a greater margin of safety from risks of hydraulic origin. However, it should be noted that the stability of small gravity dams is very sensitive to maximum water levels.

As a general rule, a concrete dam will be considered whenever the discharge structures are important to the project, which is often the case for diversion dams at hydroelectric schemes.

What are the pre-requisites in designing a rigid dam?

The first condition concerns *foundation quality*. The rule that a rigid dam requires a good quality rock foundation gives a good first approach. That rule must be applied without restriction for small arch dams, which require practically undeformable foundations. For gravity dams, it will be possible, to a certain extent, to adapt the cross section to foundation quality by designing more gentle slopes.

The second pre-requisite for construction of a rigid dam is the availability, under acceptable economic conditions, of high quality aggregate (invulnerable to frost and with no risk of concrete swelling). These two requirements are in fact often met simultaneously.

Mechanical functioning of rigid dams

A distinction must be made between arch dams, which transmit hydrostatic thrust by "arch effect", i.e. by transferring that thrust to the river banks by means of compressed arches, and gravity dams, where balance is achieved by means of the dam's weight, making use of the forces of friction on the foundation.

Arch dams

Arch dams transfer hydrostatic thrust to the foundation by means of arches working in compression. It is the geometry of the arch and the difference in stiffness between the concrete and the rock that determine how the dam functions. The search for an ideal shape is in fact aimed at transmitting thrust by entirely compressed arches. Traditionally, arch dams have been designed with maximum stress in the compressed arches limited to 5 MPa, which corresponds to a factor of safety of 4 or 5 for average quality concrete. That condition determines the thickness of the arch and the formula (σ = p.R/e) is still an effective technique for preliminary dimensioning of small arch dams.

The result is a set of four pre-requisites for design of an arch dam (small or large):

- *topography:* the valley must be narrow. Arch dams have been built on sites where the crest length to height ratio (L/H) is close to 10, but generally arches are more interesting when L/H is less than 5 or 6;
- *foundation stiffness:* for "arching" to work, the foundation must be sufficiently stiff, or the arches will not reach their abutments and the structure will tend to work as a cantilever. To give an idea of magnitude, an arch should not be considered without detailed studies when the rock's deformation modulus (measured by jack testing or "petite sismique" techniques) is less than 4 or 5 GPa;
- *foundation mechanical strength:* it has already been stated that an arch transmits high stresses to the foundation, which must remain within the elastic range for the forces in question;
- *stability of foundation blocks,* under the effect of uplift and given the compression due to the arch, which may prevent uplift pressures from dissipating.

When detailed site investigations on the foundation can prove that all these pre-requisites are met, an arch dam is often an economical solution for small dam projects as it keeps the volume of concrete to be placed to a strict minimum. For example, a cylindrical arch 25 metres in radius for a dam 25 metres high will be approximately 1.25 metres thick if a maximum compressive stress of 5 MPa is considered, in accordance with the formula (σ = p.R/e), while an average thickness of 10 metres would be required for a conventional gravity design.

In addition, design and construction are quite simple for dams under 25 metres high, provided the designer contents himself with simple geometrical shapes.

An arch dam also offers the advantage of being relatively invulnerable to overtopping, as long as it does not last very long and is of only modest extent (because of the risk of erosion of the dam toe). This type of dam therefore tolerates an underestimated design flood. However, we shall not go into arch dams in greater detail as experience has shown that such a design has rarely been used in France in the past few decades for dams less than 25 metres high above their foundation (BLAVET, CHAPEAUROUX[1], LE PASSET).

1. Photo 22 p. VIII.

Gravity dams

A gravity dam functions in a completely different way: it is the weight of the dam (and not its geometry) that balances hydrostatic thrust and uplift (*see photo 21*).

Uplift is not generally taken into consideration for arch dams, as the thin cross section reduces the role of uplift to a negligible element. On the other hand, for gravity dams uplift plays a major role in balancing forces.

A classic stability study for a gravity dam consists in analysing the general stability of the dam or part of it under the effect of weight, hydrostatic thrust, uplift and possibly other secondary actions (e.g. pressure of sediment or earthquake).

The criteria for dimensioning the dam concern distribution of normal stresses (limiting tensile stress at the dam heel and limiting compressive stresses) and the slope of the resultant. This method of calculation reveals the essential role of uplift in the stability of a gravity dam, and therefore the importance of drainage.

Maximum compressive stress under a traditional gravity design with vertical upstream face and downstream batter of 0.8H/1V is 0.35 MPa for a gravity dam 25 metres high, one tenth that for an arch dam of the same height. The slope of the resultant varies from 27° to 42° depending on drainage conditions.

Finally, it should be noted that a concrete gravity dam is a rigid structure; the modulus of conventional concrete is about 25 GPa, which is generally higher than the modulus of the rock foundation the dam rests on.

This brief description of how a gravity dam functions mechanically is the reason for the main requirement imposed on a concrete dam, which is the need for a good quality rock foundation. The condition of low deformability is generally the most severe, in particular for soft or weathered rock foundations, but conditions as to shear strength will also preclude a gravity design when the foundation's shear strength is low (marl foundation, subhorizontal clay joints in the foundation, etc.).

MATERIALS USED: HISTORY

Masonry

Historically, the most widely used construction material has been masonry, both for arch dams (Zola dam in France, very old dams in Iran, etc.) and for gravity dams.

In France, a large number of masonry gravity dams were built in the 19th century to supply canals and for water supply. Most of these dams have behaved very well over the years despite cross-sections which would be unsatisfactory in modern design. However, it is noteworthy that one of the most catastrophic dam failures in France was that of Bouzey dam, a masonry gravity dam with an unsatisfactory profile. Analysis of that failure revealed the major role played by uplift in the dam body, which had never been considered until that event.

Masonry dams are no longer built in France, mainly because the technique is labour-intensive due to the requirement to cut and place facing rock. However, the technique is still used in some countries (China, India, Morocco, Sahelian Africa, etc.) for small dams.

Conventional concrete

The technique of building gravity dams with conventional vibrated concrete (CVC) was developed starting in the second decade of the 20th century. It resulted in a very great number of dams of all sizes for all kinds of use.

The technology of conventional concrete gravity dams uses coarsely graded concrete aggregate (up to 80 millimetres) and cement contents in the range of 200 to 250 kg per cubic metre of concrete. Heat development relating to the hydration of the concrete as it sets leads to high increases in the concrete temperature and a risk of cracking as it cools.

For that reason, conventional concrete dams are built in blocks routinely measuring 15 by 15 metres horizontally, which means many contraction joints, both transversal and longitudinal, must be placed (at least in the case of high dams). For small dams, it is generally possible to build only transversal joints.

The dam is given its monolithic character by placing shear boxes and by grouting the joints between the blocks.

The technique of building conventional concrete gravity dams, like masonry, is labour-intensive, in particular to set formwork. This requirement for labour and the parallel development of modern earthwork techniques at high work rates have resulted in a progressive preference for earthfill or rockfill dams instead of concrete gravity dams.

Roller compacted concrete (RCC)[1]

The renewed interest in gravity dams is the result of the invention of RCC, which is a major technical innovation in dam technology.

That innovation consists in placing and compacting the concrete, not by traditional means (transport by crane or cableway and compaction by vibration) but rather using the techniques of earthworks, e.g. dump truck transport, spreading by bulldozer and compacting with a heavy vibrating roller. However, this construction technique requires a working surface area greater than approximately 500 m^2 to allow the machinery to travel freely.

The possibility of reducing to a strict minimum the quantity of mix water, and good compactness by compacting in 30 cm layers, make it possible to limit the quantities of cementitious material to values of 100 to 150 kg/m^3 in order to decrease heat development.

In fact, this new construction method does not readily accommodate the many joints used to control cracking of thermal origin in conventional concrete.

1. See *Bibliography*, reference 5, p. 139.

In today's design of RCC dams, only transversal joints are used, generally spaced much more than the 15 metres apart that is usual with conventional concrete dams.

One of the major advantages of RCC, in particular in developed countries, is the speed of construction: the body of a small dam can be built in only a few weeks, which reduces the cost of capital equipment, engineering, and often river diversion as a dam can be built during low flow periods with a minimum of diversion works.

In France, RCC technology has taken an original path. RCC has often been used to build the body of a gravity dam at least cost, but as it does not guarantee watertightness, a special element is required:

- PVC membrane at Riou dam (in the French Alps)[1];
- reinforced concrete wall built as the RCC is placed and serving as formwork for the upstream face at Petit Saut (French Guyana) and Sep (Central France) dams;
- reinforced concrete upstream face at Aoulouz dam in Morocco, designed jointly by French and Moroccan engineers.

With this design, the RCC materials used in the dam body are essentially unsophisticated materials of variable mixes chosen according to the availability of components to create the least cost mix on site. Cementitious material contents are low, of the order of 100 kg/m^3, and total fines content is at least 12% or thereabouts.

Hardfill[2]

With the prospect of considerable savings in making RCC, attempts have been made to further decrease cementitious material content and use natural alluvial material, if possible with no prior treatment. However, the dam design must be tailored to the acceptable stress levels for such material. This has given rise to the concept of hardfill with the following characteristics:

- symmetrical cross-sections between 0.5H/1V and 0.9H/1V (to give a general idea), with optimum mechanical strength achieved with slopes of 0.7H/1V;
- separation of the watertightness function, which is provided by the upstream face, from the stability function, which is provided by the hardfill body[3] ;
- use of hardfill, which is actually an RCC with maximum savings through use of natural materials with minimum treatment and cementitious material content (about 50 kg/m^3) ;
- a hardfill deformation modulus that can be estimated at a significantly lower level than that of CVC but that of course will depend on the nature and grading of the aggregate as well as the cementitious material content.

The symmetrical cross-section transmits only low pressures to the foundation. With only dead weight, stresses are uniform at values approximately half of those under the upstream heel of a conventional gravity dam. Filling and operating the reservoir cause only slight changes in normal stresses and the entire concrete/foundation interface remains practically uniformly compressed.

1. Photo 23, p. VIII.
2. See *Bibliography*, reference 2, p. 139.
3. For some dams, e.g. flood routing projects, the facing can be dispensed with.

Finally, the slope of the resultant to the vertical is very modest (14° to 22° depending on drainage conditions).

These characteristics mean that such a gravity dam can be considered on a rock foundation of mediocre quality that would not be suitable for construction of a traditional gravity dam. The symmetrical dam still presents the advantages of a rigid structure in terms of its hydraulic functions and can accept a rock foundation of poor mechanical characteristics (which in other words means it can be founded on subsurface layers and not necessarily on deeper-lying sound rock).

The limited changes in stresses during reservoir operation, along with construction of the upstream face after construction of the dam body, make it possible to accept such a foundation: in fact, because of thermal effects and settlement in the foundation, the risk of cracking is maximum at the end of construction, i.e. before the watertight element is placed.

Let us simply add that the symmetrical hardfill dam behaves well in earthquake conditions, and can support high construction period floods with no major damage.

Several dams have already been designed on these principles, in particular in Greece, Spain and Morocco. One dam of medium height (25 metres) has been built in Greece.

CONCLUSIONS ON THE CHOICE OF A CONCRETE DAM

In lieu of a conclusion, just a few comments:

- a concrete arch is still a good alternative for rock sites in narrow valleys, especially if the dam must accommodate major discharge structures;
- masonry gravity dams, despite their excellent performance, seem to be reserved for a context where labour is abundant;
- conventional concrete gravity dams are generally only warranted when there are complex discharge structures, in particular for barrages;
- the RCC gravity design has been revealed to be an economical and reliable alternative, whenever concrete volume exceeds 35 000 to 40 000 m^3.
- the symmetrical hardfill dam with watertight facing should be considered for difficult sites with rock foundations of poor mechanical characteristics, high floods or risk of earthquake.

The classic gravity dam (CVC or RCC)

By the term classic gravity dam, we mean a conventional concrete or RCC dam with upstream face subvertical ou nearly subvertical a downstream batter of the order of 0.8[1].

This is the most commonly encountered type of small concrete dam. The massive structure withstands the pressure of water and uplift thanks to its dead weight.

In comparison to an arch or buttress dam, design and computation of such dams are still very simple, and no sophisticated techniques are needed to build them. The amount of formwork required is reduced, but concrete volume is greater.

Foundation

The classic gravity dam should be built on unweathered rock, except in special cases that require specific measures to be taken (*see Foundation treatment p. 121*).

The requirement of good quality rock is of course less strict than for a large dam (maximum stresses are considered proportional to height as a first approximation). However, there are three arguments in favour of a good quality foundation:

- the dam's rigid structure can hardly accommodate differential movements;
- the diagram of the stresses transferred to the foundation is radically different between a situation with a full reservoir and that with an empty reservoir, which can cause fatigue in a poor quality rock as the reservoir is emptied and refilled;
- hydraulic gradients in the foundation are high and could result in internal erosion if the rock is of poor quality.

When several metres depth in the foundation consist of loose soil or decomposed rock, the fill dam alternative will naturally be preferred for small and medium sized dams. In fact, except in special cases, availability of fill materials on site inspires a preference for that alternative under present economic conditions, given the performance of modern earthmoving plant. However, it is true that in some countries small earthfill dams are routinely accompanied by a massive concrete spillway, comparable to a gravity dam and usually resting on a soft foundation. This alternative, rarely used in France, is limited to dams only a few metres high, and requires special precautions to control hydraulic gradients in the foundation.

1. Downstream batter is here defined on the line between the downstream toe of the dam and the point in the upstream face situated at normal reservoir water level.

FOUNDATION TREATMENT

Hydraulic gradients in the foundation (and in the dam body) are just as high for small gravity dams, as for large dams.
Despite common notions on the subject, foundation watertightness must therefore be monitored just as rigorously.

The dam's foundation level is unweathered rock, which is usually cracked. Grouting is therefore required in most cases, including at small dams. However, to save money on this operation, grouting is often carried out in a single stage.

If the dam has a gallery grouting will be in drillholes from this gallery (*see figure 1-a*). The gallery's dimensions and access to it must therefore be sufficiently large for drilling machinery (which is now fairly compact, it is true). As an indication, a minimum of 2.0 metres in width and 2.5 metres in height can be considered. When there is no gallery - which is most often the case for small dams - grout holes must be drilled from the downstream toe (*see figure 1-b*). Where appropriate, for dams of a certain size or on a mediocre foundation, the grout curtain is flanked on either side by two lines of fairly shallow groutholes drilled when the excavations begin, which obviously requires two separate grouting stages.

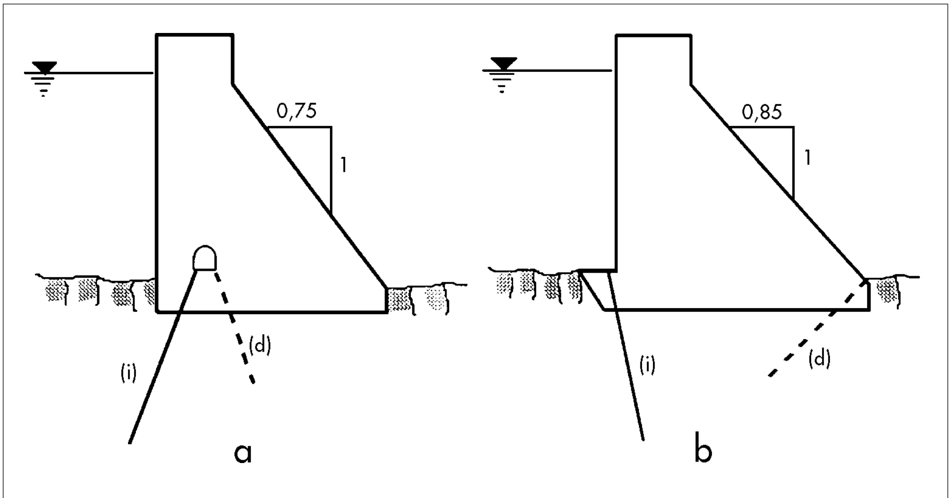

Fig. 1 - Layout of the grout curtain (i) and the drainage curtain (d): a - with gallery; b - without gallery

It must be verified that the area around the grout curtain is always compressed in any load combination.

In the first few metres depth, grout pressure must be limited (not exceeding 0.5 MPa) to avoid bursting the rock and heaving up the dam.

The use of more fluid grout results in the same quality of treatment at lower grout pressure.

Drainage

The stability of classic gravity dams is strongly linked to the uplift under the structure. Foundation drainage is therefore advisable. However, to be truly effective drainage must be set farther upstream, meaning from a gallery (*see figure 1-a above*). But the cost of building a gallery with its access and the resulting requirements on the construction management often mean that, for small dams, preference will be given to increasing the overall slope of the dam. In addition, in a narrow valley access to a gallery can be difficult from the downstream toe, and a check must also be made that the gallery will never be flooded.

As an indication, it can be considered that conventional concrete dams less than 15 metres high will not have galleries, while dams over 15 metres high from the foundation generally will have; that limit goes to 20 to 25 metres for RCC dams, as a gallery is a major construction imposition with the RCC technique that, if possible, will be eliminated.

For dams with no gallery, drainage can consist of a line of drillholes near the downstream toe and sloped towards the upstream (*see figure 1-b above*). This alternative improves the uplift situation under the downstream wedge of the dam. It is therefore only of interest when width at the base is less than 10 to 12 metres, which means height is less than 12 to 15 metres. In any case, drainage drillholes must remain accessible for cleaning or even re-boring.

The risk of clogging in foundation drains must systematically be taken into account as well as the risk that their outlet may be blocked by ice. These drains must also remain accessible for cleaning or re-boring.

Finally, internal drainage[1] in the dam body, which is practically systematically installed in large gravity dams of modern design, is generally not used at small dams. In fact, it can be accepted that the mass of concrete has significant tensile strength, which, for small dams, meets the conditions of internal stability without additional internal drainage.

Stability analysis

In stability analysis of a gravity dam, it must always be borne in mind that a large majority of gravity dam failures recorded around the world occurred during floods. This is easy to understand as the thrust of water varies as the square of the depth of water, so that any exceedance of the design flood level causes a decrease in dam stability, which is proportionally stronger for small dams. As an example, an extra depth of one metre at a dam 10 metres high means thrust increased by 21% and overturning moment increased by 33%.

The design flood and the level reached by the water must be precisely evaluated,

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1. At large dams, internal drainage consists of a line of subvertical holes drilled from the dam crest and leading into the drainage gallery set a few metres from the downstream toe.

and account will be taken of inaccuracy or uncertainties in hydrological data by examining the consequences of a significant exceedance of the selected design flood (*see Design flood and safety flood in chapter II, p. 25*), and see bibliography, reference 8.

Actions

We propose to classify the forces to be considered in computations in three groups:
- permanent actions;
- variable actions;
- accidental actions.

Permanent forces

Dead weight

The density of conventional vibrated concrete in a gravity dam is usually of the order of 2.4. Higher or lower values may be taken into account when the density of the aggregate differs significantly from 2.7. Around 2.4 the density of an RCC is variable according to aggregate grading and cementitious material content. The density of an RCC with a low fines content can go down to 2.3. For small dams, it is advisable to take into account the possibility of a gallery in calculating dead weight.

Pressure of sediment deposited at the upstream heel

Sediment that is consolidating exerts pressure that, at first approximation, is slightly angled to the horizontal. The earth pressure coefficient used for the sediments can be taken as:

$K_o = 1 - \sin \varphi$ (Jacky's formula)
φ : internal friction angle of the sediment.

The calculation should be done in effective stresses, meaning with buoyant weight for sediment[1], as the pressure of water is considered in the calculations over the entire height of the dam.

Variable actions

Thrust of water and suspended solids

This thrust is exerted perpendicularly to the surface of the upstream face. The density of water with suspended solids can routinely reach 1.05 to 1.10.

The water level to be taken into account is maximum water level during the design flood. That level must be precisely evaluated because stability of small dams is very sensitive to any rise in water level above the normal, as indicated above.

If necessary, account may be taken of the beneficial effect of pressure due to downstream water level. It is noteworthy that hydraulic flow conditions downstream from the dam often mean that this thrust rises faster than upstream pressure. So the worst case is not always that of the design flood. Intermediate levels must also be considered.

1. i.e with $\gamma' = \gamma - \gamma_w$

Finally, during a flood on a ski-jump-shaped free overflow block, the water will exert centrifugal force in the hollow of the ski jump and this beneficial force can be considered.

Uplift in the foundation

Uplift is generally calculated in situation of design flood. When no drainage is to be provided, the designer usually considers a trapezoidal diagram with full uplift (u_m) at the upstream heel and uplift (u_v) equal to the water level at the downstream toe (*figure 2-a*).

When there is no drainage, the uplift diagram may be less favourable than the trapezoidal diagram (a) in figure 2 if cracking in the rock has any tendency to close at the downstream toe. When the geological study gives reason to fear such a phenomenon, drains must be drilled at the downstream toe.

If drainage is provided, and assuming that the drains are maintained regularly, it is recommended to consider that drainage is 50% effective, which means uplift will be cut in half at the drainage curtain:

u_A-u_B = (u_A-u_C)/2 *(figure 2-b).*

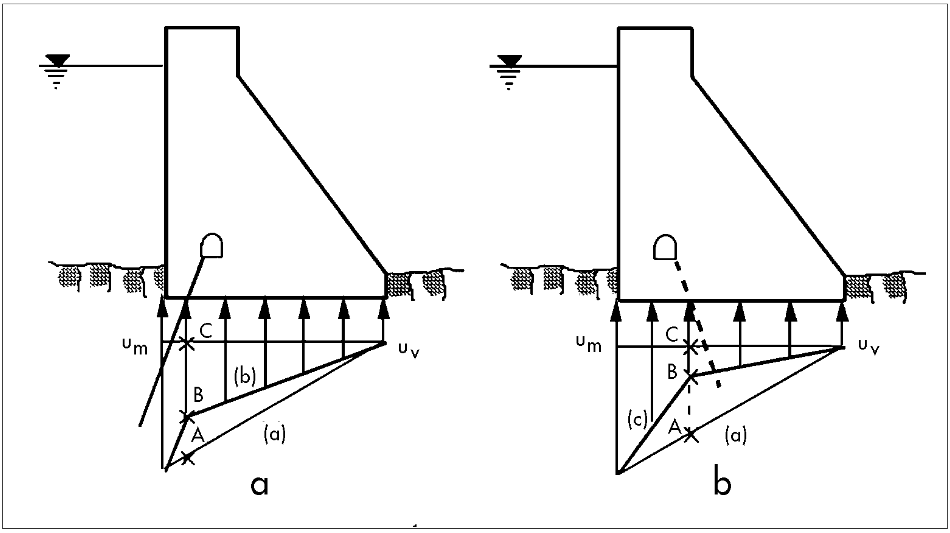

Fig. 2 - Indicative diagram of uplift:
(a) - with no grouting or drainage
(b) - with a grout curtain
(c) - with drainage

Similarly, if a grout curtain is placed in the foundation near the upstream heel, and provided that the upstream heel is not subject to tensile stress, it is considered that the effect of this curtain is to decrease uplift just downstream from it by one-third (versus the trapezoidal diagram with full uplift on the upstream side):

u_A-u_B = (u_A-u_C)/3 *(figure 2-a)*

Thrust of ice

This force need only be considered if the climate at the dam site warrants it. In general, it is not a determining factor for stability, as it does not occur at the same time as the design flood. To evaluate this force where necessary, the reader may usefully consult the volume *Techniques of Dam Construction in Rural Development* or the volume *Design of Small Dams*[1].

<u>Accidental actions: earthquake</u>[2]

For small gravity dams, the dam size generally will not warrant dynamic calculation, and earthquake forces are taken into account conventionally by what is known as the "pseudo-static analysis", which consists in modifying the vector of the gravity forces in the calculation of the dam's dead weight:

- the vector g has a horizontal intensity component of αg;
- at the same time, the pressure of water at depth z is increased by a value of ΔP for which WESTERGAARD proposes the following expression:

$\Delta P = 0.875\ \alpha\ \gamma_w \sqrt{Hz}$ (in kPa; with H and z in metres and γ_w in kN/m^3).
Where H is dam height.

In the case of a free overflow dam, total pressure of water is therefore increased by a value of:

$\Delta P = 0.58\ \alpha\ H^2$ (in kN for one metre of dam length)

In the same way as for fill dams (*see Pseudo-static analysis in chapter IV p. 88*), values of α can be found for each region of France in the AFPS[3] recommendations.

Unlike fill dams, the horizontal component due to earthquake is not given a reducing factor β. In fact, checking the stability of a gravity dam essentially consists in checking that there are no tensile forces, which could occur at the precise moment of the worst earthquake.

Combinations of actions

The design load effects result from combinations of the actions listed above and the worst case situations are considered versus the envisaged failure mechanism. This means that three types of combinations of forces can be distinguished:

- *frequent or quasi-permanent combination:* this is the state of forces corresponding to the normal service level of the dam. In general, it will be the combination of dead weight, thrust of deposited sediment, thrust of water at normal water level (NWL) and the corresponding uplift in the foundation;
- *rare combination:* this is the combination of actions during the design flood (maximum water level - MWL). The calculation considers dead weight, thrust of deposited sediment, thrust of water which may include suspended solids and the corresponding uplift in the foundation;

1. See *Bibliography*, reference 7, p. 139.
2. See *Bibliography*, references 3 and 4, p. 139.
3. See *Bibliography*, reference 3, p. 139.

- *accidental combination:* in general, this results from an earthquake occurring with the reservoir at normal water level (NWL).

In every case, different hypotheses must be emitted concerning the uplift diagram (which is the major unknown factor) and the sensitivity of results must be tested.

Stability analysis

Gravity dams (even the largest ones) are often analysed in two dimensions. A 3D analysis is justified when the dam is located in a relatively narrow valley and/or when the dam is curved in plan. The contribution to stability can in some cases be significant, even if it is always difficult to assess precisely.

For small gravity dams, a 2D analysis is suitable. In fact, when the valley is narrow, an arch dam design will probably be preferred. On the other hand, the stability analysis should not be limited to the highest dam block, but should also look at the stability of blocks under different conditions. This is especially true when some dam blocks are free overflow sections or include a gallery. The stability of both types of blocks should then be checked.

The methods used for small dams consist in considering a dam block as an undeformable block subjected to combinations of the forces described above. The analysis successively looks at stability against sliding, stability against overturning, and internal stability.

Stability against sliding

If N and T are the normal and tangential components of the resultant of the forces on the foundation, the most commonly used criterion is:

$$\frac{N.\tan\varphi}{T} \geqslant F$$

This means that foundation cohesion is neglected. The angle of friction φ between the dam and its foundation is generally taken as 45° for unweathered rock, but may have a much lower value in some cases (e.g. $\varphi = 25°$ for marl foundations). The factor of safety F should be greater than or equal to 1.5 for frequent or rare combinations and 1.3 for accidental (earthquake) combinations.

Normal stresses

Rather than stability against overturning (which will be preceded by local failure due to compression at the dam's downstream toe), checking normal stresses consists in verifying that the stress diagram at the base of the foundation remains within an acceptable range, both in terms of tensile stress at the upstream heel and compression at the downstream toe.

The NAVIER hypothesis of a trapezoidal distribution of stresses at the dam base is accepted; this hypothesis is related to the elastic behaviour of the concrete and the foundation, which is valid for small and medium sized dams.

The commonly accepted criterion of zero tensile stress at the dam's upstream heel is equivalent to the "central third rule", meaning the eccentricity e at the point where the resultant of the forces is applied should be less than B/6, where B is the width at the base of the dam. This criterion should be strictly met for frequent or practically permanent combinations of forces (at NWL). On the other hand, moderate tensile stress may be accepted at the upstream heel for rare or accidental combinations of forces (σ_t < 0.2 MPa for conventional concrete and σ_t < 0.05 MPa for RCC).

Internal stability

Stability in the top part of the dam is studied along a horizontal plane at a depth z under the reservoir water level. Maurice LEVY has proposed a criterion in which normal stress σ_v upstream, calculated with no consideration of uplift, is always greater than water pressure at the same level:

$$\sigma_v > \gamma_w z$$

In fact, this criterion seems very severe, and the quality of modern concrete makes it possible to reduce that requirement. The criterion most usually used is therefore:

$$\sigma_v > 0{,}75\, \gamma_w z$$

This criterion must be checked for rare combinations of forces (design flood).

Internal stability against sliding must also be checked, in particular when there may be a problem with strength between layers (which is the case of RCC).

Preliminary dimensioning of a small gravity dam

The stability criteria described above are usually met in the following cases:
- gravity dam with no gallery and overall batter (upstream + downstream) of the order of 0.85, provided that it is acceptable to consider an uplift diagram that is fairly close to the trapezoidal shape (*see Actions above p. 123*);
- gravity dam with a gallery and foundation drainage, and overall batter (upstream + downstream) of the order of 0.75.

These values must be increased in three cases:
- a free overflow dam with a high head on the sill during the design flood;
- a dam built in an area with average to strong seismic activity;

Construction techniques for conventional concrete dams

Joints

A conventional concrete gravity dam must be built with joints, dividing the dam into blocks, in order to absorb the effects due to hydraulic shrinkage of the concrete and annual temperature variations. From this standpoint, small dams present no special differences. Joints are generally spaced 15 to 20 metres apart in CVC dams, and 20 to 50 metres apart in RCC dams.

It is vital (and all the more so for small dams) to place a joint at each break in the foundation profile considered from one river bank to the other, which may mean placing joints closer together than the spacing recommended above. Each block must be as homogeneous as possible in terms of foundation level and cross-section.

Concrete

Traditionally, conventional concrete gravity dams are built with unreinforced concrete with about 250 kg of binder. For small dams, in order to simplify the discussion, standard concrete will be considered. When the water is aggressive, special cements must be used (rich slags or containing flyash).

It must be systematically checked that the aggregate is not sensitive to alkali-aggregate reaction (AAR), and how well it will withstand frost.

For small dams, the quantities of concrete to be used sometimes do not justify installation of a batching plant on the site. In this case, ready-mix concretes will be supplied from batching plants in the vicinity. Prudence is called for in using such concretes, as they include many admixtures, some of which may not be desirable in dam construction. In this case, strict specifications must be imposed on the suppliers as to the concrete mix, transport and placement times, and accepted or prohibited admixtures.

Finally, in the same way as for large dams, concreting in cold weather ($\theta < 0°C$) will be prohibited, and precautions must be taken between 0° and 5°C. If concreting takes place in dry hot weather, special attention must be paid to concrete curing, which should make use of water rather than admixtures.

Upstream-downstream tunnel

For RCC dams, construction of a gallery is always a constraint on the construction management. In cases where a gallery cannot be avoided, the dam designer should be attentive to group all of the conventional concrete structures (outlets, intakes, gallery) at the base of a single block, or even in one of the abutments, to enable construction to be scheduled in the least adverse way possible for efficient RCC placement.

Spillway

The most common design for conventional concrete or RCC gravity dams consists in building a surface spillway (gated or ungated), in the central part of the dam. In order to dissipate a good part of the energy, a stepped chute is built on the downstream face, with conventional concrete[1]. The sill usually has a standard ogee shape. Steps are built as high up the chute as possible, and their height increases to 0.60 to 0.90 metre in the central section of the chute.

The steps may be built *in situ*, possibly using the technique of slipformed concrete (used at Riou dam) or may be built of pre-cast elements for RCC dams. When specific discharge on the chute is high, the steps must be anchored in the dam body.

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1. See *Bibliography*, reference 5, p. 139.

MONITORING SYSTEMS

The general principles of monitoring are explained in chapter VII *(p. 162)*. Here, we deal only with specific features for concrete dams.

Monitoring systems for dams should be designed to follow important parameters for safety (or stability), as well as to monitor ageing. In particular, it must be checked that the hypotheses used in dam design are actually met. From this standpoint, gravity dam monitoring is oriented in the following directions:
- *monitoring water pressure* at the concrete-rock interface under a drained dam, using pore pressure cells or open-tube piezometers;
- *monitoring the effectiveness of foundation grouting,* by measuring drainage and leakage flows in the foundation;
- *possibly monitoring movements of the dam blocks* by means of topographic measurements, e.g. alignment and altimetry of benchmarks installed in the crest concrete, and differential displacement measurements between blocks, using vinchon gauges.

Furthermore, leakage at joints, whether vertical (between blocks) or horizontal (concrete lift joints), are measured to monitor trends and, where necessary, schedule repair works (obviously, leaks should never be plugged from the downstream end!).

At dams over 15 metres high, displacement measurements may also be made by direct pendula if the dam includes a gallery, or by inverted pendula if not. Displacement measurements on any axis that might be chosen (horizontal, vertical, sloped) can also be envisaged using elongation meters leading out into a gallery or at the downstream toe. Finally, in all cases, measurement of the reservoir water level is required, as it is the basis for management and monitoring of the dam. This will be done by a water level indicator (or water level recorder, if fine monitoring of reservoir management or flood analysis is also desired).

SOME EXAMPLES OF RECENTLY BUILT RCC DAMS

In France, no dams less than 15 metres high have been built with RCC or hardfill. The main reason for this is certainly the high fixed cost of installing a concrete production unit, which can only be cost-effective when a high volume of material is involved. However, we always recommend considering an RCC or hardfill alternative, even for the smallest dams, when the two following circumstances are combined:
- rock foundation at a relatively shallow depth;
- presence of a ready-mix concrete mixing plant near the site.

Two other circumstances further reinforce the advantages of this alternative:
- high flood flows to be discharged;
- very long dam body.

Here we give four examples of medium-size dams (21 to 25 metres) and one small dam (16 metres) that illustrate the various paths that may be followed. Design of small or medium height RCC dams is not fundamentally different, and these case studies could serve as examples for smaller dams with no major changes.

Riou dam (see photo 23 p. viii)

Riou dam (figure 3) was built by EDF; the dam is 21 metres high and involves 42 000 m^3 of RCC. It was extensively analysed within the framework of the French national research project on RCC entitled BaCaRa. The selected cross-section was a trapezoidal shape with a vertical upstream face, a downstream face sloped at 0.6H/1V and a crest width of 5.40 metres. The dam's watertightness was provided by a PVC geomembrane fastened on the upstream face with steel anchors, while the foundation was treated with a grout curtain. The dam body and its foundation were drained from a gallery in the lower part of the cross-section. The downstream face was free overflow for a width of 65 metres, and used as a stepped spillway chute.

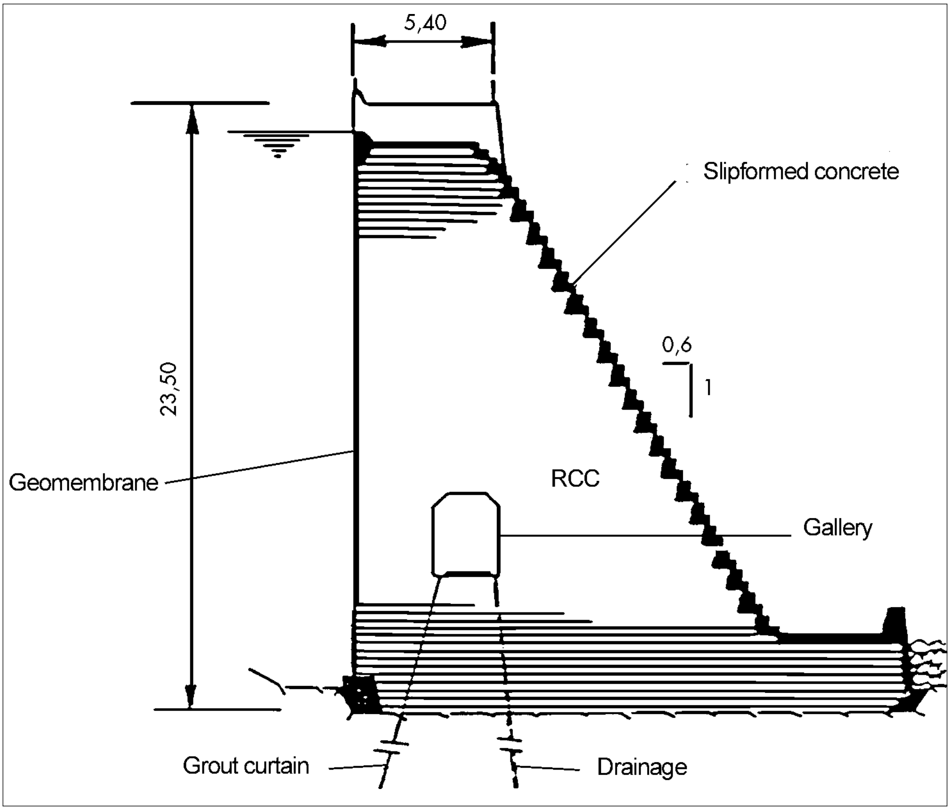

Fig. 3 - Typical cross-section of the Riou RCC dam - Faced with impervious geomembrane

The typical cross-section of this dam was noteworthy in comparison to larger structures because of the use of the impervious facing geomembrane and the absence of vertical contraction joints.

VILLAUMUR DAM

Villaumur dam (*figure 4*) is the smallest French dam built of RCC; it stands 16 metres high and involved 10 000 m^3 of RCC. The trapezoidal cross-section is thick, which meant no gallery had to be built. Here the dam was made watertight by a reinforced concrete face built slightly in advance of RCC placement. A geotextile connected to a series of PVC outlets served to drain the facing. The dam foundation interface is drained through a layer of porous RCC that comes out slightly below natural ground level in a wedge of rockfill.

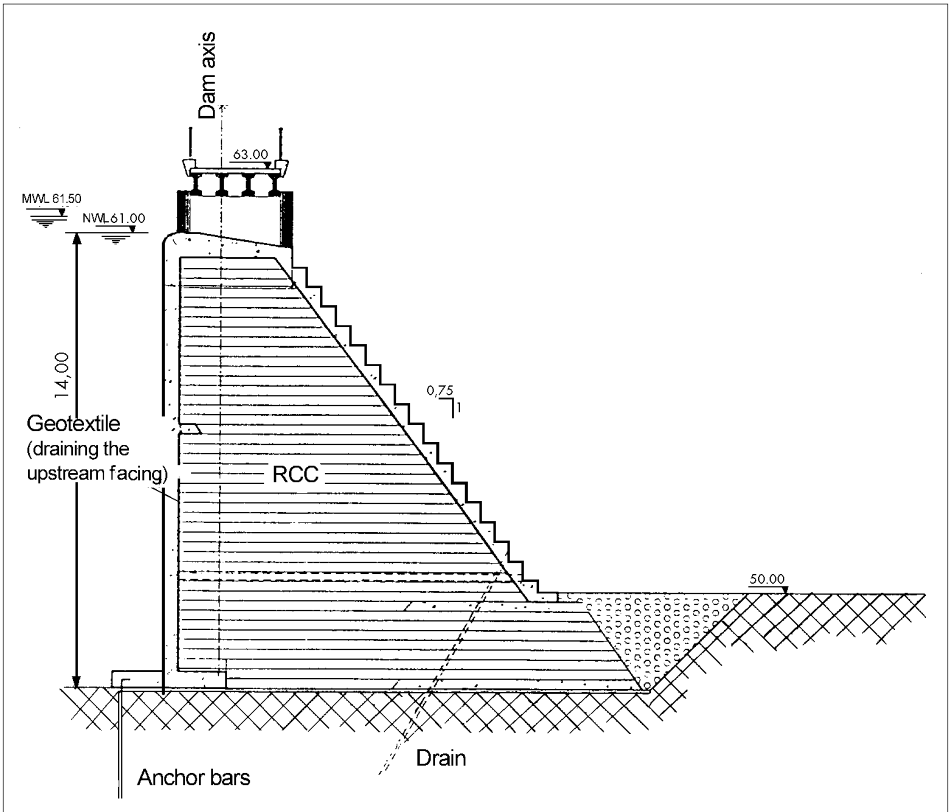

Fig. 4 - Typical cross-section of Villaumur RCC dam

El Koreima dam

El Koreima dam (*figure 5*) stands 26 metres high for an RCC volume of 25 000 m^3 and was built in Morocco near Rabat in 1989. It is a particularly instructive case, as the project was deliberately designed as a small dam. It was in fact designed and built by an administrative structure working with the resources of the armed forces within a program for construction of small dams. El Koreima was built with severely limited resources in terms of materials but abundant labour, following an approach derived from traditional techniques of masonry dam construction.

It is noteworthy that the cross-section shows a double slope (upstream 0.2H/1V, downstream 0.75H/1V for the non-overflow part, and 0.6/1V for the overflow part), with the downstream slope intended to help in placing the formwork for the dam face. Watertightness was provided by a reinforced concrete upstream face extended into the foundation by a cut-off trench.

Drainage for the dam body and the foundation was provided by vertical and horizontal drains leading out at the downstream toe, which meant no drainage gallery was required. An economic comparison with small masonry dams, conducted by the Hydraulics department, revealed a clear advantage for RCC (approximately 40% savings).

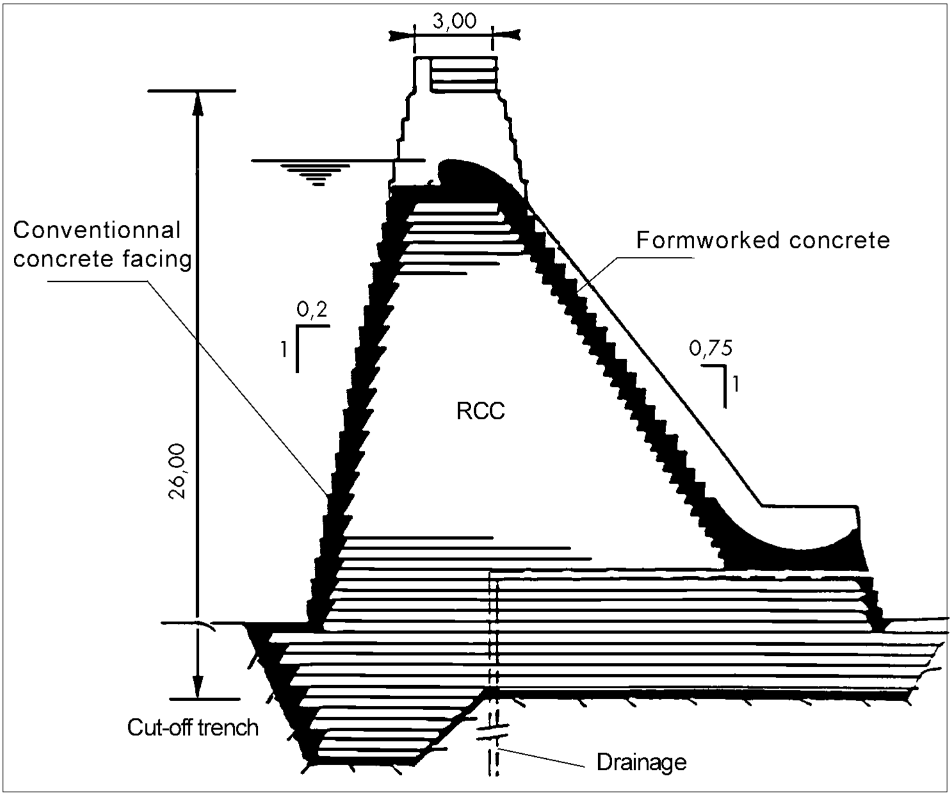

Fig. 5 - Typical cross-section of EL KOREIMA dam

Louberria dam

Louberria dam (*figure 6*), standing 25 metres high with 48 000 m^3 of RCC, is a flood routing dam with no permanent reservoir that is still in the project stage on the Nivelle river in the western Pyrenees.

The selected cross-section is trapezoidal (crest width 5 metres, upstream face 0.15H/1V and free overflow downstream face sloped at 0.6H/1V). The dam requires no major watertight structure, simply a zone of cement-rich RCC that is more impervious than the main dam body. No seepage control treatment is planned for the foundation and the dam is not provided with drainage.

This design takes advantage of the dam's use for flood routing only, in order to simplify the typical cross-section of the dam and optimise its cost.

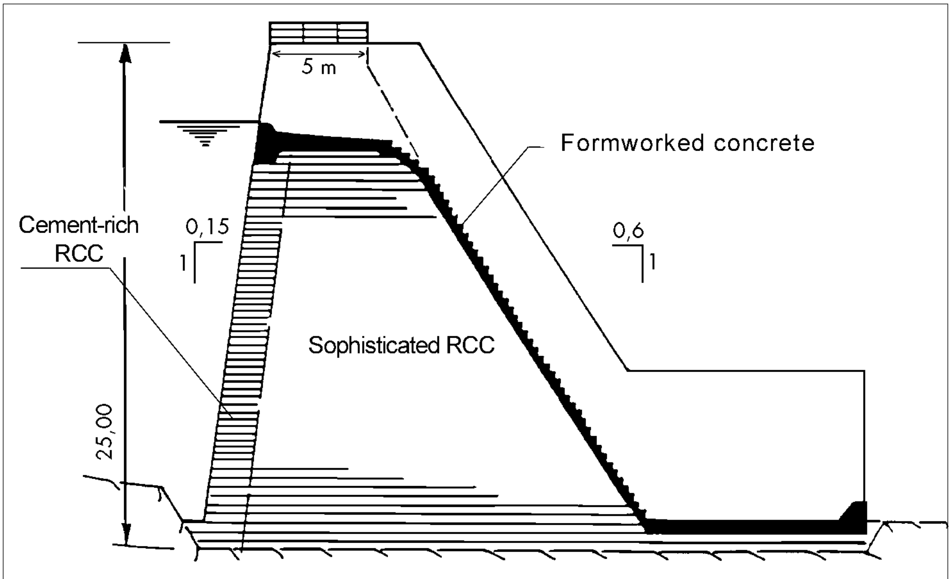

Fig. 6 - LOUBERIA dam

Mykonos I dam

Mykonos I dam *(figure 7, p. 134)*, 25 metres high and located in Greece, merits special attention as it is a symmetrical concrete-faced hardfill dam rather than an RCC dam proper. The originality of this design resides in the combination of a symmetrical shape (here the faces are sloped 0.5H/1V) and the use of hardfill, which is very unsophisticated RCC[1].

1. See *Bibliography*, reference 2, p. 139.

Watertightness is provided by a reinforced concrete face extended into the foundation by a grout curtain. At the facing/rock interface, there is an inspection gallery. For such a small dam, it would have been perfectly feasible to eliminate this gallery, possibly by building gentler slopes.

A symmetrical cross-section is well suited to sites with mediocre foundations as the forces transferred to the foundation are lower. It is of special interest in areas of high seismic activity as the dynamic stresses, in particular tensile stresses, are approximately one tenth of those with a conventional gravity dam design.

The search for an economical solution for the upstream watertight structure makes this kind of design attractive, even for small dams.

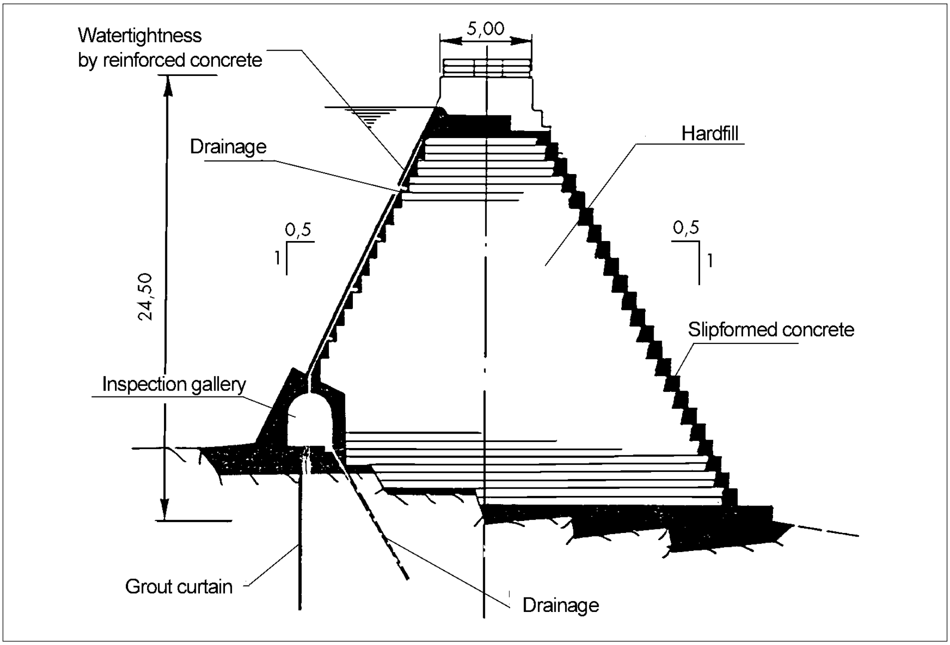

Fig. 7 - Typical cross-section of MYKONOS 1 dam with a symmetrical cross-section

Barrage type dams

Foreword

Barrage type dams take a special place among small dams. They are intended to create reservoirs of generally limited capacity and to regulate often high flows in rivers. They are required in many cases: hydroelectric power generation, navigation, or any other project requiring creation of a reservoir.

Some of these barrages, although limited in height, discharge high flows (exceeding 1000 m^3/s), which means they must be considered as large dams and suitable design rules must be applied; the details of their design are outside the scope of this volume.

Dimensioning principles

Once the reservoir water level has been set and the design flood determined, the necessary flow section must be dimensioned. The level of the invert is chosen in such a way as to avoid disturbing bedloads, which are often considerable in the rivers in question. In general, an elevation close to the river bottom is chosen.

Once the reservoir water level and the invert elevation are established, the barrage's length must be calculated. This can be done using the De Marchi empirical formula concerning flows over sills and narrow sections.

$$Q = \varphi \, L.h_v \sqrt{2g\,(H_a - h_v)}$$

where:

Q : flow in m^3/s

φ : contraction factor (around 0.9)

L : barrage opening in metres

h_v : height of water level above the sill 300 metres downstream from the barrage

H_a : head above the sill 100 metres upstream from the barrage.

Total opening is distributed among a certain number of sluices of a width calculated according to the type of gate used.

In modern barrages, the choice is essentially between tilting gates (*see figure 8 below, p. 137*) for relatively small closure surfaces of the order of 3 metres height for 20 metres width, or radial gates with arms articulated on the downstream side (*see figure 9 below, p. 137*) when the arms work in compression, or on the upstream side when the arms work in tension. This type of gate can be used to close off sluices up to 15 metres in height and 20 to 25 metres in width.

Then comes dimensioning of the concrete structure consisting of piers and their foundation slabs and of the invert. A barrage type structure functions like a gravity dam and the actions involved are:

- pressure of water on the gates transferred via the arms to the trunnions, as well as pressure on the cutwater;
- dead weight of the structure;
- uplift;
- ground reaction.

Using these various actions, stability is studied as explained previously under the heading *Stability Analysis (see p. 122)*. Foundation slab dimensions must enable a distribution of forces such that stresses are less than the ground's bearing capacity.

Very often, this type of dam is built in alluvial valleys, so a check must be made that there is no risk of flow around the structure or of piping.

This often means building a watertight curtain, which may consist in a diaphragm wall or grouting, or in a sheet-pile cut-off. This cut-off must also prevent any flow around the barrage on the sides.

In order to decrease uplift, a drainage blanket connected to the downstream water level is placed under the invert downstream from the grout curtain.

One last point that warrants special attention is downstream protection, which must prevent any risk of erosion. In fact, given the flows involved, the energy dissipated downstream can be very high, and account must be taken of situations where the gates are opened in unsymmetrical combinations.

For rivers with high sediment load, special measures must be taken to avoid erosion of the inverts and piers. Presently, resin-based coatings give good results at competitive costs.

The approach described above is a basis for design of a barrage-type structure. Many other points must also be examined, including the possibility of partly closing off the dam for maintenance of the gate, operating instructions, etc.

Construction techniques

Construction of this kind of dam can be envisaged in several ways:

- in successive stages with part of the river bed closed off in each stage;
- in a single stage behind a cofferdam, either by temporarily diverting the river or by building the structure outside the river bed and then "forcing" the river to flow through the barrage.

Inflatable barrages

Among small barrage-type structures, inflatable barrages can also be mentioned, another technique to create small reservoirs or raise dam sills[1].

Such barrages consist of a flexible membrane (reinforced elastomer) fastened onto a concrete beam and inflated either with water or with air. Their height varies in general from 1.5 to 3 metres and rarely exceeds 5 metres. Length may be up to 100 metres.

The principle of how such a water inflated sill functions is shown in *figure 10 (p. 138)*. The casing is connected to a well supplied with water to create a load Q 30% to 50% higher than the head P corresponding to the reservoir. If the water level increases upstream, the increase in pressure P forces the water out of the well and the membrane deflates. In this way, the barrage lowers automatically during floods. It is returned to its original level by pumping, started up either manually or automatically by means of a water level detector.

1. See *Bibliography*, reference 6, p. 139.

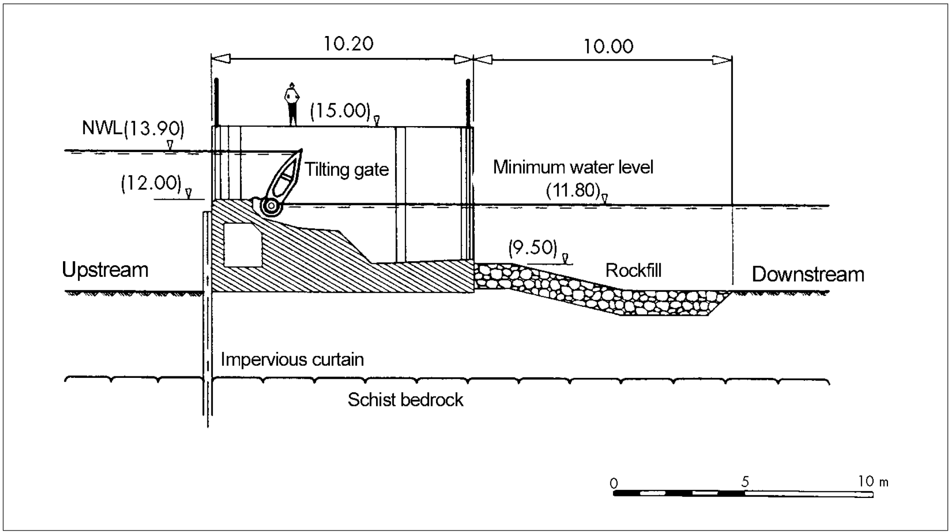

Fig. 8 - Dam with a tilting gate.

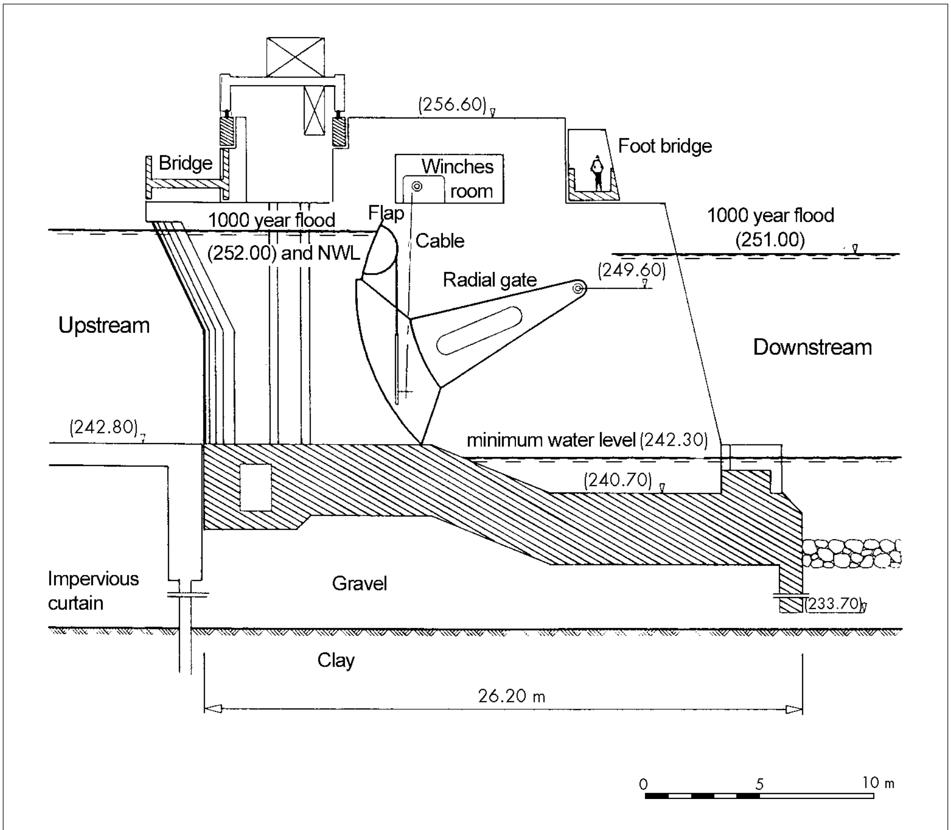

Fig. 9 - Dam with a radial gate.

For an air-inflated barrage (which will be more sensitive to oscillations), a compressed air unit is required.

These structures have good resistance to impacts from floating debris or to bedload transport. They can suffer from vandalism but this does not jeopardise safety in flood periods.

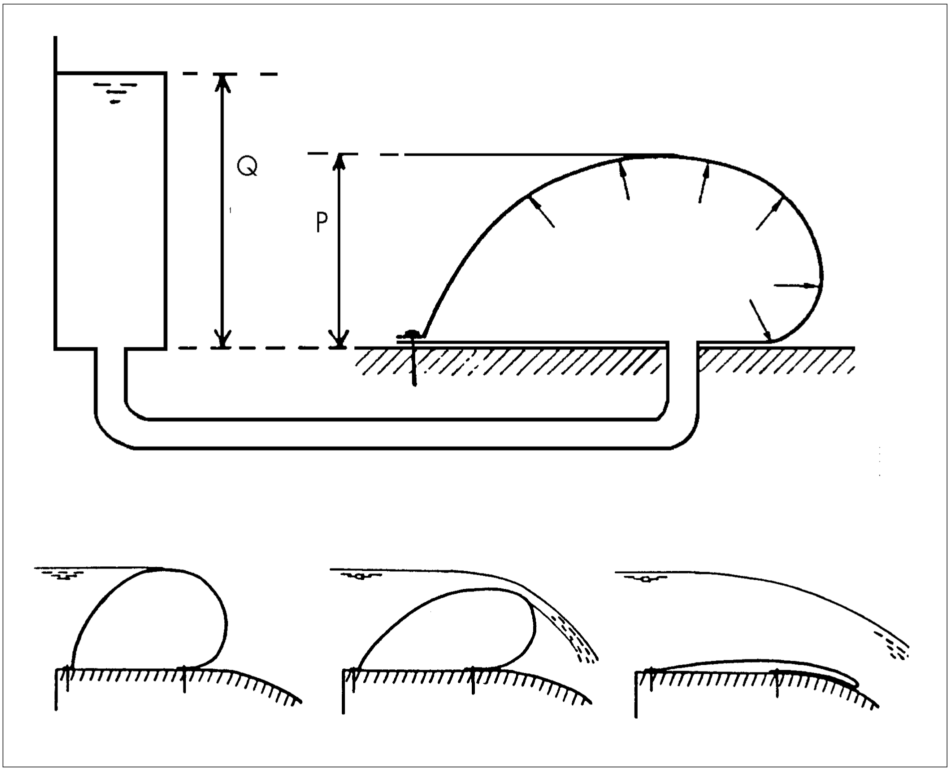

Fig. 10 - Schematic principle of a water inflated barrage.

Tendering and trial embankments

The section concerning *Tendering* in Chapter 4 *(p. 99)* is applicable to concrete dams.

Similarly, the section concerning *trial embankments* in that same chapter *(p. 102)* is also applicable with no major changes to RCC dams.

The reader is therefore invited to consult these sections.

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