French Committee on Large Dams
Comité Français des Grands Barrages

SMALL DAMS

Guidelines for Design, Construction and Monitoring

Coordination : Gérard Degoutte

Cemagref EDITIONS

FRENCH COMMITTEE ON LARGE DAMS
COMITÉ FRANÇAIS DES GRANDS BARRAGES

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Coordination : Gérard DEGOUTTE (ENGREF)

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Preface

These guidelines for the design and construction of small dams have been drafted under the aegis of the French Committee on Large Dams. Small dams are conventionally defined as dams less than approximately 25 metres high.

The designer of a small dam may face a difficult task because he will not have access to the design and monitoring resources commonly deployed for larger dams.

The recommendations and advice in these guidelines have been drawn up on the basis of the climate, population and economic conditions found in France. They may require some modification if used in a different context.

For example, safety criteria may be relaxed if the valley downstream of the dam is sparsely populated; greater caution may be required in the absence of abundant rainfall and streamflow records; and some of the arrangements recommended may be considered needlessly costly luxuries.

Nevertheless, the guidelines provide a consistent foundation on which country-specific alternatives can be built up. It is thus of undeniable general interest, and the French Committee has decided to support an English language version.

† André GOUBET

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FOREWORD

What need for guidelines for small dams?

by Gérard DEGOUTTE, ENGREF,
Chairman, Working group

The French Committee on Large Dams decided to draft guidelines for small dams in consideration of the fact that the available professional literature focused primarily on large dams. The Guidelines are therefore a summary of past experience and French research in the form of recommendations for the design, construction and surveillance of dams between approximately 5 m and 25 m high in mainland France.

Nevertheless, they are also applicable to other parts of the world.

WHAT DAMS?

Dams built in France in this height range are:
- earthfill dams, the most common type, and
- concrete or RCC gravity dams[1].

Arch and rockfill dams are dealt with briefly because there are few examples less than 25m high. Masonry dams are no longer built in France and buttress dams and multiple arch dams are no longer cost-effective for these heights, and are ignored.

1. *RCC :* roller compacted concrete. See chapter V "small concrete dams", p. 113...

Features specific to small dams

In principle, small dams present as many difficulties as large dams. Of course, low height means low stresses and the response of the foundation rock will remain in the elastic range; flow velocities at the downstream end of a spillway chute will not be such as to cause cavitation. But on the other hand, it will not always be reasonable in cost terms to try to have a complete understanding of the watertightness of the reservoir area; the absence of any stream gauging stations will boost the uncertainty on flood flows... Uncertainty and grey areas will therefore complicate the designer's task, and he will need to design a dam that allows for these uncertainties. If he cannot, he will be faced with the dilemma of declaring the site to be unsuitable, or taking risks. Risk of dam failure, but more often, risk of excessive leakage.

The many cases of deterioration reported in the past lead to the site being declared definitely unfeasible in this case, even if this means rejecting it unnecessarily... but who will ever know?

What is meant by "small dam"?

There is no universally accepted meaning of the term "small dam". In France, a "large dam" is frequently considered as being more than 20 metres high[1], because since 1966, they must be submitted to the Permanent Technical Committee on Dams (CTPB); yet the relevant regulations do not use the term large dams.

The International Commission on Large Dams defines large dams as being more than 15 metres high or in some cases between 10m and 15m high[2].

In fact, there is no need for any precise demarcation between small and large dams. A 20m-high dam will be the largest of the small dams category for some, while others see it as the smallest large dam! We would suggest, to provide a general guide, calling dams less than 15 metres high *small* dams and those 15-25m high, *medium-sized* dams. This is not intended as a universal definition and has no legal significance. The Guidelines are only recommendations, not regulations applying with any force of compulsion to any category of dam or any category of dam owner.

1. Height measured from lowest point of natural ground level, including for depth of main stream channel, if any.
2. Height measured from the lowest point of the foundation.

Dam size and safety

Small ,or medium-sized dam is not synonymous with risk-free dam. With a reservoir of more than 100,000 m^3 capacity for example, there is always the possibility of risk for anyone standing at the dam toe at the wrong time. So there can be no short-cuts on safety. Anyone who does not believe this might usefully reflect on the fact that there are some 10,000 small and medium-sized dams in France.

Recommendations must of course match the risk involved. A 10m-high dam impounding 15 million cubic metres of water bears no comparison with a similar dam impounding 50,000 m^3.

These two factors - height and capacity - are conventionally combined by using the parameter $H^2\sqrt{V}$ in which H is dam height in metres from the lowest point of the natural ground level and V is the reservoir capacity to Full Supply Level in million cubic metres (hm^3). The graph *(figure 1)* shows several values of $H^2\sqrt{V}$ between 5 and 3000. The area bordered by the dashed lines contains all the French dams (ignoring river embankments).

Readers are invited to plot the few dams they are most familiar with on this graph!

Parameter $H^2\sqrt{V}$ has no particular scientific significance. It should simply be considered as an indicator of potential risk downstream of the dam. For example, it correlates well with the peak downstream flood wave in the event of complete breaching of the dam.

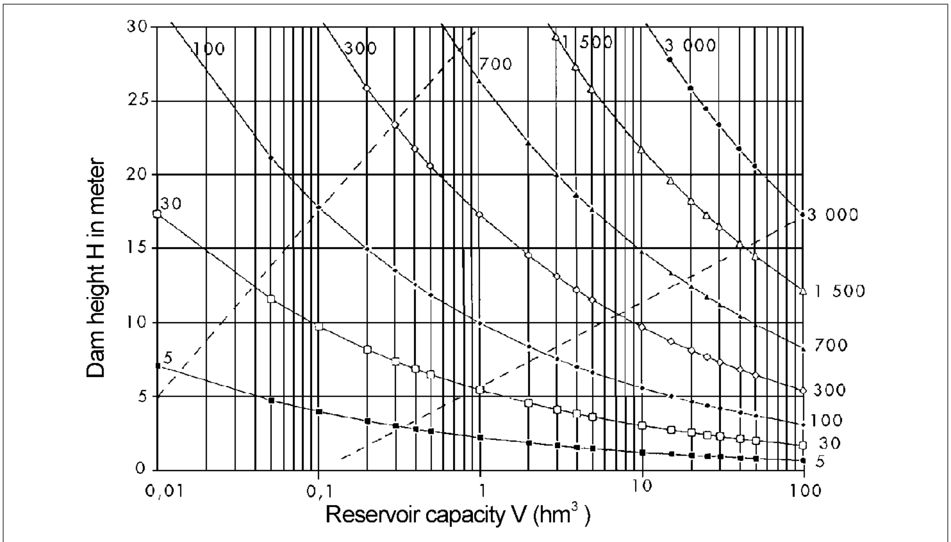

Fig. 1 - Height I Reservoir Capacity Relationship with values of $H^2\sqrt{V}$ (V in hm^3, H in metres)

These Guidelines therefore recommend minimum requirements based on this parameter. More stringent requirements may be dictated by engineering criteria if required by special circumstances.

Aspects discussed

The Guidelines cover the preliminary geological and geotechnical surveys *(see chap. III, p. 37)* and hydrological studies for flood estimation *(see chap. II, p. 23)*. It next deals with the design and construction of the two main types of dam: embankment dams *(see chap. IV, p. 67)* and concrete dams *(see chap. V, p. 113)*.

Environmental aspects are increasingly important. Chapter VI deals with the fundamental issue of water quality in the reservoir and downstream of the dam. While we have been building dams for thousands of years, their impact on water quality has only been measured at best for the last few decades. This chapter is therefore more of a review of our current state of knowledge rather than a set of precise recommendations.

Although some owners now attach more importance to the appearance of their dams, this is not discussed here, because there is no generally accepted doctrine to our knowledge. We simply advise engineers and contractors to give more consideration to this aspect in terms of the finish on exposed surfaces and blending concrete dams and appurtenant buildings into the scenery.

Lastly, it should be remembered that a dam is not finished when the construction contractor quits the site or even when the spillway operates for the first time. A dam is a living thing, and ages. It must be carefully watched and monitored, as described in Chapter VII.

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CHAPTER I

Choice of site and type of dam

By Jean-Pierre BECUE (SAFEGE)
with the participation of Gérard DEGOUTTE (ENGREF)
and Danièle LAUTRIN (Cemagref)

Dam types can be classified in different categories according to the material used in construction and how they withstand the thrust of water:

- *homogeneous drained earthfill dams, either zoned or with a man-made impervious element;*
- *gravity dams, whether concrete or RCC;*
- *arch dams;*
- *and buttress or multiple arch dams (not dealt with here).*

Fill dams are flexible structures while the other types are rigid.

The main parameters to be taken into account in choosing a dam site and type are the following:

- *topography and inflow in the catchment area;*
- *morphology of the river valley;*
- *geological and geotechnical conditions;*
- *climate and flood regime.*

In many cases, after consideration of all these aspects, several types of dams will remain potential candidates. Economic considerations are then applied to rank the available alternatives.

Topography and inflow in the catchment area

If we ignore the case of lakes for recreational purposes and small dams for hydroelectric power generation, reservoir storage is the main factor influencing the entire dam design. The objective is in fact to have a volume of water available for increasing dry weather river flow, irrigation or drinking-water supply, or free storage capacity to attenuate flooding.

The first task therefore consists in calculating the volume of water that can be stored in a basin, possibly at several different sites. A first approximation can be achieved using a 1/25 000 scale map with contour lines every 5 or 10 metres, except for reservoirs with storage of several tens of thousand cubic metres. The second task will then be to check whether conditions in the catchment area are such that the reservoir will be filled and to calculate the risk of shortfall.

Morphology of the river valley

A dam is by nature linked to an environment. The morphology of the river valley therefore plays a vital role in the choice of a dam site and the most suitable type of dam.

Of course, the ideal and most economical location will be a narrow site where the valley widens upstream of the future dam, provided that the dam abutments are sound (i.e. a narrowing with no zones prone to rockfall or landslide).
Such a site is rarely found, either because the natural structure of a valley does not offer any point of narrowing or because the choice of the site is not solely dependent on engineering considerations.

As a first approach, a wide valley will be more suitable for construction of a fill dam. A narrow site will be suitable for a gravity dam as well, and a very narrow site will be suitable for an arch. In every case, of course, provided that the foundation is acceptable.

Geology and foundation conditions

The nature, strength, thickness, dip, jointing and permeability of the geological foundations at the site are a set of often decisive factors in selection of the dam type.

Rock foundations

Except for severely jointed rock or rock with very mediocre characteristics, rock foundations are suitable for construction of any type of dam, provided that suitable measures are taken to strip off severely weathered materials and, if necessary, treat the foundation by grouting. Fill dams will always be suitable. For the other types, requirements are more severe for RCC, still more for conventional concrete, and finally most stringent for arch dams. The most important aspect is cracking (faults, joints, schistosity).

Gravel foundations

Provided that they are sufficiently compacted, gravel foundations are generally suitable for earth or rockfill dams, at least in terms of mechanical strength. Leakage must be controlled by suitable impervious barriers and drainage systems. In practice however, this type of foundation essentially is found on rivers with high flows. The dam must therefore be able to discharge high floods, which precludes earthfill dams. Very small concrete dams may also be suitable provided precautions are taken with leaks and seepage (risk of piping) and with differential settlement.

Sandy-silt foundations

Silt or fine sand foundations can be suitable for construction of earthfill dams, and even, in exceptional cases, for very small concrete gravity dams provided strict precautions are taken.

Clay foundations

Clay foundations almost automatically impose the choice of a fill dam with slopes that are compatible with the mechanical characteristics of the geological formations.

Available materials

Availability, on the site or near it, of suitable materials to build the dam has a considerable influence and one that is often decisive in choosing the type of dam:

- soil that can be used for earthfill,
- rock for rockfill or slope protection (rip-rap),
- concrete aggregate (alluvial or crushed materials),
- cementitious materials (cement, flyash, etc.).

If it is possible to extract the materials from the reservoir itself, reservoir storage can be increased. This also usually keeps the cost of transport and restoring borrow areas to a minimum.

As a general rule, if silty or clay soil of satisfactory quality (fines content, plasticity, condition) and quantity (1.5 times or twice the volume of fill required) is available, a dam construction alternative using homogeneous earthfill or quarry-sorted materials - setting aside the coarsest[1] materials for the downstream shoulder - will be the most economical provided that the flood flows to be discharged are moderate.

If only a limited quantity of impermeable materials, and coarse or rockfill materials as well, is available, the possibility of a zoned earthfill dam or a rockfill dam with a watertight core can be considered. The disadvantage of this alternative is placement in zones, which is all the more complicated when the site is narrow, hindering movement of the machinery.

If only coarse materials are available, they can be used to build a homogeneous embankment with a watertight diaphragm wall built in the centre of the dam, by grouting after the fill has been placed or by an upstream watertight structure (concrete or bituminous concrete facing).

If only rockfill is available, a compacted rockfill dam with external watertight structure (geomembrane, hydraulic concrete or bituminous concrete facing) on the upstream face, will be suitable. A concrete alternative, especially RCC, can also prove to be competitive provided the foundation is good enough (rock or compact ground) with no need for excessive excavation.

1. This term should be taken to mean not only coarse but also lacking the necessary fine particles to be watertight.

Floods and flood discharge structures

The cost of flood discharge structures depends on the hydrological characteristics of the catchment area.

When the catchment area is large and floods are likely to be high, it may be advantageous to combine the dam and spillway functions and build an overspill dam.

On the other hand, if the spillway can be kept small, a fill dam will be preferred if all other conditions are equal.

When construction of the spillway would require significant excavation, the possibility of using the excavated materials will also be a factor in favour of building a fill dam.

When a tunnel is required for temporary diversion of the river during the work, it can usefully be incorporated into the flood discharge structures, if necessary increasing its cross-section slightly *(see Chapter IV, p. 67)*.

The choice of an RCC dam can be attractive if it is a means of shortening construction lead time and removing the risk of damage from flooding of the site before construction is complete[1], a risk that, with any other alternative, would mean building costly diversion or protection structures.

Economic criteria

In many cases, the considerations set out above will be sufficient to select several types of dam as potential alternatives. For example, if the foundation is rock, loose materials are available near the site and flood flows are high, the choice will be between an RCC dam and an earthfill dam with a costly spillway.

The studies must then be pursued for these two types of dam, taking care to refine the cost estimates as the studies progress. As soon as one of the dam types seems significantly more economical, it is preferable to waste no further time on the other option.

1. Because the dam can be completed in one dry season.

Conclusions on selecting a type of dam

The choice of a type of dam is imposed by natural conditions in many cases, with no need for in-depth investigations. For example, if the rock substratum is at a depth of more than 5 metres, the only reasonable alternative will be a fill dam, at least for any project less than 25 metres high. In some regions, the geological context is such that only one type of dam is usually built.

In other cases, the choice of dam type will be a compromise between different aspects - type of foundation, availability of materials in the vicinity, hydrology - to arrive at the best option economically speaking.

However, it is always an advantage to make a decision as quickly as possible, as a rule after the feasibility studies.

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CHAPTER II

Preliminary determination of design flood

Jacques LAVABRE, Cemagref

At first consideration, methods to calculate design floods do not change according to the size of the project or the size of the catchment area, at least for catchment areas smaller than several hundred square kilometres.

But in practice there are significant differences when it comes to small and medium sized dams:

- *the first concerns the recurrence interval of the flood event the designer is seeking protection from; in fact, fairly short recurrence intervals may be accepted for very small dams where dam failure would have practically unnoticeable consequences downstream; conversely for dams representing a risk for populations downstream, long recurrence intervals must be considered;*
- *the second difference stems from the fact that small and medium sized structures are generally located in small catchment areas which rarely have gauging stations. In this case the quality of the available hydroclimatological information is less good. Greater imprecision in the results of the hydrological study must be taken into account when the type of spillway and its dimensions are chosen.*

PRELIMINARIES

Today it is clearly accepted that the flood study, which serves in dimensioning spillway and discharge structures, cannot rely simply on flow observations. The methods used are of the hydro-meteorological type and combine rainfall data with flow data. Those methods may rely simply on statistical concepts (GRADEX method or the AGREGEE[1] model) or propose a deterministic approach to the transformation of rainfall into flows.

In many countries (in particular in Anglo-Saxon countries), the *Probable Maximum Precipitation - Probable Maximum Flood* (PMP - PMF[2]) method, developed by North-American engineering, is commonly used. This method defines a probable maximum flow for the catchment area studied which is the highest flood that can reasonably be imagined. The risk of such flow occurring is in principle infinitely low, and in any case cannot be quantified.

In other countries, like France, a design flood is computed along with a corresponding risk of occurrence. Depending on the hazard this represents for the downstream area - human fatalities, economic considerations, whether or not the dam is overtoppable, etc. - the selected recurrence interval may be of the order of 10^{-2} to 10^{-4}. In terms of frequency, this would mean for example that a dam designed to handle a flood with a frequency of occurrence of 10^{-3} has a risk of $1 - (0.999)^{100} = 9.5\%$ of suffering the design flood during 100 years in operation. The risk is of the order of 1% if the dam is likely to have to discharge the 1000-year flood (in 100 years operation). The designer must therefore be aware that the dam runs a non negligible risk of being confronted with a design flood determined this way, while bearing in mind that it could well withstand a higher flood thanks to *freeboard.*

Although at first consideration all of these methods are only applicable for sites where gauging records are available, practical considerations sometimes result in applying them in downgraded mode. This is often the case for small catchment areas where flow records are rarely available. The reliability of the hydrological study is however always highly dependent on the quality of the available hydroclimatological information.

1. Defined below, pp. 26-29.
2. *PMP - PMF :* Probable Maximum Precipitation - Probable Maximum Flood.

Design Flood and Safety Flood

The *design flood* is the flood with the longest recurrence interval considered in the reservoir. It is taken into account to determine the *Maximum Water Level* (MWL) and dimension the spillway, incorporating possibilities of flood routing. Often, the considered design flood is the flood with the highest peak flow. It is not always certain that this flood is the worst case in calculation of the spillway. A flood with a lower peak flow but lasting a longer period could have worse consequences. The minimum recurrence interval recommended for such a flood is between 100 and 10 000 years (10^{-2} to 10^{-4}). The choice of a recurrence interval depends on the risk involved in dam failure. The *dam's intrinsic risk* can be quantified by means of the parameter $H^2 \sqrt{V}$. Table 1 sets out recommendations for the choice of a design flood relative to this criterion. However, *global risk* is also related to the *vulnerability* of the downstream area (population density in the zone likely to be flooded in the event of a failure). The recommendations in table 1 must be beefed up in case of serious vulnerability (for example by going from a 500-year flood to a 1000-year flood). When the dam is of public safety interest, the recurrence interval should never be less than 1000 years, whatever the value of $H^2 \sqrt{V}$.

Once *MWL* has been calculated, the dam crest is set at a higher level. The difference between these two levels is called *freeboard*. Freeboard is essentially intended to avoid overtopping due to wave action but also plays an essential role in safety from flooding. A method for calculating it is given in Chapter IV, p. 73.

Thanks to freeboard, a dam should be able to withstand a flood (which is known as safety flood) higher than the design flood. This is by definition the worst case flood that could occur at the dam without any risk of failure. In the case of an ungated spillway at an embankment dam, the safety flood would be any flood that would cause overspillage, provided that it did not cause overtopping at any point in the chute that would jeopardise the fill itself. For a gravity dam, the safety flood would also correspond to the crest of the non-overflow part. For a dam with an impermeable core, the safety flood would be reached when the reservoir water level reaches not the dam crest but the crest of the core.

$H^2 \sqrt{V}$	< 5	5 to 30	30 to 100	100 to 700	> 700
Recurrence interval in years	100	500	1 000	5 000	10 000

Table 1 - Minimum recurrence interval of the design flood for an earthfill dam without consideration of vulnerability downstream (H: dam height in metres; V: reservoir volume in hm^3)

THE GRADEX[1] METHOD

This statistical method, developed by Electricité de France (EDF), is the standard used in France. Its success is in particular due to its (relative) simplicity of use, which results from an extreme simplification of the process of transforming rainfall into flow.

ASSUMPTIONS OF THE GRADEX METHOD

Rainfall is considered globally over a certain period of time, equal to the average duration of the hydrographs.

The probability of precipitation events lasting various durations is a simple exponential decay function. The main parameter is proportional to the standard deviation of maximum precipitation values. It is called the exponential gradient, GRADEX. GUMBEL's law is often applied. Its distribution function is as follows:

$F(P) = EXP\ (-EXP\ (-(P-Po)/a)$

The GRADEX (a) may be obtained using the event method. In this case, it is equal to 0.78 times the standard deviation. a is of course a function of the duration of the precipitation considered.

Remarks :

- When $P \rightarrow \infty$, $F(P) \rightarrow 1 - EXP\ (-(P-Po)/a)$ and the Napierian logarithm at recurrence interval $T = (1/(1 - F(P))$ is equal to $(P - Po)\ /a$. The rainfall depth varies linearly with the logarithm of the recurrence interval, the slope *(a)* of this straight line being equal to the GRADEX..
- If $P_{1\,000}$ and P_{100} indicate respectively the rainfall depth at recurrence intervals of 1 000 and 100 years, then:
  $P_{1\,000} - P_{100} = a\ (\ln 1\,000 - \ln 100) = 2{,}3\ a$
  (ln designating the Napierian (or natural) logarithm).

When the catchment area reaches a certain saturation level, any increase in rainfall generates an equal volume of runoff for the same lapse of time. As a first approximation, this state is reached for recurrence intervals of ten years (impermeable catchments, with low retention), to 50 years (permeable catchments, with high retention).

The runoff law is obtained quite simply by translating the rainfall depth law to the point of the 10 or 50 year recurrence interval.

A *physical* interpretation of this process can result from observation of the graph of runoff variations relative to the amount of rainfall *(see fig. 1, p. 35)*. The retention capacity of the catchment area is schematically represented by the difference between the bisector (rainfall = runoff) and its parallel, plotted in the middle of a cloud of points.

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1. See *Bibliography*, references 5, 7 and 11, p. 36.

Figure 2 *(p. 35)* illustrates this principle: the adjustment of the rainfall values has the GRADEX as slope. In this application, the recurrence interval retained for the hypothesis concerning saturation of the catchment area is 20 years (this corresponds to a reduced GUMBEL variable equal to 2.97). For volumes exceeding the runoff corresponding to this recurrence interval, adjustment is made by plotting a straight line with a slope equal to the GRADEX.

In this example the catchment area is instrumented and it is therefore possible to make a statistical adjustment of runoff, up to the 20 year recurrence interval.

In the case of small catchment areas without flow records, this is not possible. A regional approach based on nearby, if possible similar, catchment areas is necessary. It is however possible to consult national analyses such as SOCOSE or CRUPEDIX[1]. These methods essentially require rainfall data and give an order of magnitude for peak flow at recurrence intervals of 10 years (10 and 20 years for SOCOSE). It appears that even a sizeable error on a 10-year (or 20-year) flood has a relatively weak influence on the 1000-year flood or the 10 000-year flood calculated with the GRADEX method.

It is noteworthy that for small catchment areas with no gauging stations, the evaluation of rare flood flows is almost exclusively based on rainfall information. Luckily this information is generally available for most of France.

A simple ratio of affinity is used to go from runoff in the considered time-frame to peak flow. This ratio is estimated from hydrographs; its average value is used (laws governing the probabilities of ratios and average runoff may also be combined, resulting in ratios that increase with the recurrence interval). For catchment areas with no water level gauging stations, we may use a ratio determined for similar catchment areas.

An example of this application is given farther along in this chapter *(p. 31)*.

Difficulties in applying the Gradex method

Strictly exponential decrease in precipitation with recurrence intervals leads to assigning extremely high recurrence intervals to certain observed events at a random point in France. It is true that total rainfall over 500 mm in 24 hours is not really exceptional in certain areas of France, but it is limited to certain regions: around 1000 mm in the Canigou region of the eastern Pyrenees, in October 1940; 800 mm in the Solenzara region of Corsica, in October 1993, etc. The standard design flood used in dimensioning a dam is therefore not the maximum flood that could occur.

There is no well-defined rule to calculate the time during which the hypothesis of equal increase in rainfall and in runoff is applied. Only a detailed coupled analysis of rainfall and floods will result in an estimate that is not too risky. In the absence of

1. See Bibliography, reference 1, p. 36. The SOCOSE method is derived from the work of the American Soil Conservation Service.

precise data, the formulation of the characteristic time of the catchment area, developed in the SOCOSE[1] method, may be used. In this method, the characteristic time is defined as the period during which the runoff is more than half of peak runoff. If no data is available on runoff on the site, the following regionalised formula may be used:

$$\text{Log } D = -0{,}69 + 0{,}32 \text{ Log } S + 2{,}2\,(Pa/Pta)^{0{,}5}$$

D : characteristic time (hours)
S : surface area of the catchment area (sq. km)
Pa : average annual rainfall (mm)
P : daily rainfall with a 10-year recurrence interval (mm)
ta : average yearly temperature (°C)

Note:
This method is frequently used at a daily time-step when the catchment area is of a certain size, by virtue of a greater availability of daily information on rainfall and runoff.

The sudden rupture that affects the runoff equation at the pivot point (start of the rainfall equation) leads to an over-estimation of flow with intermediate recurrence intervals (50 to 500 years).

The affinity ratio to obtain the peak runoff is extremely variable. The method recommends keeping its average value. If we have properly chosen the duration in which the increase in runoff is equal to the increase in rainfall, it should be of the order of 1.5 to 2.0.

This method does not give a design hydrograph in a form suitable for simulation of flood attenuation. A bi-triangular shape which respects the duration, the peak runoff, and the volume of runoff can be used. In general, these design hydrographs result in over-estimations of attenuation capacity, as they represent only a part of the flood. One must often take into account the base flow in the river before the flood, when it represents a non-negligible proportion of the flood flow.

The AGREGEE model[2]

A recent development by Cemagref[3], this model is an extension to the GRADEX method. Re-using its statistical concepts and the hypothesis that when the catchment area is saturated, any increase in rainfall generates an equal increase in runoff. The modifications are based on:
- combining the laws of probability of rainfall and runoff to progressively go from the runoff equation to the rainfall equation;

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1. See *Bibliography*, references 1 and 12, p. 36.
2. See *Bibliography,* reference 9, p. 36.
3. Cemagref : The french agricultural and environmental research institute – http://www.cemagref.fr

- taking into account the statistical distribution of the affinity ratio (passage from average runoff to peak discharge);
- the probability approach to instantaneous flow in order to obtain an overall design hydrograph.

This model makes no hypothesis on the equation of rainfall probability. The simple exponential decay function for rainfall relative to the recurrence interval is not imposed. Thanks to the progressive passage from the flow equation to the rainfall equation, the model avoids the over-estimation of the discharges of intermediate recurrence intervals (50 to 500 years). Although not very realistic, the single-frequency hydrographs obtained are easy to use in calculation of flood attenuation.

The AGREGEE model and the GRADEX method give similar results in estimation of extreme floods (1000 years to 10 000 years).

The PMP - PMF method[1]

This method is very rarely used in France. It is based on knowledge of the *Probable Maximum Precipitation (PMP)* in the catchment area and a rainfall - runoff model to calculate *Probable Maximum Flood (PMF)*. It results in a design hydrograph.

The *PMP* is defined as the highest theoretical precipitation that is physically probable for a specific geographic location, over a defined period of time. Its estimation is based on observed rainfall data and on maximising the meteorological parameters linked to precipitation: humidity, temperature, pressure of saturating vapour in the air, wind speed, convection, etc. Such calculation requires the skills of a meteorologist. In order to facilitate the calculation, some countries have published regional *PMP* estimates.

The Shypre model: simulation of flood scenarios[2]

Cemagref[3] has developed a model for simulating flood scenarios called the SHYPRE model.

This approach is based on gaining maximum value from temporal information about rainfall episodes in order to generate flood hydrographs with realistic shapes. By coupling a stochastic model for simulation of hourly rainfall and a simple model for transforming rainfall into discharge, the method generates a collection of flood hydrographs over a very long period.

1. See *Bibliography*, references 4 and 8, p. 36.
2. See *Bibliography*, references 2, 3 and 10, p. 36.
3. The french agricultural and environmental research institute – http://www.cemagref.fr

In this way, with no prior assumptions concerning the law of probability, the frequency distribution, from routine to exceptional, can be constructed empirically. This goes for both peak flows and for average flows and threshold discharges of various durations. Such floods are available to calculate hydraulic transients, including attenuation in a reservoir and modelling a flood plain.

In this method, the stochastic model of rainfall not only complies with temporal information, it also gives an original approach to infinite rainfall behaviour. Processing some 50 rainfall gauging stations along the French Mediterranean coastline has given an idea of regional trends simply through the characteristics of daily rainfall and has opened up the possibility of building a spatial regional model. The method has proven to be more stable than simple statistics, which depend heavily on sampling, and the model helps to evaluate the impact of anthropic effects.

Empirical formulas and regional formulas

These methods of estimating flow are extremely succinct and under no circumstance may be a substitute for a complete hydrological study.

The Francou-Rodier envelope curve[1]

From observations of maximum floods over the last 2 centuries in 1400 catchment areas throughout the world with surface areas in the 10 - 2.10^6 sq.km range, FRANCOU and RODIER established an envelope curve formulated as follows:

$$Q/Qo = (S/So)^{1-k/10}$$

- Q is the peak discharge of the flood (m^3/s) for an area of the catchment S (km^2).
- $Qo = 10^6$ and $So = 10^8$.
- k is a regional parameter. It varies in France in the 5.5 bracket (Mediterranean zone) to 3.5 (oceanic zone in the north of France). At a global scale, the highest k values (and thus in proportion the highest flow) are close to 6 like in Texas, New-Mexico and in some of the Pacific areas affected by typhoons (Korea, Japan, Philippines, …). On the opposite, the large African tropical streams are quite well characterised by an exponent k value close to 2 (it is the case for the Niger and Senegal rivers).

Concerning an envelope of observed maximum floods, these estimates of discharge are not affected by the frequency of appearance. The authors, however, consider that a good part of the floods correspond to a recurrence period of around 100 years.

1. See *Bibliography*, reference 6, p. 36.

Synthesis of flow with a recurrence interval of 1000 years calculated with the Gradex[1] method

The GRADEX method has been applied by EDF for numerous French catchments of surface areas varying between a few square kilometres to a few thousand square kilometres. The regression established on 170 catchment areas of the peak discharge with a 1000-year recurrence interval relative to the surface area of the catchment is:

$$Q = \lambda . S^{0,72}$$

S is the area of the catchment in square kilometres and λ a parameter given in the table hereafter for the following three zones:

- *zone I :* the catchment areas of tributaries to the lower Loire river (Vienne, Creuse, etc.) located in the north of Central France, those of the Saône and Moselle rivers, and Brittany;
- *zone II :* the catchment areas of the eastern and central Pyrenees, of Aude and Ariège, of the Dordogne and the Lot rivers, the catchment areas of the Durance, the Fier and the Arve rivers, the Dranses, and the Isère rivers;
- *zone III :* the catchment areas of Haute Loire, Cévennes, the Tarn river, the right-bank tributaries of the Rhône river downstream of Lyons (Eyrieux and Ardèche rivrers, etc.), Alpes-Maritimes, and Corsica.

Zone	λ	Bracket for 90 %	Bracket for 70 %
I	4.05	3.07 - 5.36	3.4 - 4.8
II	7.4	5.2 - 10.4	5.9 - 9.2
III	16.4	9.1 - 29.7	11.3 - 23.9

This formula does not apply to catchment areas smaller than a few square kilometres. It only gives an order of magnitude which must always be rendered more precise with a local study. This order of magnitude is only to give a first opinion on the spillway capacity. It should never replace a more complete study.

Rainfall runoff analysis in a well documented catchment

It is very rare to find a small catchment where we have a good understanding of the hydrology. The subject of this study is therefore not determination of a dam's design flood. It presents data from an experimental catchment which support the recommended methods and justify the conclusion that the peak discharge of the design flood depends very little on how long the considered rainfall lasts.

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1. See *Bibliography*, reference 5, p. 36.

Presentation of the catchment area

Located in the Massif des Maures, a Mediterranean mountain zone, this catchment has been monitored by Cemagref since 1967, in the frame of the Réal Collobrier Research Project. The lands adjacent are mostly compact gneiss. There is little soil cover. The vegetation consists of scrubby briar and evergreen arbutus. The catchment area measures 1.47 square kilometres. Mean annual rainfall (1967-1990) is 1164 mm and the corresponding runoff is 626 mm.

The largest river floods are caused by intense storms occurring mostly in September and October. The following two events, with contrasting time histories, are an excellent illustration of the different types of response of the catchment *(table 2)*.

Date	Maximum rainfall (in mm)		Maximum runoff (in mm)		Peak discharge (m^3/s)	Ratio Peak discharge/ Runoff in 24 h
	24 h	1 h	24 h	1 h		
13.09.68	159	88	57	21	15.5	15.5
29.10.83	165	25	25	3,4	1.61	3.6

Table 2 - Floods of 13/09/68 and 29/10/83

Application of the Gradex method

The GRADEX method is applied with time steps of 1, 2, 4, 8, 12 and 24 hours, although these last times are much longer than the catchment's characteristic time. Figures 1 and 2 *(p. 35)* are examples of observed 4-hour rainfall and runoff.

The plots show all events during which rainfall in excess of a given value was recorded.

Catchment retention

Figure 1 *(p. 35)* shows that the retention loss was between 70 and 100 mm in the three heaviest rainfall events, and between 55 mm and 100 mm in the three largest flood events. Statistical analysis as described later indicates that the "pivot point" for the GRADEX method should be taken as the 0.95 frequency (20-year recurrence interval, reduced GUMBEL variable, $u = 2.97$).

Table 3 shows retention versus duration of rainfall.

Duration (hours)	1	2	4	8	12	24
Retention (mm)	30	43	57	76	86	98

Table 3 - Retention for a 20-year recurrence interval for different durations of rainfall.

Statistical rainfall and runoff distributions

Figure 2 *(p. 35)* shows rainfall and runoff on the vertical scale versus the standardised GUMBEL u.

The concave shape of the two curves is explained by the fact that events of very short return periods are plotted. The asymptotic rainfall distribution appears if we limit ourselves to the 27 most severe events (one event per year). It is not unusual for the most severe event (September 13th, 1968) to lie some distance off the curve.

Since there is considerable sampling uncertainty in a 27 value sample, the computations were run on the 150 most severe rainfall events observed for each of the durations considered in determining the asymptote slopes, proportional to the standard deviations. For this example, the fitting method would appear to overestimate the frequency of the highest values. This is attributable to the sampling alone, and application of this fitting method to samples of other rainfall durations yields estimates that are either entirely consistent, or underestimated (points concave downwards). Results are listed in table 4.

Duration (h)		1	2	4	8	12	24
Runoff (L)	a	2.24	4.43	7.72	11.9	14.9	22.3
(mm)	b	3.90	7.60	13.5	22.4	29.3	44.2
Rainfall (P)	a	6.82	10.9	15.9	22.5	26.8	34.3
(mm)	b	20.1	31.5	46.5	66.2	79.7	106

Table 4 - GUMBEL parameters a and b for rainfall or runoff: P (or L) = au + b

Application of these equations to a recurrence interval of 0.95 (u = 2.97) was the basis for preparing table 3.

The rise in the GRADEX and therefore the trend in the a values (slope of the rainfall distribution line) matches duration perfectly and can be expressed as:

GRADEX (t) = $7{,}4\ t^{0{,}51}$
Rare runoff is derived directly from these equations; by setting the pivot point at 0.95 (u = 2.97), runoffs of different duration at recurrence intervals of 0.999 are estimated at the values shown in table 5.

Duration (hours)	1	2	4	8	12	24
Runoff (mm)	37.2	63.3	98.6	146	179	245

Table 5 - Estimation of runoff at recurrence interval 0.999

Peak/mean flow coefficients

Ratios between peak flow and mean flow have been calculated for all the events considered. Their distribution makes it possible to estimate the ratios to be used in determining peak flood flows for very long recurrence intervals.

The ratios are very variable, especially for very small flood volumes. Obviously they increase with the duration of the event: between 1 and 2 for 1 to 2 hour events, they may be as high as 25 for some 24 hour events.

With the ratios recorded during the three largest floods for the 1, 2, and 4 hour durations (i.e. 1.6 - 1.74 and 2.67), the estimated 1000-year flood peaks are very similar, i.e. 24.3 - 22.5 and 26.9 m^3/s.

Conclusion

This application highlights the insignificant impact of the choice of duration on the GRADEX estimates of flood peaks for very rare events.

This is similar to what was found for much larger catchments where floods were estimated by studying one and two-day rainfalls.

Design flood hydrograph

The design flood hydrograph is calculated within the following constraints:
- A flood peak of 24.5 m^3/s (average of the 3 estimates);
- Flood volume of the 1000-year 24-hour value (245 mm, runoff yielding 0.36 hm^3).

The following formulation is used:

$$q(t) = \frac{qp \cdot 2 \cdot (t/D)^{\alpha}}{1 + (t/D)^{2\alpha}}$$

qp : peak flow
q(t) : flow at time t
D : characteristic catchment time, as defined above page 28.

A value α of 2.7 meets these two requirements and gives the design hydrograph shown in figure 3.

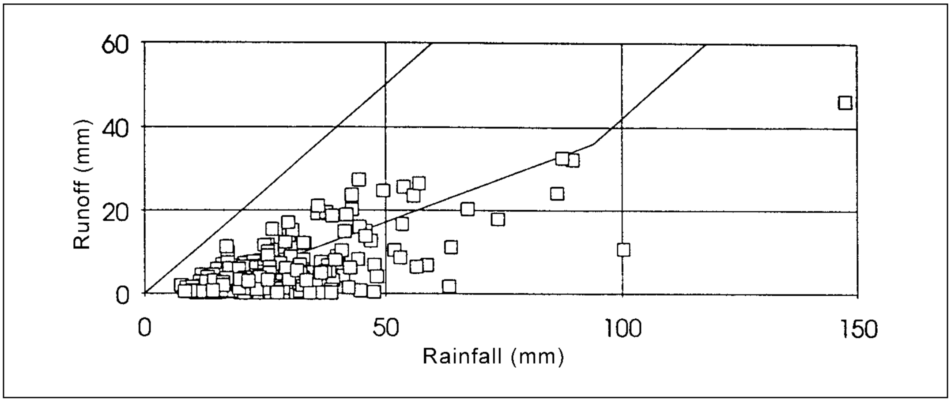

Fig. 1 - 4-hour runoff and rainfall

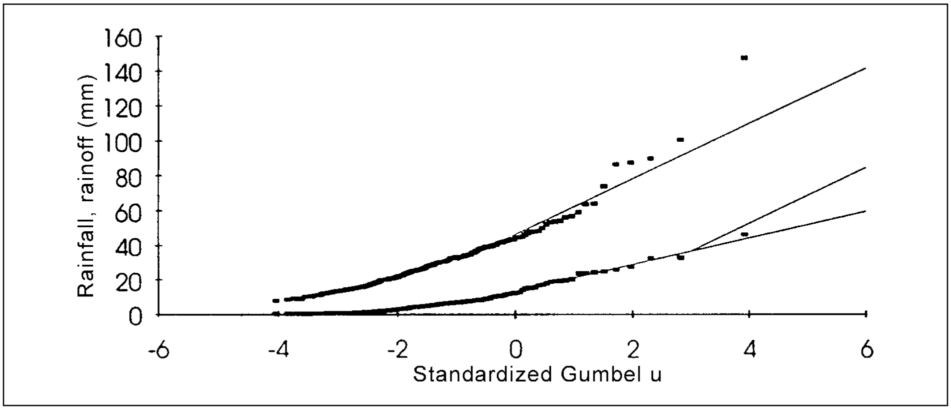

Fig. 2 - Statistical distribution of 4-hour runoff and rainfall

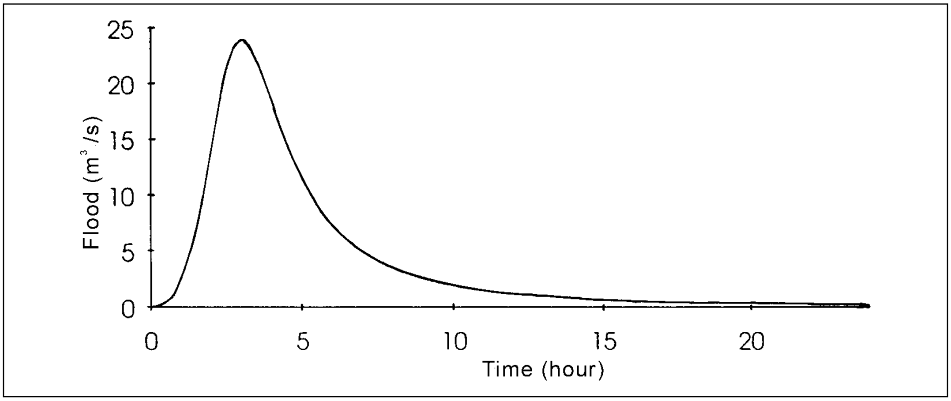

Fig. 3 - Design flood hydrograph

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CHAPTER III

Geological and geotechnical studies

Chairman: Patrice BRUNEL (CARA)
Committee members: Emmanuel ALONSO (Cemagref), Jean-Pierre BECUE (SAFEGE), Jean-Luc BRODIN (CACG), Bernard COUTURIER (BRL), Danièle LAUTRIN (Cemagref) and Georges MICHEL (SCP).

On the topic of geological and geotechnical studies, special attention is given to:

- *the various techniques available;*
- *identification and choice of a dam site;*
- *geological and geotechnical studies and site investigations to adapt the dam project to the chosen site, in particular including the choice of the most suitable type of dam for the site;*
- *geological supervision during dam construction.*

PRELIMINARIES

This chapter is intended to give some recommendations based on the authors' experience and current practice. Given the specific nature of geological and geotechnical problems, no typical program can be given but the minimum that is usually accepted is stated.

In fact, the fundamental importance of knowledge of the geological and geotechnical context in which the dam will be built, combined with the great variety of foundations that might be encountered, making each dam project unique, preclude any prior definition of the extent and nature of the required site investigations.

Only an experienced professional has the competence to adapt the studies undertaken to the geological context that they gradually reveal and to identified or assumed problems, following an iterative process that usually results in a site investigation program planned in several stages (with the orientation and contents of each stage defined as the preceding stage is concluded).

Those site investigations should make it possible to avoid, insofar as is possible, encountering any unforeseen problem as the dam is built, leading to improvisation, cost overruns and delays, which are always adverse (especially when the dam is small and financing generally limited with little margin).

Herein we successively look at:
- the various techniques used in geological and geotechnical studies for dam projects;
- methodological recommendations concerning the various stages in the geological and geotechnical investigations, from site identification to construction.

The basic concepts of geotechnics and rock mechanics are assumed to be familiar to the reader who, if necessary, may refer to the many specialised manuals published in those fields.

Techniques

This section looks at all of the techniques that are applicable to dams under a height of about 25 metres. Depending on the nature of the site and the size of the dam, only some of these techniques may need to be employed.

Site inventory

A systematic inventory of dam sites is undertaken when there is a need to find potential storage sites to meet specific needs for water resources within a given area, which may be small (a catchment area measuring a few hundred hectares) or vast (a large catchment area measuring several hundred square kilometres). This approach makes use of several techniques:
- desk study using topographical maps of a scale suited to the size of the envisaged reservoir (1/25 000 for the dams addressed in this document, bearing in mind that very small sites are not identifiable at that scale);
- aerial photography (stereo pairs);
- direct field investigation.

These three techniques are very complementary and it is always preferable to combine them whenever possible, as each provides information that helps to better assess the characteristics of the site:

- *maps* give a "precise" estimation of surface areas and volumes (dam, reservoir, catchment area);
- *aerial photography* shows the nature of the plant cover as well as land use (the date of the photography is essential and must be as recent as possible);
- *field investigations* take into account morphological details, among other things, give some preliminary indications on site geology and can reveal smaller sites.

SURVEYING

Surveying is an essential support for a geologist, who must always be able to locate more or less accurately the formations he observes both in planimetry and in altimetry. The sophistication of the studies done and importance of the problems under study make it possible to distinguish "simplified surveying" from "standard surveying".

Simplified surveying

It is always a good idea to carry out levelling of the first holes drilled without waiting for later detailed surveys in order to plot the analytical geological cross-sections with sufficient accuracy.

Surveying a summary topographical cross-section on the dam axis (with automatic site levelling and land chaining) is also extremely useful in estimating the dam's volume from the very beginning of the studies, as construction costs are largely proportional to that volume.

Standard surveying

Dam design requires accurate topographical maps based on surveys of the dam site (at scale 1/500 or in greater detail), of ancillary structures if any (scale 1/200 or in greater detail) and the reservoir basin, at least up to the exceptional highest water levels (1/2000 or 1/2500 is generally fairly suitable for basins measuring several dozen hectares; a scale of 1/5000 may be used for larger basins, although accuracy is reduced, and 1/1000 for small reservoirs measuring only a few hectares).

Such surveys are usually done by government designated expert surveyors, usually using computer means (electronic logs, automatic point plotting, contour levels). It is recommended that the surveyor provide a paper copy and a computer file of the survey points that can be used by a design engineer with CAD (Computer Assisted Design) facilities.

It is wise to take advantage of the surveying to indicate on the map all of the specific items whose accurate position must be known (boreholes, investigation pits, springs, various geological structures, etc.). The design engineer may help out by leaving clearly visible numbered markers in the field, or accompany the surveyor in his work.

In some cases (difficult-to-access sites, dense vegetation, very large basin, ongoing land ownership problems, etc.), the map of the basin may be drawn up by aerial stereophotogrammetry (complemented and adjusted by groundcontrols), which is less accurate but which may prove to be sufficient and sometimes more realistic.

Photogeology

Geological interpretation of aerial photographs (in stereo pairs) can be a useful complement to classic mapping (in particular for structural aspects in a rock context of severe weathering but with no significant plant cover), and even replace it in some cases (when no useable topographic documents are available), but in any case must be accompanied by adjustments in the field.

The usefulness of this technique is most notably the possibility it offers of highlighting structures that are not directly observable in the field by integrating many details that can only be seen with the distance aerial photography permits, and sometimes also large morphological features that are hard to see on the ground or that are masked (by vegetation or other).

The analysis of aerial photographs (preferably recent ones) also gives indications on plant cover and land use in the catchment area. They often prove highly useful in hydrological studies and analysis of sediment transport in the river.

Geological mapping

If outcropping permits, a geological map may be drafted at a scale selected according to the desired precision, the stage of study and the size of the site, generally using existing topographical records (existing maps blown up when necessary) or using maps from standard surveying work or more detailed maps if any are available.

The geologist will be attentive to indicate the nature of the formations in the substrate at any outcrops (shown to scale), and overburden in other places, while distinguishing as many different categories as necessary. The map should indicate any useful information on structure (dip, fold, schistosity, faults and cracks, veins, etc.), hydrogeology (springs, losses), and geomorphology (in particular old or recent landslides, indication of karstic formations, etc.).

Trial pits

Trial pits can be dug out with a mechanical shovel for site investigations at the dam site, in borrow areas for an earthfill dam, and possibly for evaluation of reservoir watertightness.

This is essentially done for earthfill dams but may also be used in site investigations for concrete dams when the rock substratum is not very deep in order to assess the extent of the stripping that will be required. However, in the case of a concrete dam, the tests will serve no great purpose as the loose overburden will be removed.

Pits dug with a mechanical shovel (preferably caterpillar type to give access everywhere) are probably the most widely used technique in studies of the geological feasibility of a dam site because of their low cost and the mass of information they provide within a limited time (15 to 20 pits 4 metres deep can be dug per day in most ground formations).

In general, pits are dug "from place to place" in the area to be occupied by the dam and in the reservoir basin (for potential borrow areas). In some cases it may be preferable to dig trenches along a carefully chosen line.

The depth of investigation is limited by the power of the mechanical shovel, the length of its arm and the nature of the ground. Depths from 4 to 5 metres are commonly reached with a shovel of at least 100 HP and a bucket 80 to 100 cm wide with well designed teeth (bucket of the kind used in earthworks).

The geologist can survey fairly precise cross-sections as the pits are dug by measuring the depth of each change in the ground and possibly taking remoulded or undisturbed samples of the various layers crossed (which will later permit comparisons between pits and may help in the correlations to plot analytical geological cross-sections). The level at which any water appears should be noted. Finally, it is important to be sure that no one goes down into a pit that could collapse at any time. No sample should be taken from the bottom of the pit unless it is shored.

It is generally a good idea to leave the pits open for a few hours or even a few days (if safety conditions permit - with the site suitably marked out if necessary), in order to improve possibilities for correlations and observe any phenomena that might take some time to manifest themselves (inflows of water) or to stabilise (ground water level).

Provided some precautions are taken and some information is approached by approximation, the pits can be used for permeability testing when it is deemed useful to roughly estimate the permeability coefficient in the foundation ground. Because they are more or less rudimentary, such tests (LEFRANC, NASBERG, etc.) give results that must be taken with caution because of the great number of parameters that are difficult to control but highly likely to influence them.

Routine practice is to align the pits on the centreline of the future dam. On the river banks, attention must be paid to ensure that the pits give an unbroken cross-section of the substratum. To this end, the work should proceed from the top of the river bank with each pit staggered from the preceding one by a distance such that the drop in elevation of the natural ground between the two points is less than or at most equal to the depth the preceding pit cut down to the substratum (in any case this will only apply if the thickness of overburden varies only slightly between the two points).

Geophysical studies

In some cases (rock foundations, consideration of a rigid type dam), techniques of *seismic refraction* and *"petite sismique"* (which are relatively quick and easy to perform) can be used to zone seismic wave velocities. They can generally be related to the degree of cracking and weathering in the rock, which sometimes makes it possible to locate accidents (faults or other) thanks to the velocity anomalies they generate.

When the site's morphology offers practically no clear local narrowing, use of this technique in the feasibility or preliminary design stage can help in choosing the dam location by locating one or several zones in which the unweathered bedrock lies nearer the surface.

The position of the unweathered rock in terms of depth is often close to layers with high velocities (> 4000 to 5000 m/s) and is generally decisive in choosing the level at which a concrete dam is set.

In zones of weathered rock or alluvium, electrical prospecting techniques can also be used, whether alone or in combination with seismic techniques. In the former variety, variations of electrical resistivity in the ground are used to deduce variations in lithology, weathering and cracking. Measurements can be made in the form of drillholes, electrical tracing or resistivity panels.

Geophysical measurements should be interpreted by an experienced geophysicist and require calibration on mechanical drillholes (generally with core sampling).

The best way of going about such calibration consists in asking the geophysician to draft an interim report from which calibration drillholes can be set out as well as possible. The geophysicist will then write a final report after familiarising himself with the results from those drillholes and, if necessary, refining his interpretation.

In very difficult sites where there is a potential or proven risk of running into underground cavities (karst, former mines or quarries, soluble rocks such as [gypsum], etc.), the technique of microgravimetry can be used in order to seek out, through systematic exploration, any adverse anomalies in the gravity field. Other site investigations (such as core samples) will then be needed to check the nature of those anomalies but they are then set out in full awareness of objectives and savings can be made versus systematic but blind site investigations (which compensates for the cost of the initial prospecting itself).

Core sampling

Core sampling is systematically carried out for dams over 20 metres high but is rarely used for dams under 10 metres high.

Core sampling is intended to give sufficient knowledge of the lithological make-up and structure of the various foundations layers, to be able to draw the analytical geological cross-sections that are necessary for a good understanding of the dam's foundation conditions.

This technique makes it possible to extend the investigations into every type of terrain, at greater depth than is possible with pits, and to undertake more reliable water testing than can be done in pits (although restricted to fewer points), and also, under certain conditions, to take undisturbed samples of the ground. Today it is possible to record the drilling parameters, which gives continuous information on the nature of the ground being crossed.

The high cost of this technique means that it is rarely used in preliminary stages of study (only for large dams or ones posing problems that justify use of the technique).

When it is used, it is advisable to set out the drillholes according to prior observations made in the field in trial pits, with the goal of answering the precise questions that arose when those pits were investigated (e.g. depth of the competent or watertight substratum of a significant layer).

The depth of the drillholes must be suited to the size of the dam and the geological context. In a valley bottom a drillhole will routinely be as deep as the planned dam is high and should cross the substratum for a thickness of at least 5 metres. On the river banks it should reach the first layers of the substratum found at the valley bottom (taking into account the structure and discontinuities due to dip, faults, etc.), in order to correlate as well as possible the results of all the core samples taken along the entire length of the dam area.

The following principles must be followed to get good quality core samples:
- the core drill must be chosen according to the nature of the ground being sampled. It may be either the ram type (with thin wall, with or without inside sheath, with a stationary piston, with a thick wall), or rotary type (single, double, triple, with or without extension). The French AFNOR standard referenced in the bibliography *(p. 66)* gives various uses of this equipment;
- the diameter of the core sample should not be less than 60 mm at the final depth of drilling, taking into account any reductions in diameter (caused by poor stand-up of the ground that would impose the use of temporary tubing). The usual diameter for site investigation core samples varies from 86 to 101 mm;
- after extraction, and removal of the "fines" created by the drilling, generally caked around the core sample (the residue of drilling in clay), the core samples must be carefully protected and boxed.

It is recommended to have colour photographs made of the core boxes as soon as the drilling is done, as in most cases these will be the only traces of the samples after a few years (it is rare to be able to keep core samples in good condition because of storage problems, spontaneous crumbling of some materials, theft, etc.). The core samples should be photographed after carefully washing or scraping off the fines, wetted to bring out details, correctly labelled (with the drillhole number and depth), and accompanied by a palette of standard colours and a scale showing a length, preferably in the artificial light of a flash.

As the cores are taken, the geologist should take detailed records of the samples including:
- a description of the ground's lithology according to depth including all pertinent data (nature, appearance, colour, porosity, oxidation, dip of the contacts, etc.) in order to permit correlations with neighbouring drillholes. This is accompanied by a drawing in a log, using a symbolic representation of the samples' nature, if possible a standard one;
- if necessary, the depth of the lower limit of the oxidation zone. This corresponds to the presence of metal oxides at the surfaces of cracks and joints, whether these come from deposits or weathering in situ (this limit generally coincides with the level of flow of surface waters and therefore with the lower limit of the decompressed zone below which cracks can be considered as tight);

- structural indications, depending on the nature of the ground, e.g. stability of the wall, intense cracking or crushing, percentage recovery or rate of undisturbed sampling (total length of core over footage drilled), cracking factor (median of the length of the samples within a sampling pass), RQD[1] if the nature of the ground is suitable (RQD is not very meaningful in a very anisotropic rock, schistosised rock in particular);
- hydrogeological comments such as the water level during and at the end of drilling, fluid losses during drilling, ingress of water, artesian springs, water tests, etc.;
- any information about how the drilling went: start and finish dates, nature and dimension of drilling tools and temporary tubing, any final fittings (piezometer, etc.), position of the limits between core sampling passes, miscellaneous incidents (collapse, falling tools),water levels at the beginning and end of each shift or day.

The number, spacing and depth of the drillholes cannot be set in absolute figures but must be defined during the previous investigations, taking into account the specific features of each site, how heterogeneous the ground foundation is, and the spatial scale of lateral variations in facies, as well as any problems that may crop up.

The drillholes should be set out as far as possible in harmony with other investigation techniques (either by filling in any gaps in the observations or by permitting mutual calibration or by bringing a response to questions posed by those techniques). Drilling that only goes through rock may in general be oriented in any direction with no major difficulty. In loose overburden, the drillholes should be sloped at most 30° from the vertical.

It is common to drill at least three holes along the axis of the dam (one at the valley bottom and one at the top of each abutment), but more if the dam's crest length is greater than 100 metres. A horizontal spacing of 50 metres and vertical spacing of 10 metres between consecutive drillholes is recommended but those figures may be too high in some cases.

If the base of the dam extends more than about 100 metres on either side of the dam axis, two additional lines of drilling (at the dam's upstream and downstream limits) may be required, especially when the foundation is very heterogeneous or of mediocre quality.

Core samples may be accompanied by water tests (LUGEON, under pressure, in rock; LEFRANC, by natural flow, in loose overburden), especially on the axis of any watertight elements. Getting meaningful results from water tests requires the use of suitable equipment complying with standardised operating conditions and carrying out tests in a rational manner:
- drilling must be exclusively with clear water (no bentonite or biodegradable slurry) and before each test the drillhole wall must be cleaned (by successive passes of the tool with water injected in until clear water comes out at the drill head), in order to remove any fine deposits ("cake") that could clog the pores and cracks responsible for permeability and thereby distort measurements;

1. *Rock Quality Designation* = total of the lengths of the core samples greater than 10 cm / length of the corresponding drilling.

- maximum pressure during LUGEON tests should be adapted to depth. In general pressure[1] is limited to about 0.3 to 0.5 MPa for the range of dams considered here. It should not be forgotten that this kind of test is only valid strictly speaking if the corresponding discharge/pressure curve is practically straight, which must be checked for every test by holding pressure at various stages (held for 10 minutes), following an ascending and then descending cycle (e.g.: 0.05 - 0.1 - 0.2 - 0.3 - 0.2 - 0.1 - 0.05 MPa);
- in case of "total water loss", continue testing to distinguish between filling of pockets and permanent flow;
- positioning the packer in ground that is strong enough to support the inflating pressure with no creep and homogenous enough to avoid any perforation of the membrane;
- checking head in the annular space between the injection rod and the temporary tube at the beginning and end of the test in order to detect and quantify any flow around the packer;
- preferably carrying out LUGEON tests as the work progresses with the same packer used each time, which halves the risk of water flowing around the packer;
- preferably measuring the pressure in the measurement chamber rather than at the drilling head (which always means inaccuracy in the calculation of head losses), and recording flow and pressure continuously in order to check that they remain constant during the test;
- for LEFRANC tests, the most difficult point is to know or to check the shape of the injection chamber and especially to isolate it correctly from the rest of the drillhole. One technique that is sometimes used is to install a packer for LUGEON tests, and inject water by natural flow by the central tube (constant or variable level). To get a meaningful response, a number of these tests must be done and each change of facies must be tested.

It is possible to take undisturbed soil samples during core drilling. For fine soils, the recommended core drills are: stationary piston or thin wall sheathed ram, triple rotary drill with extension.

Undisturbed soil samples must immediately be oriented and correctly numbered; sealed at both ends with paraffin wax (in order to avoid any loss of water); and handled, stored and shipped with care, or the representativity of the tests done will be questionable.

The lengths of core corresponding to the samples taken can only be examined after the sheath has been opened in the lab. The drilling cross-section should therefore mention that a sample has been taken and the description of the ground will have to be completed later. A wooden block must always be used to replace the part taken out of the core box with an indication of the references of the sample.

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1. Here we speak of effective pressure, i.e. the pressure at the centre of the test pass. If no system is available to measure pressure directly in the testing chamber, account must be taken of head losses and overpressure corresponding to the water column in the system (Δz = drop in altitude between the pressure gauge and natural groundwater level): **Peff = Pmano - Pc + Δz/100** (in MPa).

Geotechnical laboratory tests

Knowledge of the physical characteristics and mechanical and hydraulic behaviour of the materials in the dam foundation, and of any materials whose use is envisaged in dam construction, is necessary for the design engineer to design the best, most suitable dam possible for the geotechnical context.

That knowledge is in part acquired through geotechnical tests in a laboratory.

Such tests require the use of standardised procedures and specific equipment. They can only be entrusted to experienced and fully equipped soil and rock mechanics laboratories.

Materials and overburden

For the type of material generally referred to as soil, tests are done on samples taken from the ground (undisturbed or remoulded from drillholes and trial pits). The tests can be divided up into:

- *identification tests:* natural water content, grain size (sieve and sedimentation analysis), Atterberg limits, methyl blue tests, particle unit weight, apparent bulk unit weight, etc.;
- *compaction tests on materials from borrow areas:* Standard Proctor test;
- *mechanical and hydraulic tests:* compressive strength, shear strength (using a triaxial device), œdometer compressibility, permeability using an oedometer or permeameter.

The number of tests of each type must be suited to the probable volume of the embankment, its height, the number of materials of different types that will be used (in the case of zoned dams) and to the natural variability of the materials under study.

As an indication, a minimum testing program recommended for materials from borrow areas for earthfill dams causing no special difficulty is determined according to the volume of material to be investigated[1] :

- series of identification tests (natural water content, grain size curves(sieve and sedimentation analysis), Atterberg limits): one for 5 000 to 10 000 m^3 of materials to be used with a minimum of five tests;
- compaction tests (Standard Proctor and particle unit weight): one for 15 000 to 25 000 m^3 with a minimum of five tests;
- tests of mechanical and hydraulic behaviour (triaxial shear, oedometer compressibility, permeability): one for 30 000 to 50 000 m^3 with a minimum of three tests (but no test of this type is recommended when $H^2\sqrt{V} < 5$).

For the foundation, the number of tests to be done can be of the same order of magnitude if loose overburden is thick (compaction tests are then pointless).

1. The volume to be investigated must be 1.5 to 2 times greater than the geometrical volume of the dam.

The section *Geotechnical studies* in Chapter IV, p. 68 emphasises the advantages of a simple test (water content) and explains what practical conclusions should be drawn from the results of those various tests.

Rockfill

For aggregate and rockfill, the laboratory tests required are:
- *measurement of intrinsic characteristics:* apparent density, mineralogical study (examination of thin slices under a microscope);
- *measurement of state characteristics:* water content, grain size range and block size range, shape coefficients, porosity, degree of cracking, continuity index;
- *measurement of withstand characteristics:* impact (Los Angeles or LA test), abrasion (Micro-Deval test with water), compressive strength Rc (on cylindrical core samples), freeze thaw cycles.

SITE TESTS AND MEASUREMENTS

Several types of tests and measurements can be done in situ at various stages of study on a dam site according to a variety of criteria (nature of the foundation, geotechnical problems, dam size, etc.).

They can be used to roughly measure the foundation's mechanical characteristics, incorporating the effect of discontinuities in the rock mass.

The tests done will depend on the nature of the foundation. In loose overburden, we can cite:
- *static or dynamic penetrometer tests,* which in particular distinguish between layers of different consistence without offering the possibility of directly and reliably linking peak resistance to their mechanical characteristics;
- *pressure meter tests* to obtain a stress-strain relation in the ground (e.g. determination of a strain modulus, creep pressure and limit pressure);
- *shear vane apparatus* to measure the undrained cohesion of the ground when it is less than 0.1 MPa. It is not possible to measure the cohesion of layers whose thickness is less than the height of the vane apparatus blades;
- *phicometer* to give an approximate direct measurement in the drillhole of the shear strength of heterogeneous or coarse soils.

These various in situ tests are less rarely used to study a dam than for other constructions except when the foundation is of very mediocre quality, for example mud.

On rocky terrains, the classic tests are as follows, the first two concerning the foundation, and the second two knowledge of materials:
- *dilatometer deformability test in the drillhole, plate bearing or flat-jack test* to measure the strain modulus of the rock in various directions (these tests are difficult to do, require highly qualified labour and are very expensive, which generally restricts their use to the largest structures);

- *measurement of seismic velocities in the foundation rock,* which gives a global value for the quality of that rock;
- *rockfill blasting tests* using various blasting systems and charges should be done, with a determination of the characteristics of the resulting rockfill (block size distribution, shapes, amount of tailings, etc.);
- full size rockfill compaction test embankments.

Recommendations for conduct of the studies

The sequencing suggested hereafter is obviously subjective. It corresponds to routine practice for dams of a certain size but obviously it would be possible to envisage a different breakdown of the geological and geotechnical studies, in particular for small dams (with the phases simplified and combined), or according to the specific features of the site whether they are technical or not (for example land ownership problems may cause difficulties in access and can speed up or delay some site investigations).

Similarly, the naming of these phases is not standardised, and practice varies considerably from one project to another. Outside questions of terminology, the important point is that the content of the studies corresponds to the stages that must be gone through in succession when a dam project is set up and carried out.

Site identification and selection

Objective

If possible, the ideal situation is a location near the needs to be met, permitting storage of the required volume of water (in relation with the results of the hydrological study, i.e. filling by natural or mechanical means), with a geometrically optimal structure (ratio of the volume stored to the volume of the dam, which is directly related to its cost). In the case of a reservoir for recreational purposes, the surface area of the lake will be more important.

Methodology

Systematic inventory of potential dam sites is generally carried out over a certain area and according to criteria that are related to the nature of the needs to be met.

Projects can be sorted in a preliminary manner in this stage according to various considerations (suitability to the size of the catchment area and therefore natural inflow to meet storage requirements; land use constraints such as flooding of inhabited areas, roads or other networks, etc.; potential of the site's geometry; proximity to needs; and so on).

Brief preliminary calculations will be done on each site to determine the main geometrical characteristics:
- depth-area curve, depth-volume curve (dam and reservoir);
- for one or several values of storage: height, crest length and volume of the dam (where necessary for each type of dam envisaged at this stage), flooded surface area, ratio of storage to dam volume.

When carrying out these studies, measurements of lengths and surface areas from available topographical documents will be used as well as calculations of volumes through incorporation of depth-area curves. The use of specific computer programs on PCs helps in these calculations and makes it possible to print out comparative charts with graphs.

A preliminary cost estimate for the dam according to one or several dimensioning hypotheses is generally carried out in order to compare sites and/or seek out an economic optimum. This is done by applying ratios or benchmark costs taken from the designer's previous experience with dams of the same type, using data bases specific to each engineering firm complemented by a calculation of the same ratios for the recorded cost of dams of the same type (after discounting all of those prices).

It is then possible to rank the sites selected after this first sort, according to criteria that will vary in nature and priority according to the nature of the project, and which must be chosen by the designer.

Dam selection according to the principles set out above is in particular applicable for the various alternatives for a single site. It can in fact be necessary to study several possibilities for storage and/or layout unless there is only one clearly localised topographical narrowing at the site.

Geological surface study

Objective

After the stage of dam site identification, a visual examination by a geologist with experience in dams is vital before the studies are continued. This only takes a day or even half a day.

The site tour is intended to determine the broad lines of the site geology before any heavier site investigation techniques are implemented. It has many purposes:
- to situate the site in the local and regional geological context;
- to detect any prohibitory geological conditions that might be immediately visible;
- to orient the rest of the studies, and in particular define and set out later site investigations;
- possibly to refine the dam layout, taking into account geomorphological or other details.

Methodology of the geological surface study

Before the site tour, the geologist refers to existing geological maps (1/50 000, or if none 1/80 000) which essentially enable him to place the site in its local geological lithostratigraphic and structural context. In some cases, this preliminary examination alone can result in a strong presumption that the geology is unfavourable (for example karstic formations).

Prior knowledge of the geological context is vital as it enables examination of the site to be oriented towards a search for certain types of indications using previous experience with similar contexts.

Depending on the region concerned by the site in question, locating major regional tectonic structures can be of great importance in the rest of the studies and can sometimes explain the special behaviour of some foundations.

The methods used in this kind of site investigation will vary according to the size of the dam under study and the geological context. They will consist at least in touring the dam site and all or part of the reservoir basin while noting any possible observation, and can be complemented by geological mapping at a suitable scale, surveying lithostratigraphic cross-sections with sampling, examination of aerial photographs and in some cases examination of satellite images.

The result of this first site investigation is a preliminary assessment (sometimes called prefeasibility) on the wisdom of undertaking more detailed studies. In this stage sites can be classified in the following categories:

- *favourable sites,* when no prohibitory condition has been revealed;
- *unfavourable sites,* where problems appear that would be hard to solve and/or would have an economic impact out of proportion with the advantages to be procured by dam construction;
- *doubtful sites,* that can be divided up between those where no surface observation is possible because of outcropping conditions and those where uncertainties persist as to the interpretation and/or gaps in those observations. Investigations in continuous pits dug out with a mechanical shovel are then necessary to rank the site in one of the two preceding categories.

It is useful to record the observations made during this phase of study and the conclusions drawn from them on a summary sheet like the one given hereafter and proposed by B. COUTURIER[1].

In addition to filling in the data required in this sheet, sketching a rapid geological profile on the spot along the proposed dam axis is an opportunity for the geologist to express his vision of the site after these preliminary investigations. The geological cross-section should distinguish between observed facts, interpolation, and pure intuition, and will be more precise when outcropping conditions are good.

1. See *Bibliography*, reference 3, p. 66.

<table>
<tr><td colspan="2">CONTRACT</td><td colspan="3"></td></tr>
<tr><td colspan="2">NAME OF THE SITE</td><td colspan="2"></td><td>N°</td></tr>
<tr><td rowspan="2">LOCATION</td><td>MAPS</td><td colspan="2">Surveyor :</td><td>Geol.:</td></tr>
<tr><td>COORDONNATES</td><td>X. :</td><td>Y. :</td><td>Z. :</td></tr>
<tr><td rowspan="4">DAM SITE GEOLOGY</td><td>LITHOLOGY
STRUCTURE</td><td colspan="3"></td></tr>
<tr><td>FOUNDATION QUALITY</td><td colspan="3"></td></tr>
<tr><td>STABILITY OF ABUTMENTS</td><td colspan="3"></td></tr>
<tr><td>WATERTIGHTNESS
(hydrogeology)</td><td colspan="3"></td></tr>
<tr><td rowspan="3">BASIN GEOLOGY</td><td>LITHOLOGY
STRUCTURE</td><td colspan="3"></td></tr>
<tr><td>SLOPE STABILITY</td><td colspan="3"></td></tr>
<tr><td>WATERTIGHTNESS
(hydrogeology)</td><td colspan="3"></td></tr>
<tr><td rowspan="2">MATERIALS</td><td>AGGREGATES</td><td colspan="3"></td></tr>
<tr><td>FINE SOILS</td><td colspan="3"></td></tr>
<tr><td colspan="2">INVESTIGATION WORKS</td><td colspan="3"></td></tr>
<tr><td colspan="2">CONCLUSIONS & COMMENTS</td><td colspan="3"></td></tr>
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It can sometimes be difficult to classify a site in the "unfavourable" category, and the engineer will then tend to qualify it as "doubtful" so that it will not be abandoned without further investigations. But although it is difficult to dictate a general rule, experience shows that it is generally preferable in case of a serious doubt to abandon a site that in fact is favourable rather than undertake in-depth and therefore costly studies on a site which could prove to be unfavourable at a later stage of the project. This approach is all the more essential when there are other alternatives available.

Geological feasibility study

The term "feasibility study" calls for a few preliminary remarks:

- this essential step in a dam project is given this name to indicate that it is the point beyond which only a very small possibility of failure should persist (i.e. that the project will be abandoned because of geological problems that were not detected previously) and on which the initiation of more in-depth (and therefore more costly) studies depends;
- however the term does not mean that the project's geological feasibility cannot be further questioned during later design stages, in especially difficult cases which should remain exceptional;
- it would seem preferable that the person in charge of the study have the means to form an objective opinion at this stage and one that is as reliable as possible, even if, in dubious cases, this means carrying out a few more detailed site investigations that were not initially foreseen rather than delaying the decision to the preliminary design stage (which would mean financing a complete preliminary design without being assured of a successful conclusion);
- a certain variety of practice among engineering firms can be noted on this topic, between the explanation given above and the definition of the feasibility diagnosis report in the preliminary design stage, according to the type and size of dam, and maybe also to the dam owners involved. In particular it may occur that development is no longer handled by the same entity in the preliminary design stage. The essential point is still that the engineer should carry out his mission in accordance with his client's expectations and provide him with assistance in decision-making throughout his project.

Objective

After the geologist gives a preliminary opinion in favour of the site, a number of more in-depth studies and investigations must be carried out in order to:

- confirm that there are no prohibitive geological and geotechnical conditions that could contradict the previous opinion;
- specify in greater detail the project's geological context;
- progressively refine the definition of the most suitable type of dam for this context and its exact layout;
- specify in greater detail, in the case of a fill dam, what might the best location for the spillway;
- orient and define the site investigations that will be necessary in later development phases (preliminary design).

The geological feasibility studies are generally conducted in parallel with other types of feasibility studies:

- *land ownership,* which is of growing importance and can predominate over other aspects of the project;
- *environmental,* as ecological aspects of dam projects are studied in environmental impact assessments that should take into account the consequences of the work envisaged on the natural milieu, both at the site and in its vicinity and on the river downstream;

- *economic,* as the viability of the project must be studied according to its purpose, or possibly purposes (irrigation, increased dry weather river flow and pollution control, flood routing and protection from flooding, tourism and recreation);
- *local development,* as a dam project is sometimes an opportunity to initiate reflection on the future of rural areas in difficulty (revival of the local economy by attracting vacationers, etc.).

Although it may seem logical to conclude on the geological feasibility of the project before initiating these studies, their growing importance and the time needed to carry them out often mean that they must be initiated simultaneously and sometimes even that geological studies must be suspended until these other feasibility studies have reached a favourable conclusion.

This last point reinforces the importance of the surface geology study mentioned above *(see Geological Surface Study, p. 49)* and is the reason behind the recommendations made concerning the potential consequences of an overly optimistic diagnosis in case of any doubt.

It is in this stage of study (feasibility) that the essential characteristics of the site should be known and, as far as possible, major problems that could lead to a decision against the project should be detected.

Although there is generally a technical solution to any problem encountered, the cost may in some cases be disproportionate versus the dam's economic advantages and thus render it unfeasible.

Methodology of the geological feasibility study

Geological feasibility studies require the use of various techniques according to the size of the dam, the nature of the geological context and/or the habits of the geologist.

The methodology recommended hereafter for geological feasibility study of dams of the type addressed in the present volume therefore does not pretend to be universal and must often be adapted to the specific features of the case at hand.

A typical feasibility study could be conducted in whole or in part in the following stages:

STAGE 1: Desk study

For any dam, even the smallest, it is of great interest to consult existing geological maps and the accompanying documentation. For dams about twenty metres high or more, it can be of interest to search within regional geological literature (articles in specialised journals, theses, monographs, notes to geological maps) for any previous studies done on the region of the site under study or simply mentions of details that could be useful in understanding the context and the geological history of the area (location and description of outcroppings, of fossil-bearing formations, borrow areas for various useable materials, particular structures, cavities, springs or watershed leakage, etc.).

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STAGE 2: Detailed site tour and mapping

The geologist undertakes a detailed tour of the dam site and the reservoir basin and maps all of the details. If it is believed that a saddle dam may be necessary because of the reservoir's altitude, its site must also be given attentive geological study. In fact, although such dams are generally very small, failure can release a significant volume of water. At the same time, saddles are often in zones where the substratum is of mediocre quality. If the context is suitable, a geological map may be drafted in the course of this study (at a scale of 1/25 000 to 1/5000 depending on the surface area concerned).

Conversely, local geology can in certain cases make any surface observation ineffective, because there are no outcroppings (in France the soft Tertiary sandstone of the Aquitaine area is an example), and this stage, although still necessary, can be shortened. Among the investigations to be done, special attention must be paid to detecting any instability, either at the dam site itself or in and around the reservoir basin (landslides, effusions due to solifluxion, rockfalls, boulder falls, or unstable slopes, rock walls or entire river banks).

This kind of investigation must be systematic, especially in areas where there are frequent signs of instability.

The activity and extent of any phenomena observed should be evaluated as well as the risk that they would be reactivated or worsened by the works (excavations, quarrying) and by the dam itself being operated (emptying the reservoir), in order to define any strengthening or river bank drainage systems that might be necessary.

The presence of extensive zones of instability is generally a very unfavorable circumstance for dams, and a project can only pursued in such a context if the greatest possible precautions are taken to guarantee the dam's safety, and if due thought is given to take into account the requirements that these problems would inevitably impose on the developer or the dam operator during the scheme's lifetime.

STAGE 3: Hydrogeological and structural studies

These brief studies are done during the detailed site tour and can in some cases result in either specific mapping or simply data marked on the overall geological map.

- *brief hydrogeological study*

In this stage, the goal is to define the broad outlines of the hydrological conditions in the foundation of the dam and the corresponding reservoir basin, and their local context.

Generally, this study involves an inventory of trial pits (collecting data such as water levels and variations in them, geological layers, operating discharge, etc.), springs, resurgences or losses.

Depending on how dense the inventoried points are, an attempt can be made to sketch out the natural groundwater level in the dam area and deduce some preliminary ideas of its probable behaviour due to creation of a lake (if the water table on the river banks is higher than the level of the future reservoir,

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it will play the role of a hydraulic barrage and prevent any leakage to the sides even if the ground is permeable; if there is a karstic system at a lower level than the future lake, there is a risk of flows past the dam if any outlet exists at a lower level downstream of the site or in an adjacent valley, etc.).

The results of the hydrogeological study can be quite heavily dependent on the season it is done in and rainfall in preceding months or years. The consequences for the project must therefore be deduced with precaution, in particular making sure that the worst cases have been properly identified and taken into consideration.

- *brief structural study*

When the terrain is suitable (rock foundation), a brief survey of the main structural features of the site (dip, strike and density of cracking, major faults, folds, etc.) can support the geologist's opinion as concerns the mechanical strength and watertightness of the foundation. Sometimes this will help in choosing the most suitable type of dam and in general contributes to the definition and layout of later site investigations.

Furthermore, searching in the literature for data on the regional structural context can orient the field study or help in interpreting field data. For example, it would make it possible to distinguish between families of regionally significant strikes, to associate them through that distinction to one or another tectonic phase, to foresee what state they are likely to be in (open or closed joints), or later to explain their behaviour during in situ tests, according to their orientation versus the present regional tectonic stress field.

STAGE 4: Trial pits excavated by mechanical shovel
(site and borrow areas for fill dams)

This is a vital complement to the detailed site tour discussed above. It is all the more important when outcrop conditions are poor.

In rocky ground, the depth of investigation is in general limited to the weathered surface layer, but at least indications can be obtained on the depth of the first competent layers, and in particular those of the substratum under the alluvial matter that is generally present in a valley bottom.

In particular it is recommended to check whether any fossil river valley exists, by the river's own meandering or due to human action.

In accordance with the principles set out above *(see "Trial pits" above in this chapter, p. 40)*, it is recommended to dig at least:

- a line of pits along the envisaged dam axis, with one pit every 15 to 20 metres (horizontal distance) on average,
- pits distributed in the area(s) envisaged for extraction of dam construction materials, one per hectare on average. The objective is to obtain preliminary indications on the thickness and nature of the loose material available in the reservoir basin, and/or on the cover that would have to be stripped off to reach useable rock materials if necessary, which would help in orienting the choice of the type(s) of dam that could be built on the site in question.

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STAGE 5: Core sampling, if any

Core sampling at this stage of study is generally restricted to dams of a certain size or with particularly tricky geological problems, for which this technique alone is deemed likely to enable the geologist to emit an objective opinion (risk of high permeability in the foundation that would influence feasibility, substratum that cannot be reached with a mechanical shovel in pits, etc.).

STAGE 6: Brief survey

Levelling of the investigation pits and especially the drillholes is vital to achieve a good correlation between cross-sections. A rapid levelling instrument and land chain give satisfactory accuracy.
It is wise to leave marks (rods, benchmarks) that can be reused in regular surveys during later design stages (the drilling can then be set out more accurately).

STAGE 7: Interpretation and drafting of the final report

Interpretation of the data collected during site tours and in the various trial pits or drillholes makes it necessary to set up geological profiles in numbers and in locations tailored to each case (in general, there will be a profile for each axis envisaged for the dam, along with cross-sections running perpendicular to or obliquely across this axis to show the geological make up of the foundation; cross-sections in the potential borrow areas on the site can also be drawn).
The final opinion is then given in the form of a report that reviews the various problems that may be encountered on a dam site, for example along the following outline:

1 *Introduction*
- 1.1 Purpose of the study
- 1.2 Geographical location
- 1.3 Summary of previous studies

2 *Geology of the dam site*
- 2.1 Morphology
- 2.2 Lithology
- 2.3 Structure
- 2.4 Foundation quality - abutment stability
- 2.5 Foundation watertightness

3 *Geology of the reservoir basin*
- 3.1 Morphology
- 3.2 Lithology
- 3.3 Structure
- 3.4 Stability of reservoir slopes
- 3.5 Reservoir watertightness

4 *Materials available near the site*

5 *Conclusions*
- 5.1 Site investigation program
- 5.2 Opinion on feasibility

One important element in the conclusion on the feasibility study is the choice of a type (possibly types) of dam that is/are the most suitable to the geological context revealed by the study with special attention paid to foundation quality and with account taken of the availability of useable materials on site.

A precise definition of the site investigation works that must be undertaken during later stages of study is also of great importance. It enables the investigations to progress continuously by foreseeing the means needed to answer any questions that have been raised or fill in gaps in the data that remain after the feasibility study.

Geological and geotechnical studies in the preliminary design stage

Objective

Specialised studies prior to establishment of the preliminary design of a dam are only done if the feasibility study concludes in favour of the project and should allow the owner to make a decision as to whether to undertake the process of building the dam.

The purpose of the preliminary design study is to define the general outline of the dam, meeting the needs expressed by the client and fitted to the context, if necessary reviewing the various alternatives that may be considered and costing each of them approximately but realistically.

The importance of this first estimation of capital cost is very great as it will often serve as a basis in seeking financing and evaluating the economic advantages to construction.

For this reason, the owner must be safeguarded from any later nasty surprise by trying to have a generous approach to this cost - without accumulating contingencies - in particular with no attempt to hide any of the technical problems that may have been revealed or that are simply suspected.

In France this stage of study for dams 20 metres high or higher corresponds to the Preliminary Brief that must be submitted to the Permanent Technical Committee on Dams. Conversely, for the smaller dams that are more specifically addressed in this work, it frequently happens that this phase of study is combined with the tender design in the form of specialised design studies.

Methodology for geological and geotechnical studies at the preliminary design stage

Specialised geological and geotechnical studies for establishment of the preliminary design of a dam usually include the following stages:

Detailed site investigations

The geologist supervises the site investigations as defined during the feasibility studies and where necessary modifies the programme to adapt it to the information collected (layout, depth, number of drillholes, nature of the in situ tests, sampling, etc.).

Within this effort, he may make additional observations in the field, sometimes after conditions of visibility have been improved (when scrub has been cleared, for relatively large dams), may draw up a detailed geological map if necessary and may survey cracking on outcrops or in the bottom of trial pits (when the foundation is rock).

The investigations carried out are generally the following:

- *detailed survey:* it is generally in this stage of study that the surveys of the dam site and reservoir basin described in the section on topography at the beginning of this chapter are done *(p. 39)*. All the drillholes and trial pits in the dam foundation and reservoir basin must be indicated on detailed maps.
- *trial pits:* this technique is often used at the stage of design investigations to complement other investigations, clarify any areas of doubt, and study one or several dam and, if necessary, spillway layouts.

Trenches oriented in meaningful directions with respect to the dam or geological structures (generally along the dam axis and according to how the dam intersects the pre-existing topography) are recommended in this stage of study for large dams or where the geology is particularly complex.

- *geophysical studies:* in this stage rock foundations are investigated by seismic refraction or by "petite sismique". A classic system consists in:

- one seismic profile along the dam axis (which should extend quite extensively outside the dam area into each bank);

- one or several profiles running perpendicular to that axis (for example one in the valley bottom and one on each bank);

- one or several profiles on the axes foreseen for the ancillary works.

- *core sampling:* this is generally done in this stage of study, essentially along the dam axis (and/or that of any watertighten structure), and more rarely in the reservoir basin or in borrow areas (to study specific problems or thick overburden, investigate borrow areas of considerable thicknesses or rockfill quarries, etc.).

- *detailed investigation of borrow areas (for fill dams), including sampling:* this consists in digging trial pits by mechanical shovel in a tighter pattern than in previous investigations and taking remoulded samples.

In this stage of study, the trial pits should not be spaced less than 50 metres apart (four pits per hectare, depending on local conditions). It is preferable to space the pits as regularly as possible while organising them along parallel topographical profiles and perpendicularly to the contour levels.

A detailed cross-section of each pit is recorded, mentioning any samples taken. The thickness of the unusable material that must be stripped off from the surface (organic top soil); the depth at which flows of water are encountered and if possible of the water table (as well as foreseeable variations in it), the thickness of any intermediary layer that must be stripped off; and any instability in the walls should all be noted.

Samples should be taken according to changes in the ground's nature, which often means taking a sample every metre of depth on average, limited to the ground that, at first consideration, can be used (above the water table in particular).

The weight of the samples must be sufficient to carry out the planned geotechnical tests (approximately 2 kg for a simple identification, at least 20 kg for compaction tests, or even more in the case of very coarse materials). If possible, the sampling should be done by the laboratory in charge of the testing.

- *investigation of concrete aggregate in borrow areas (conventional concrete or RCC):* these borrow areas are investigated by mechanical shovel in the same way as the earthfill borrow areas; as for investigation of potential quarries, this involves study of the rock levels involved (petrographic nature, structural study, density of cracking, thickness to be stripped off), if necessary accompanied by core sampling and geophysical tests. In general, there can be no question of opening new quarries, except any that would be flooded by the reservoir.

- *in situ tests:* the tests mentioned in the section on In situ Tests *(p. 47)* and Measurements may be done, if the dam or the problems encountered are important enough to warrant them.

Interpretation of data

All of the information provided by the site investigations is interpreted by the geologist who thereby improves his knowledge of the site. Depending on the size of the planned dam, the geological nature of the site and the problems encountered, he will establish any necessary documents: geological cross-sections, permeability and fracturing profiles, structural diagrams.

Geotechnical tests in the laboratory

A more or less considerable part of the necessary testing *(see "Geotechnical laboratory tests" above in this chapter, p. 46 ...)* is done at this stage of study according to various criteria (dam size, budget or land ownership requirements, consultant's engineering practice, etc.). However, it is economically advantageous to do all of the sampling at once even if only some samples are studied in the laboratory at the preliminary design stage, especially for small dams. In any case, all of the recommended tests must be done in the tender design stage. It is acceptable to do only a limited number during the preliminary design, for example restricting investigations to definition tests.

Geotechnical summary

The geotechnical summary is based on the reports on laboratory tests, site investigations and in situ measurements, and is focused on distinguishing between families of homogeneous materials, whether in the foundation or in borrow areas, indicating for each the ranges of values for the various measured parameters.

The first stability calculations can be done on the basis of this summary and make it possible to define probable cross-section(s) for earthfill dams and foundation level for other types of dams.

The geotechnical summary should also include a judgement on materials availability at the site according to the type of dam planned and possibly draw attention to the need to investigate new borrow areas before the works begin.

Final report

The final report is drafted after all the preceding stages have been completed and includes:

- a summary of all the geological studies done since the beginning;
- the geotechnical summary mentioned above (preliminary geotechnical study);
- definition of investigations for the tender design stage (core sampling, undisturbed or remoulded samples, geophysical tests, geotechnical tests in a laboratory and *in situ*, etc.);
- supporting arguments for the type of dam recommended as the most suitable for the site;
- a preliminary sketch of the dam (zoning, slopes, cutoff trench excavations, etc.) and an opinion on foundation treatment and if necessary treatment of the reservoir basin.

Geological and geotechnical studies in the tender design stage

For most small dams, this phase is combined with the preliminary design phase.

Objective

Except in special cases, geological aspects are familiar at this point, and only extremely localised site investigations will generally be necessary (in particular on foundations of ancillary works such as the spillway, outlet tunnels, diversion tunnels, inspection adits, intake tower, secondary dams if they were not considered in the preliminary design).

However, if there is any significant change in the dam layout (or just the axis of the watertight system) since the preliminary design site investigations, new core sampling and water tests will be necessary on the new layout.

On the other hand, in this stage of study most of the geotechnical studies proper (soil and rock mechanics according to the case) are necessary:

- taking samples in numbers suited to the size of the dam and the conditions encountered (geological complexity and variability of the soils), in the foundation and in the planned borrow areas;
- laboratory tests (with a schedule tailored to give good knowledge of the foundation for the structures and the borrow areas). Their purpose is to give the designer the necessary elements for tender design of the dam and to enable recommendations to be made on geological and geotechnical aspects during the works and later monitoring of the dam throughout its life. In France this stage of study for dams 20 metres high or higher corresponds to the *final brief* that must be submitted to the Permanent Technical Committee on Dams.

Methodology of the tender design geological and geotechnical studies

Geological and geotechnical studies in tender design phases include all or some of the following elements, but are vary variable depending on the characteristics of each dam (size, complexity, type, etc.).

◆ Any additional site investigations needed on the foundations of the dam and ancillary works, in particular relatively cohesionless terrain, changes of layout, or insufficient previous investigations (because of land ownership problems for example), including trial pits, core drilling with water tests and/or undisturbed samples, in-situ testing (by penetrometer, etc.).

◆ Detailed investigations of borrow areas in trial pits with sampling for laboratory tests (in particular prospecting for new borrow areas if the volume of material available is insufficient, sometimes because dam volume is increased or in case of a late change in dam type).

◆ Geotechnical tests in the laboratory (soil and/or rock mechanics) on undisturbed and remoulded samples, for the foundation and construction materials.

◆ Chemical and radiometric analyses of materials or foundation soil.

◆ Geotechnical (and geological where necessary) summary on the foundation and materials from the borrow areas, leading to stability calculations to define a dam cross-section for an earthfill dam or the foundation level for a rigid structure.

◆ Final choice of axes (main and ancillary structures, watertight system), of the type of structure, of the materials to build it.

◆ Precise definition of the nature and form of the watertight systems, dam zoning (where appropriate), borrow areas, conditions for material placement, monitoring systems for the foundation and the dam.

◆ Recommendations for works supervision and monitoring of the dam in operation (precautions to be taken, in particular stability of river banks and excavation slopes during the works and when the reservoir is emptied; control of water content when building fill dams; hold points requiring the geologist's release before works are continued; detailed measures to be decided depending on what is observed during the work; etc.).

Geology and geotechnics in the construction phase

This phase essentially involves drafting the geological and geotechnical parts of the *Particular Technical Specifications*[1] in the construction tender documents (which may be for earthworks, preparatory works for civil works construction, grouting, diaphragm wall, monitoring, etc.).

The *Particular Technical Specifications* are most notably intended to make perfectly

1. Chapter IV (p. 101) gives information on how to draw up such specifications for a fill dam.

clear to the contractors tendering for the works any special points that should be taken into account in making their bids, and especially in carrying out the work. They include various documents to inform the contractors about hydrology, geology, and the results of site investigations.

The special points that should appear in the *Detailed Technical Specifications* are as follows and include, but are certainly not limited to:

- conditions for sorting, selecting and using fill materials, compacting standards (range of water content and compactness, limit values for the degree of saturation and/or pore pressure, gradingcurves, techniques and instructions for inspection of materials, etc.);
- criteria for hold points and acceptance of excavations;
- intended slopes after excavation (excavations and borrow areas);
- particular specifications concerning grouting, where appropriate (depth, pressure, holding criteria, e.g. pressure/volume, etc.);
- particular specifications concerning the diaphragm wall, where appropriate (hold criteria, keeping samples, precautions concerning piezometry, etc.);
- particular specifications concerning the drainage curtain, where appropriate (orientation of drillholes versus dip, spacing, depth, treatment of the cut-offs);
- specifications concerning piezometers (nature, sensitive areas requiring special surveillance, etc.).

Geological supervision during construction

Objective

The participation of the person who did all of the preliminary geological studies and was involved in setting up the design is vital as the dam is being built.

In fact, changes to details or sometimes even more important aspects may be required at any time during the work as the preliminary site investigations, even if they are very detailed, really only concern a very small part of the terrain involved. This is all the more true when the geological and geotechnical parameters are highly variable.

It is important to note that the construction phase is an integral part of the studies, because it is only at this moment that geology can be seen life-size and continuously and that any problems or important elements that may have been overlooked in the site investigations can be detected. Account must be taken of them through adaptations or modifications to the initially planned construction techniques, for example by deepening the watertight systems in places, stripping off cohesionless ground, adapting the zoning of fill, adding drains, adapting the depth of piezometers, etc.

Such modifications, which must sometimes be decided on very quickly on the construction site because of the often swift pace of construction, must be notified in good time to the contractors concerned, in writing (work order, inspection report, mention in a site log) with drawings and sketches.

They must then be filed with the *As Built Documents, as the History of construction* that is highly useful when the origin of problems or abnormal behaviour must be investigated several months or years after the work is finished. This recommendation is just as valid for the smallest dams.

Methodology for geological work supervision

To meet this objective, the geologist must be involved in construction through periodic visits, some of them scheduled according to works phases that require his assistance (acceptance of excavations, etc.), and the rest on a regular basis to be determined according to the characteristics of construction (dimension of the structures, geological complexity, contractor's experience).

It is therefore important that the geologist be kept informed in good time of how the work is progressing in order to be able to schedule his visits and avoid:

- not being able to carry out the planned observations and acceptance;
- delaying the works (works stoppage or filling in excavations, etc.), which is always a difficulty and creates conflicts that usually have an adverse effect on the quality of the work.

A non exhaustive list of the tasks to be accomplished during construction of a dam is as follows:

- *Supervising excavation of all kinds (abutment contact, cut-off trench, surface stripping, tunnels excavated in the rock, etc.):*

  - comparison with plans, decision to stop or continue the excavation (adapting the level where the excavations are stopped to actual geological conditions);

  - monitoring the stability of excavations, river banks and slopes (excavated or backfilled), if necessary including the decision to reinforce (pinning unstable boulders, gentler slopes, nailing or shoring up loose slopes, etc.).

- *Geological survey of excavations (abutment contact, cut-off trench, surface stripping, ancillary concrete structures such as tunnels, spillway or intake tower):* this is generally done for structures of a certain size when the excavations are contractually taken over as foreseen in the contract and can involve photography, sampling, plotting crack patterns, and precise surveying of particular points visible in the excavations.

- *Geological survey of tunnels excavated in the rock (generally restricted to the largest of the structures dealt with herein):* representation of the nature of the terrains and their structure (cracking, dip, schistosity, porosity) on a detailed drawing of the tunnel's cross-section, indicating all of the hydrogeological elements (cavities, inflows of water, leaks, etc.).

- *Supervision of sampling of construction materials for earthfill dams:*

  - comparison of the terrains actually encountered with plans and inspection that borrow material is in compliance with the contract specifications;

  - search for new borrow areas if necessary;

- adapting to operating conditions: sorting materials, zoning fill, treating materials by drying, wetting, screening, etc.;
- monitoring the stability of the remaining slopes and adapting operation of the borrow areas in case of problems.

◆ *Supervision of how aggregates, borrow areas and quarries are run (for RCC, riprap, etc.):*
- comparison with plans;
- inspection of conformity of extracted materials to contract specifications (grain size or block size, nature, shape, mechanical characteristics, etc.).

◆ *Supervision of drainage and draining shaft construction in the foundation.* Attention must be concentrated on:
- checking that the zones to be drained or decompressed are actually and correctly crossed (supervision of drilling, examination of cuttings, recording parameters, logs, etc.), and if necessary adapting the depth and orientation of the drains;
- the absence of siltation during drilling (no use of drilling slurry, careful cleaning at the end of the drilling until clear water is achieved);
- development of the pits;
- conformity of filter devices (size, nature and layout of the strainers) to the specifications and to the conditions actually encountered, and adapting them if necessary;
- proper isolation of any zones that will not accomodate flows of water;
- final check on functioning (which may happen only after partial or total filling of the reservoir), including in particular no entrainment of soil particles in the collected water (which might mean the beginning of piping).

◆ *Supervision of grouting in grout curtains (if any):*
- definition, in liaison with the contractor's specialists, of the size of the test section and grouting instructions (limit pressures and/or volumes, hold criteria);
- establishment of reconstituted geological cross-sections by recording the parameters in uncored drilling;
- possibly modifications in the depth of the curtain (deepening at permeable faults, etc.), or its length to each side (extending wing cut-offs);
- possibly modifications in spacing between drillholes, grout mix and sometimes type, grouting parameters (pressure, volume, flow);
- definition and supervision of investigation and inspection drilling including water tests;
- where necessary deciding to re-treat the foundation in case of poor results after tests;
- summary and interpretation of grout consumption and grouting pressures.

◆ *Supervision of construction of diaphragm walls (if any):*
- establishment of the cross-section of the land crossed by taking samples from the bucket, if the type of tool used and/or the nature of the terrains permit;
- acceptance of the excavation (taking a sample by bucket at the end of excavations), especially when a particular socketing stratum must be reached;

- special attention to monitoring the continuity of the wall;
- surveillance of the stability of walls, of the width and verticality of the excavation (by observing how cables or rods are centred with respect to the sides of the excavation as the bucket goes up and down);
- inspection of any overbreaks (which are visible on the concreting curves, in the case of plastic concrete diaphragm walls where the excavation slurry is replaced with plastic concrete);
- monitoring of groundwater level on either side of the wall during the work (it must be constantly ensured that the slurry or grout in the excavation is at least 1.5 to 2 metres lower than the surface of the water table in the natural ground in order to guarantee stability of the walls);
- any decisions to modify the structure's geometry (increasing or decreasing depth and sideways extension) according to the geological conditions actually encountered.

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CHAPTER IV

Fill dams

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Thousands of small fill dams ranging in height from only a few metres to some thirty metres have been built in France and others countries, and there are still many such projects to be built. These are relatively modest dams and are usually located in rural areas. Most are earthfill dams and their main roles are low flow support, irrigation, flood routing, recreation and/or drinking water supply.

Because of the number of such projects, the chapter on fill dams is more extensive than that on concrete dams.

TYPES OF FILL DAMS

The major types of fill dams are:

- *type 1:* homogeneous earthfill dams, built of watertight materials;
- *type 2:* zoned dams with an upstream shoulder or central core providing watertightness;
- *type 3:* dams of permeable materials (sand, gravel, pebbles, pit-run materials) with a man-made watertightening system.

Rockfill dams (type 2 or type 3) will be mentioned, but more briefly, as this technique is rarely used for small dams.

Geotechnical studies

This section is complementary as concerns earthfill dams to the information in Chapter III *(p. 37)*, which the reader is invited to consult. It focuses essentially on interpretation of test results.

Table 1 summarises the geotechnical studies required for dam design as concerns the foundation and borrow areas. But in addition to the search for materials, other studies may be required on the reservoir basin, including its watertightness, where that problem cannot be solved at the dam site, and the stability of the banks, which must be checked in some cases.

It is vital to emphasise the importance of a very simple test, especially for homogeneous dams, which is a test of moisture content. This is the basic test which, when considered in association with observations made during sampling to determine the nature of the material (clay, silty sand, gravel, etc.) and its state (dry, very wet, etc.), will give a good first assessment of the site. The designer should not hesitate to carry out a number of such tests, which are inexpensive (for example in borrow areas), by taking a sample every 0.5 to 1 metre of depth in order to evaluate the humidity gradient.

In general, the moisture content of materials varies little in the course of a year, except at the surface to a depth of about 1.5 metres where the materials may be dry or wet, depending on atmospheric conditions.

Both in the foundation and in borrow areas, samples are generally taken only in some pits, but they all play a role in description of the nature and state of the materials, depth of layers, and flows of water.

The mechanical and hydraulic tests on materials from borrow areas must be done with the moisture content that will prevail when those materials are placed. The shear strength of fine soils, especially in the short term under total stresses, falls significantly when moisture content increases.

Comments on the results from the main laboratory tests

Moisture content of fine materials

Materials from borrow areas must have a moisture content close to the optimum Proctor standard. However, before determining that value, it can be pointed out that when water contents are less than 10 or greater than 40, the materials can be considered as mediocre, in a first appreciation, and it may be difficult to use them for construction.

Particle size analysis

Materials containing more than 30% of particles smaller than 0.08 mm are probably watertight; with a content of less than 15%, they rarely are. In envisaging an artificial watertightening structure for a fill dam, it should be carefully checked that no materials suitable for construction of a watertight core are available.

<table>
<tr><th colspan="3">Site investigations</th><th colspan="4">Laboratory tests</th></tr>
<tr><th>Initial study</th><th colspan="2">Additional study on the foundation</th><th>Moisture content</th><th>Identification</th><th>Compacting construction materials</th><th>Behaviour</th></tr>
<tr><td>Pits dug with a mechanical shovel (minimum depth 4 m for investigations) to study the surface layers of the foundation and identify borrow areas. Use of an auger if some zones are inaccessible.</td><td>Corehole with LEFRANC water tests in loose foundations and LUGEON tests in rock with maximum pressure of 3 times future reservoir depth. Continuous pits in some areas.</td><td rowspan="2">Other in situ tests (penetrometer, shear vane, pressure meter, geophysical test, etc.)</td><td rowspan="2">On each sample taken (with a minimum weight of 2 kg)</td><td rowspan="2">Atterberg limits on sufficiently fine materials and particle size analysis (where necessary, organic materials content, methyl blue test, mineralogy, gypsum content).</td><td rowspan="2">(+ identification) Proctor standard and specific weight (sample weighing at least 20 kg).</td><td rowspan="2">(+ identification and + Proctor standard for borrow areas).
Triaxial consolidated undrained CU and unconsolidated undrained UU tests, compressibility for fine materials, shear box for coarse materials.
Permeability.</td></tr>
<tr><td colspan="2">Taking remoulded samples and also undisturbed samples from the foundation if the materials are fine.</td></tr>
</table>

Table 1 - Geotechnical analyses for a fill dam

Atterberg Limits

Fine materials with a plasticity index over 35 pose problems not only of stability but also of settlement, swelling, and placement.

Proctor standard

At the optimum of standard Proctor compaction, the degree of saturation is generally between 80 and 90, and the preconsolidation stress is between 100 and 200 kPa in the vast majority of cases.

The moisture content, when materials are placed, should not be more than 2 or 3 percentage points from the optimal moisture content (on the dry side or on the wet side) and even sometimes less.

Unconsolidated undrained triaxial tests

These tests give what are called undrained characteristics, i.e. cohesion c_{uu} and internal friction φ_{uu}.

In the foundation, if the most mediocre loose layer (in general saturated or almost saturated and therefore φ_{uu} = 0 or very low) has an undrained cohesion value c_{uu} of 20, 40, 60, 80, or 100 kPa (figures above 100 kPa are very rare), a dam with a height of 5, 10, 15, 20, or 25 metres can be built according to the case without any significant widening of its base.

For fill, the wetter the material (versus optimum of standard Proctor compaction), the lower shear strength will be, and c_{uu} of the order of 50 kPa (with $\varphi_{uu} = 0$) sets a limit on use of the material. On the dry side, c_{uu} will often be higher than 100 kPa and φ_{uu} higher than 10°.

Consolidated undrained (or drained) triaxial tests

These tests give characteristics of the grains, i.e. cohesion c' and internal friction φ'.

Undisturbed or compacted fine materials rarely show cohesion c' and internal friction angle φ' outside the following limits: 5 to 30 kPa for c' and 15° to 40° for φ', (i.e. a range of 25 kPa and 25°). The nature of the materials has a major impact on those figures. Characteristics may be found that are either very mediocre (e.g. c' = 10 kPa and φ' = 20°) or excellent (e.g. c' = 25 kPa and φ' = 35°).
It should however be noted that estimation of c' is much less precise than that of φ'.

Choice of an earthfill dam

If there are fine soils of satisfactory quality and sufficient quantity (1.5 times to twice the volume required) available on site, the homogeneous earthfill or pseudo-zoned dam alternative (type 1) imposes itself as the most economical alternative. The pseudo-zoned dam is a variant of the homogeneous dam type, which consists in distributing materials in the dam body according to their grading or their humidity, without requiring filters to separate them, so there are no true zones with precisely determined limits. For example, a homogeneous dam may be built with the fine materials placed upstream and the coarsest ones downstream, or with the wettest materials placed in the centre. Pseudo-zoned dams are provided with the same drainage as homogeneous dams, but the drain does not constitute a separation between two zones that are considered to be different, unlike most true zoned dams.

If only a limited quantity of fine materials are available, but there are enough usable coarse materials, construction of a zoned earthfill dam (type 2) may be considered with a core or upstream shoulder to provide watertightness and coarse shells for stability. However, this alternative has the disadvantage of requiring placement in zones, which will be all the more complex and costly if the site is narrow, handicapping the movement of equipment. Another disadvantage is the requirement to separate different zones with transition filters. On the other hand, especially for fairly large dams, the use of coarse materials may enable slopes to be steeper.

If there are no fine materials suitable for watertightening, or if quarrying in a highly heterogeneous borrow area with fine to coarse materials is too complex, a man-made watertight structure may be used (type 3). Two techniques are particularly suitable for small and medium sized dams: geomembranes and diaphragm walls.

Adjusting the characteristics of materials available on the site by screening, adding bentonite, drying or wetting, may be envisaged. But changing the moisture content of a material with a high clay content is both difficult and costly. It requires careful and constant inspections, which can be a problem for small dams.

Dam foundation treatment

The dam foundation must always be stripped to a depth of at least 0.50 metre to remove topsoil.

The mechanical characteristics of loose materials in the foundation (alluvium, colluvium, eluvium) are often sufficient to support a fill dam less than about 10 metres high. An in-depth study must be undertaken for fill dam heights over about 20 metres. When there is a problem with the stability of the foundation *(see stability analysis below, p. 82)*, the solution consists either in deeper excavation or a considerable widening of the fill dam's base. It is rare to install a drainage system to accelerate consolidation of the loose foundation.

A filter may have to be placed at the fill dam/foundation interface because of the nature of the materials. The rules governing such practice are given under *Filters and drains below, (p. 77)*.

Settlement of a loose foundation due to the weight of the fill dam is evaluated by compressibility tests. After construction, it should generally not exceed 5% of the total thickness of the compressible layers.

Watertightening and drainage systems must be installed to achieve an acceptable leakage flow and avoid any risk of piping (retrogressive internal erosion) and uplift on the downstream side.

Foundation watertightness

The three following cases can be considered for type 1 and type 2 dams (homogeneous and zoned):

◆ *foundation consisting of relatively impermeable materials:* it is recommended that a cut-off trench be built of compacted clay materials in order to deal with any surface cracking or heterogeneous zones. The dimensions of such a trench should be:
- minimum width at the base 3 metres (width of the machinery);
- side slopes of the order of 1/1;
- several metres depth with a minimum of 2 metres below natural ground level *(see photo 1 p. I)*.

◆ *foundation with permeable layers that are not revealed beyond a depth of several metres:* the trench must cut through those layers and be anchored in a watertight layer.

If the latter is unaltered rock, after it has been cleaned and possibly its surface has been smoothed, a first layer of wet (optimum moisture content (OMC) + 2 or 3) clay a few decimetres thick is placed to guarantee good contact; it may be necessary to set a filter between the downstream face of the trench and the permeable foundation materials.

◆ *permeable foundation to a significant depth*: grouting can be used both for a loose foundation and for a more or less cracked rock mass, with the grout adapted to the material being treated (bentonite-cement grout, specially designed grouts); the cut-off will usually involve three lines of staggered drillholes; as grouting cannot be effective at the surface, either the first few metres of grouting are relayed by the cut-off trench, or treatment is started at a certain height in the fill. Another watertightening technique is the diaphragm wall made of self-hardening bentonite cement grout or plastic-concrete; this solution is quite conventional for loose ground but can also be used in rock foundations with the more costly technique of the hydrodrill. It can cause problems if major displacements occur, for example the fill may be punctured in the event of severe settlement of a soft foundation (with the parallel risk of excessive lateral friction); if it is located at the upstream base of the fill, the diaphragm may suffer considerable shear at its top.

As concerns type 3 dams (with an artificial watertightening element), the connection between the watertightening structure in the fill and watertightening in the foundation is a difficult point, except in cases where a diaphragm wall provides all watertightening from the dam crest (*see under that heading hereafter, p. 82*).

When the reservoir cannot be watertightened by a cut-off at the dam, the solution consists in sealing the reservoir basin totally or partially with a geomembrane *(see that heading below, p. 81)* or with a blanket of compacted clay materials (at least two layers about 0.20 metre thick each), with the latter protected from any risk of drying out. Such techniques always result in a high price per cubic metre of water stored.

As concerns the support for these systems, it is necessary to:

- meet filter conditions (*see Filters and drains below, p.77*) for an upstream blanket;
- eliminate any rough areas that might puncture the geomembrane;
- avoid any risk of uplift, in particular due to gases under the geomembrane.

Foundation drainage

For drainage of flows from the foundation, the most satisfactory solution consists in placing a drainage blanket at the base of the downstream shoulder, at the fill-foundation contact, leading to the vertical or sloped drain in the central part of the fill (*see Fill drainage system below, p. 78*).

This blanket, which may be compartmented in order to determine the behaviour of each different zone, should be placed for any large structure ($H^2\sqrt{V} > 700$). For smaller dams ($H^2\sqrt{V} < 700$), where geological conditions permit, the drainage blanket may be reduced by placement of draining strips (in particular in the areas judged to be the most vulnerable in the river banks).

It can only be completely eliminated for very small dams where $H^2\sqrt{V} < 100$, provided that the foundation is sufficiently watertight. The thickness of the layers must be sufficient to discharge the foreseen flow with a minimum thickness for each horizontal granular layer of 0.20 metre (drain and filter).

If there is a relatively impermeable surface layer in the foundation, covering a much more permeable layer with its upper surface at a depth of less than H/3, it is recommended that decompression[1] wells (in general with piezometers) be drilled at the downstream base of the dam, spaced 10 to 25 metres apart. The decompression wells must be protected with a filtering material from the surrounding relatively impermeable material.

Fill dam design

Depending on what usable materials are available, the fill dam will be one of the three types specified above, *(pp. 67 and 70)* homogeneous, zoned, or with man-made watertightening.

Croos-section, crest width, freeboard

Maximum recommended slopes are 1/2. However, steeper slopes may be designed when coarse materials without fines are used (gravel, pebbles, boulders).

The crest width L is generally determined by one of the following formulas where H is the height in metres:

$L = 1/3\ H$
$L = 1{,}65\ H^{1/2}$
$L = 3{,}6\ H^{1/3} - 3$
where L = 3 m minimum, in order to permit movement of machinery.
Compacting the last layers may require a larger width.

Crest width also depends on how that crest is to be used, e.g. for a road. For zoned dams, it may also depend on the number of zones at the crest.

The following minimum widths are proposed:

$H^2\sqrt{V}$	< 100	100 to 300	> 300
L minimum	3 m	4 m	5 m

Table 2 - Minimum crest width

1. Decompression wells must be accessible for cleaning.

As concerns freeboard R, i.e. the difference in elevation between maximum water level MWL (corresponding to design flood) and the crest to avoid overtopping of the fill dam by wave action, there are several formulas, in particular based on wind velocity U and fetch F, which make it possible first to calculate the height of the waves h, considering the worst wind direction for the pair F, U. It is proposed here to use the Bretschneider[1] formula, which is suitable for small reservoirs (with a surface area < 100 ha). This formula considers the depth of water D in the vicinity of the dam (*see table 4*). The value of U is the 30-year wind velocity lasting one hour. The speed of wave propagation v can be evaluated using the Gaillard formula:

$v = 1.5 + 2\,h$ where h is expressed in m and v in m/s.
Freeboard is taken as $= 0.75\,h + v^2/2g$ where $g = 9.81$ m/s/s.

A minimum freeboard for fill (providing a safety margin from maximum water level, settlement and upstream-downstream cracking of the crest) is recommended depending on $H^2\sqrt{V}$, the minimum being taken as $(H^2\sqrt{V})^{1/4}/4$. Results are given in table 3. Of course, if calculations using the Bretschneider and Gaillard formulas give a higher value for freeboard, that higher value should be used. In such a case, a less rigid wave wall (e.g. gabions) can provide protection between minimum freeboard (table 3) and calculated freeboard.

When the fill is zoned and has a watertight core with a permeable upstream shoulder, the core should rise at least to the elevation of [MWL + 0,5 R_{min}].

$H^2\sqrt{V}$	5	30	100	300	700	1 500
R minimum in m (fill)	0.40	0.60	0.80	1.05	1.30	1.55

Table 3 - Minimum freeboard R for fill dams according to $H^2\sqrt{V}$

Account must be taken of settlement of the foundation (*see Foundation treatment above, p. 71 ...*) and settlement of the fill, which essentially occur in the first few months or years after construction. A countercurve (camber) must therefore be built at the crest; this will give R + countercurve that will decrease over time. For the fill alone, settlement after construction can be estimated at 1% of height (and even less for heights below 15 metres). An optical countercurve to improve appearance can be added. In the case of a zoned dam, the core must have the same countercurve.

1. See *Bibliography*, reference 1, p. 111.

Protection of faces and crest

Placing a layer of gravel on the crest will in particular avoid the formation of ruts due to traffic and desiccation of the last layers of compacted clay materials.

The dimensions of the protection for the upstream slope (including the filter blanket) should take into account the effect of waves and the type of protection selected. ICOLD Bulletin 91, published in June 1993, specifically deals with protection of upstream slopes for fill dams and the reader is invited to consult it.

That bulletin gives rules for dimensioning rip-rap protection (median block mass, grading, thickness of the layer, thickness and grading of the filter blanket, material quality). It also describes in detail specifications concerning protections made of:
- soil-cement;
- cast-on-site concrete slabs;
- pre-cast concrete blocks;
- bituminous concrete.

Wave action depends essentially on reservoir size and geographical location (wind rose). The choice of type of protection and its dimensions are therefore independent of dam height. From this standpoint, small dams cannot be considered as special cases unless the reservoir surface area is small.

Wave height h is calculated as indicated above and in table 4 hereafter.

U: wind velocity (m/s)
D: depth of water (m)
F: length of fetch (m)
g: acceleration due to gravity (m/s²)

$$h = 0.26 \, \mathrm{th}\left[0.578 \left(\frac{g\,D}{U^2}\right)^{3/4}\right] \mathrm{th}\left[\frac{0.01 \left(\frac{g.F}{U^2}\right)^{1/2}}{\mathrm{th}\left[0.578 \left(\frac{g\,D}{U^2}\right)^{3/4}\right]}\right] \frac{U^2}{g}$$

U	20					25					30					35				
D \ F	300	600	1000	2000	3000	300	600	1000	2000	3000	300	600	1000	2000	3000	300	600	1000	2000	3000
5	0.28	0.39	0.50	0.67	0.78	0.35	0.49	0.61	0.81	0.94	0.42	0.58	0.73	0.96	1.10	0.49	0.67	0.84	1.09	1.24
10	0.29	0.40	0.51	0.71	0.86	0.36	0.50	0.64	0.88	1.06	0.43	0.60	0.76	1.05	1.25	0.50	0.70	0.89	1.21	1.44
15	0.29	0.40	0.52	0.73	0.88	0.36	0.50	0.65	0.90	1.09	0.43	0.60	0.77	1.08	1.30	0.50	0.70	0.90	1.25	1.50
20	0.29	0.40	0.52	0.73	0.89	0.36	0.51	0.65	0.91	1.11	0.43	0.61	0.78	1.09	1.32	0.50	0.71	0.91	1.27	1.53
25	0.29	041	0.52	0.73	0.89	0.36	0.51	0.65	0.92	1.11	0.43	0.61	0.78	1.10	1.33	0.50	0.71	0.91	1.28	1.55

Table 4 - Wave height h expressed in metres

Depending on wave height h, table 5 gives the recommended dimensions for classic rip-rap: thickness e of the rockfill layer (measured perpendicularly to the face) and diameter d_{50} such that 50% of the blocks by weight have a diameter equal to or greater than d_{50}. The dimensions of the largest blocks are limited to e. The smallest elements should not have a diameter less than 0.10 metre.

Wave height h (m)	Thickness e (m)	Block diameter d_{50} (m)
0.30	0.30	0.20
0.55	0.40	0.25
0.80	0.50	0.30
1.05	0.60	0.40
1.30	0.70	0.45
1.55	0.80	0.50

Table 5 - Dimensions of upstream rip-rap

The supporting layer for the rip-rap is intended to protect the fill from the hydrodynamic effects of waves and from erosion. For waves of a height less than approximately 1.50 metre, thickness will be 0.15 to 0.30 metre. The supporting layer should meet filter conditions (explained hereafter) versus the rip-rap (*see FILTERS AND DRAINS next page*). The supporting layer may be replaced by a puncture-resistant geotextile in cases where the fill material is fairly resistant to erosion.

At a small dam, the surface area of the reservoir is often very small when the reservoir is almost empty. Furthermore, the period in which the reservoir is at a low level generally lasts only a few weeks (e.g. at dams built for irrigation purposes at the end of the summer). In such cases, it may be possible to provide no protection for the lower part of the upstream slope. In this case, a berm should be installed at the base of the protected top part. That berm will serve as a support for the protective layer and will extend horizontally outward for at least one metre past that layer. The berm elevation should be at least 2 h (h = wave height) below normal reservoir water level.

Of course, partial protection of the upstream slope can be envisaged using the same technique for dams with little variation in water level, such as lakes for recreational purposes, diversion dams, etc. (*see photo 11-12 p. V*). In this case, protection by plant cover may be considered if wave height is less than 0.50 metre. A wide berm with a gentle slope (1/10) must then be installed in the drawdown zone, where suitable plant species will be planted. Of course, growth of trees must be avoided in every case.

For very small reservoirs (fetch of only a few hundred metres and good face orientation), it may be tempting to provide no upstream protection at all. It is always possible to take action after the face has deteriorated.

The downstream slope of a fill dam must be protected from the effects of rainfall runoff. Grass is practically always planted to this end on the downstream slope of small dams in mainland France (*see photo 14 p. V*). A layer of topsoil about 0.15 metre thick is placed by mechanical shovel and/or with a bulldozer in this case.

For fill dams over approximately 12 metres high, it is recommended to install an intermediate berm halfway up the downstream slope (*see photo 13 p. V*). For fill dams over 15 metres high, this recommendation practically becomes a requirement.

This berm offers two advantages:
- it limits the effects of runoff along the slope;
- it gives access to piezometers halfway up the slope, as well as to spread topsoil, to plant grass, and later to maintain the slope.

Planting grass on the downstream slope is made easier through use of synthetic or natural geotextile blankets in which seed, fertiliser, and a straw substratum are incorporated. Honeycomb geosynthetic mats can also be placed on the body of the dam; the honeycombs are then filled with topsoil. These techniques are advisable in a Mediterranean climate where periods of severe drought and intense storms make planting more difficult. In any case, all varieties of shrubs must be prohibited.

FILTERS AND DRAINS[1]

To control infiltration through the fill, a draining and filtering system must be installed.

Nature of the materials

As concerns material quality, the main tests to be done in addition to grading are the following:
- measurement of sand brittleness coefficient;
- Los Angeles test (impact) and Micro-Deval test (wear) for gravel;
- sand equivalent, methyl blue test (the presence of clay may cause cohesion and therefore cracking of the material), and organic matter content.

Limestone materials should be avoided.

The particle size criteria to be used are as follows (the first two are taken from Terzaghi's rules for uniform granular materials):
- $\frac{d_{15}\,F}{d_{85}\,M} < 5$ with F for the filter and M for the finest material that must be protected (the filter must prevent migration of particles);
- relatively uniform filters and drains (to avoid segregation and guarantee internal stability); recommended rule $2 < \frac{d_{60}}{d_{10}} < 8$;
- less than 5% of particles smaller than 0.08 mm and d_{15} of sand > 0.1 mm (applying the Vaughan and Soares formula $k = 0.35\ d^2_{15}$ where k is expressed in cm/s and d_{15} in mm, we obtain for d_{15} = 0.1 mm a permeability of $k = 3.5 \times 10^{-5}$ m/s, which is a boundary value for a draining material).

In the case of a very fine soil, the first criterion $d_{15}\,F < 5\ d_{85}\,M$ is not usable; it is then recommended to take a 0 - 5 mm sand and apply the other criteria, as long as the material is not dispersive clay (which is rarely found in mainland France).

1. See *Bibliography*, reference 2, p. 111.

In the case of a widely graded soil, with $d_{60}/d_{10} > 16$, the filter to be placed next to such material should be determined with the d_{85} of the lower part of the soil's grading curve after the change in slope (*see figure 1*).

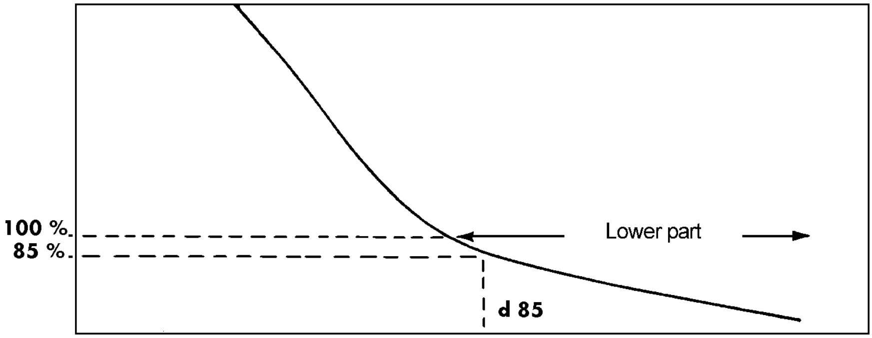

Fig. 1 - Example of a widely graded soil

Fill drainage system

A drainage system for a homogeneous fill dam consists of two parts (*see figure 2*):

- A continuous vertical chimney drain, of 0 - 5 mm sand, from the base of the fill up to normal reservoir water level + 0.20 to 0.30 metre, to avoid any risk of flow above it, under the crest near the downstream face; this drain is usually built by digging out the fill with a shovel every 5 or 6 layers that are compacted and carefully pouring the sand in (*see photo 2, p. I*);
- A finger drain discharging downstream, if possible independent of the drainage blanket or draining strips (*see above Foundation drainage, p. 72*), especially if the fill is not made of highly watertight materials; the finger drain will consist of lines of granular materials (in general gravel surrounded by sand or a geotextile) with a total cross-section that is more than sufficient to discharge the foreseeable flow. At small reservoirs where $H^2\sqrt{V} < 100$, these lines may be replaced by unperforated plastic pipes, with an external diameter of 100 mm (water collectors or similar) and minimum slope 1/100, spaced 25 metres apart; at least four pipes must be laid and connected to a perforated collector at the base of the chimney drain. The pipes must be carefully laid to avoid any risk of pipe sections coming apart or being crushed; in addition, installing a viewpoint at the downstream end of each blind pipe makes monitoring and maintenance easier.

It is proposed that the thickness of the chimney drain (0.50 m minimum) could be decreased as the fill is placed according to the value of $H^2\sqrt{V}$ corresponding to the lower level of the part in question (for a chimney drain with two or three thicknesses in all). Table 6 gives the recommended minimum thicknesses.

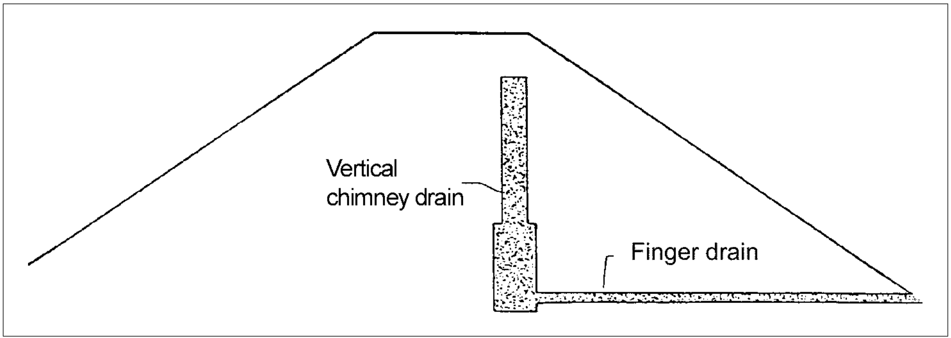

Fig. 2 - Drainage system for a homogeneous clayey fill

In general those thicknesses are very generous versus infiltration flows but give a safety margin over the long term against partial clogging by fines and/or carbonates. The nature of the fill materials may result in choosing even greater thicknesses. The available bucket widths must also be considered.

$H^2\sqrt{V}$	< 30	30 to 100	100 to 300	300 to 700	700 to 1 500
Thickness	0.50 m	0.80 m	1.00 m	1.20 m	1.50 m

Table 6 - Minimum thickness of a sand chimney drain
(H and V correspond to the altitude being considered)

Drainage system for zoned dams

The drainage system separates the watertight zone from the downstream shoulder. Its thickness will depend in particular on the number of layers required to meet filter conditions.

Use of geotextiles as filters or drains[1]

Geotextiles consist of fibres woven into strong, flexible permeable mats. There are several such products on the market, each ensuring one or several functions (filter, drain, protection from puncturing, etc.). Some applications have already been mentioned.

Of course, it is essential to avoid considering a geotextile as a general-purpose product (a geotextile that can support rockfill placement without tearing is not at all similar to a geotextile used as a filter).

In a dam, geotextiles are most often used to help in discharging percolation flows by acting as a protective filter for a granular drainage material (*see photo 9 p. IV*). However, installation of a geotextile to protect a chimney drain is no easy matter. In this case, an alternative solution consists in building the fill dam in two stages, in order to place the geotextile surrounding the drain on a sloped face that is stable in the short term (*see figure 3*).

1. See *Bibliography*, references 3 and 4, p.111.

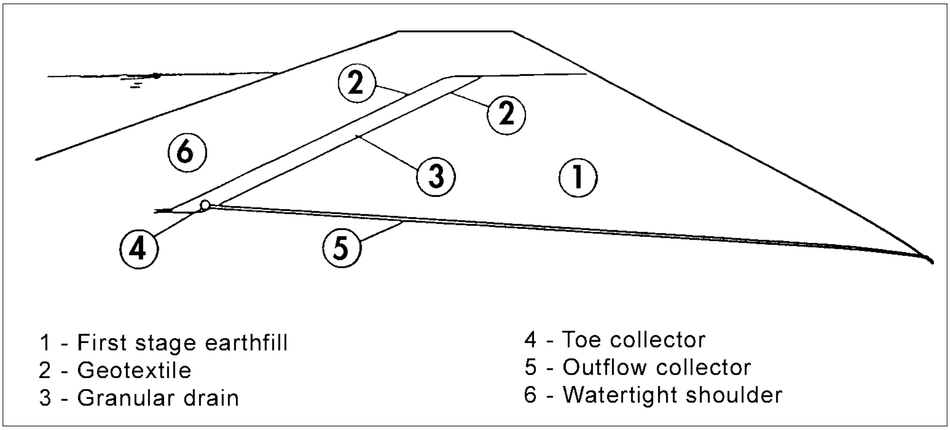

Fig. 3 - Sloped granular drainage material surrounded by a geotextile

There are also composite geotextiles capable of discharging relatively high flows in their plane. They then consist of a draining mat sandwiched between two filtering mats. Such a product can replace a chimney drain in a small dam as long as it is capable of discharging sufficient flow despite the decreased thickness due to the stress it will suffer in the fill. The geocomposite is then placed in zigzag fashion as follows (*see figure 4*):

- the fill is placed with the layers upstream compacted alternately with those downstream of the drainage system;
- when zone (1) is compacted, a first geocomposite panel is installed and connected to the collector at its foot, with the extra length folded towards the upstream;
- zone (2) is compacted, and then the extra length of geocomposite is folded back towards the downstream;
- zone (3) is compacted, and so on.

As experience with this technique to date is limited to very small dams, it is not recommended to use it when $H^2\sqrt{V}$ is over 300.

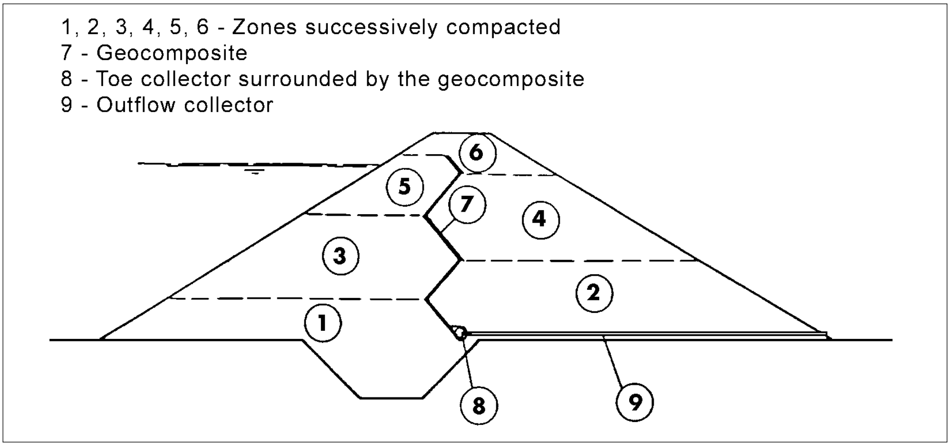

Fig. 4 - Vertical drainage by composite geotextile

MAN-MADE WATERTIGHTENING SYSTEMS

Geomembranes[1] (see photo 10 p. IV)

Geomembranes are thin, flexible, continuous watertight products at least one millimetre thick. The products on the market vary widely, with the main categories being:

- bituminous geomembranes (blown bitumen or bitumen modified by addition of polymers);
- plastomer geomembranes such as PVC (polyvinyl chloride), HDPE (high-density polyethylene), etc.
- elastomer geomembranes such as butyl, EPDM (ethylene propylene diene monomer), etc.

Good design and careful installation can guarantee satisfactory integrity of such products over time; the first practical applications of them date from the end of the 1960s.

Geomembranes are manufactured in a factory and delivered in rolls measuring several metres in width or in shop-assembled panels measuring 200 to 1000 m^2. The rolls or panels are connected together on site by welding or gluing. This must be done with great care and in compliance with certain rules depending on the products (e.g. temperature limits). On a dam face, horizontal joints must be prohibited because of the loss of strength versus the tensile stresses that could develop.

As concerns support for the geomembrane, two elements must be considered:

- risk of puncture or tearing due to an excessively aggressive support, either during placement or later under the weight of the water; in addition to efforts to improve the surface of the granular backing, a protective geotextile is often added, whether it is separate from the geomembrane or incorporated into it in the factory;
- risk of uplift when the reservoir is emptied due to leakage, which is always possible, and insufficiently permeable fill material; a continuous layer of good draining material under the geomembrane and outlets at the base of the dam must be provided to discharge water at the dam's downstream base.

For reservoir basins sealed with geomembranes, a system to discharge the gases that are likely to form under the geomembrane must also be installed.

Surface protection from ultraviolet radiation and from traffic on a geomembrane, wave action, floating debris and vandalism can be provided by rip-rap or concrete slabs. A transition is required to protect the geomembrane, and geotextiles fulfil this function very well.
It is important to verify stability against sliding at the various interfaces and avoid any tensile stress on the geomembrane.

It is not unfeasible to provide no surface protection for geomembranes at small dams, but this implies at least minimum surveillance; repairs are easy to make, but geomembranes will be more vulnerable, they will age more rapidly, and any efforts to pull a person or an animal accidentally fallen into the reservoir out again can be very difficult.

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1. See *Bibliography*, references 5 and 6, p. 111.

It is therefore necessary to fence in the dam. No protection on the geomembrane makes surveillance much easier.

It is advisable to anchor the geomembrane above the highest water level, at an elevation of at least MWL + 0,5 R_{min} (*see table 3 above, p. 74*). An appropriate junction must be planned both at the foot of the slope to the foundation watertight system and at rigid structures such as the spillway outlets, etc.

When the available materials are semi-permeable to fairly impermeable, it can be an advantage to place the geomembrane alone inside the upstream slope; the upstream shoulder, which must be sufficiently thick to deal with uplift, can then be placed carefully over the geomembrane, both sides of which must have sufficiently high friction; any leakage is then controlled with a classic chimney drain and the geomembrane ultimately serves the role of an additional watertighten element.

Diaphragm wall

The general principle consists of installing a central diaphragm wall once the fill is completely placed. The wall will run through all of the fill and the loose part of the foundation in the form of a trench 0.80 to 1 metre wide filled in with a self-hardening slurry (bentonite, cement, water).

Use of this technique is only possible if the material around the diaphragm wall is not too permeable, so that excessive losses of the slurry can be avoided, eliminating the corresponding risk of collapse of the trench. This may mean building a pseudo-core out of fairly fine, continuously-graded material at the centre of a dam built of coarser materials.

This type of artificial watertightening can be advantageous when compared to geomembranes if the dam abutments are very steep, as foundation treatment work is then very complicated.

However, the fill dam must be designed to support flooding without damage before the diaphragm wall is built. The temporary diversion must therefore be dimensioned in such a way that the fill is never subject to pressure. If this is not possible, filters and drains must be generously dimensioned to avoid any internal erosion during a construction period flood.

Stability analysis

Actions

Stability analysis of a fill dam can be broken down into two parts:
- determination of the forces exerted on the dam;
- analysis of combinations of those forces; of those combinations, the worst case scenarios are considered in terms of the envisaged failure mechanism.

Determination of forces

- *permanent force:* dead weight of the fill;
- *variable force:* pore pressure or pressure of reservoir water according to head on the spillway;
- *accidental force:* earthquakes.

Combinations of forces

In designing small and medium-size dams, three combinations of forces are usually considered:
- dead weight of the fill and pore pressures at the end of construction (a practically permanent combination);
- dead weight of the fill and pore pressure field induced by rapid reservoir emptying (frequent combination);
- dead weight of the fill and pore pressure field induced by the reservoir at full supply level (a practically permanent combination);

Where appropriate, earthquake forces (accidental combination), should also be considered.

Stability calculations

If there are no fine materials in the foundation and in the fill, the designer is confronted with one of two cases:
- the materials may be highly permeable and stability will depend on their internal friction angle φ;
- the materials may be semi-permeable and account must also be taken of the flow pattern and therefore pore pressures during rapid reservoir drawdown.

In the rest of this paragraph, we consider only cases where there are fine materials (excluding mud and peat), either in the foundation or in the fill, or in both. Fine materials generally result in using two types of calculations for slope stability:
- a short term calculation corresponding to stability at the end of construction, before consolidation, using characteristics determined by unconsolidated undrained triaxial tests interpreted with consideration of total stresses;
- a long-term calculation after consolidation and after a rapid drawdown operation[1] for the upstream slope, considering effective stresses using the characteristics determined by the consolidated undrained (or drained test in some cases) triaxial test.

Calculation methods for circular failure modes such as the FELLENIUS or BISHOP methods (the FELLENIUS method being the more pessimistic) are suitable in routine cases. Methods for non-circular failure modes such as the SPENCER method should be used for some zoned dams and when the foundation is partially (in one layer) or totally made up of weak materials.

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1. The calculation in "rapid drawdown conditions" is done assuming that emptying is instananeous, which is not very far from reality.

The selected profile should guarantee the stability of both dam slopes, in the short and the long term, with a satisfactory factor of safety, usually between 1.3 and 2.

Long-term stability (practically permanent load combination with the reservoir full or frequent drawdown combination)

When the reservoir is emptied rapidly, it can generally be considered that the saturation line is horizontal (normal water level) in the watertight zone of the fill (upstream slope or central zone) all the way to the chimney drain. This approximation has no significant impact on the results of the stability calculation, which is also relatively unaffected by variations in the unit weight of the materials. The factor of safety F mainly depends on the values of c′ and φ′[1] and therefore on how well they represent the material.

When several triaxial tests are done on a single type of material from a borrow area, its c′ and φ′ will be determined from the straight line of regression of all the failure circles represented by $(\sigma'_1 + \sigma'_3)/2$ and $(\sigma'_1 - \sigma'_3)/2$ (see figure 5). First a and α are calculated by adjusting the co-ordinate points $[(\sigma'_1 + \sigma'_3)/2, (\sigma'_1 - \sigma'_3)/2]$ for all of the triaxial tests considered. Then c′ and φ′ are deduced as indicated in figure 5.

A new approach is proposed for long-term stability calculation, in three stages[2]:

- *a first stage* consisting in studying, in a conventional manner, the stability of the slopes using the mechanical characteristics c' and φ' of the foundation and the fill (factor of safety F close to 1.5). But this calculation alone is not enough, as a single value for F can be insufficient in some cases and quite sufficient in others;
- a proposed *second stage* consisting in evaluating the influence of each mechanical characteristic on the factor of safety F with successive decreases in each c' by 10 kPa (with the boundary being c' = 0) and each φ' by 5°. Depending on the slope considered (upstream or downstream), the relative weight of each mechanical characteristic on F can be assessed; decreasing just one of them, by 10 kPa or 5°, results in F being decreased only slightly, by several hundredths, or more significantly, by several tenths, according to the greater or lesser influence of fill height and soft foundation thickness. This may mean that additional tests are considered necessary or that greater prudence is required in the choice of certain values. It can be noted in general that F is more sensitive to a 10 kPa decrease than to a 5° decrease for dams around 10 metres high and less sensitive when the fill height is around 30 metres;
- *a third stage* consisting in calculing F with a decrease in all mechanical characteristics by 10 kPa and 5°. The cross-section is considered satisfactory if F equal to or slightly higher than 1 for the upstream slope and 1.2 dor the downstream slope, which requires greater safety; if F is significantly higher than these boundary values the slopes may be steeper, while if F is lower than these values should be gentler.

Comment: these values of 10 kPa and 5° seem suitable for the following reasons:

1. c' (cohesion) and φ' (internal friction angle) are intergrain characteristics of the soil, obtained via the consolidated triaxial test interpreted in effective stress conditions.
2. See *Bibliography*, reference 7.

◆ they correspond to a realistic level of safety concerning the results of triaxial tests. Considerable prudence is nevertheless called for as concerns first the characteristics taken into account for a soft foundation (heterogeneous areas, difficulties in taking undisturbed samples, influence on F that is generally greater than that of the fill characteristics) and second the cohesion of undisturbed or compacted materials (especially due to overconsolidation), which is a difficult parameter to determine precisely whereas its importance is great, although admittedly less great as the dam is higher. It should be noted that interpretation of triaxial tests (selected failure criterion, alignment of failure circles) results in opposite trends in c' and φ', but when considering failure it is normal to seek for safety on both c' and φ';

◆ a simultaneous decrease by 10 kPa in cohesion and 5° in friction angles often implies changing the factor of safety from 1.5 to approximately 1;

◆ calculation of the third stage comes down to being stricter with cohesion than with friction angle; with routine values of c' from 10 to 20 kPa and φ' from 20 to 35°, classic calculations result in a decrease in c' by 3.5 to 6.5 kPa (c'/1.5) and in φ' by 6.5 to 10° (tanφ'/1.5); the recommended calculation therefore generally means a higher F in the classic calculation (first stage) for very small dams (5 to 15 metres) than for larger dams (20 to 30 metres); in addition, mediocre materials are more affected than good quality materials.

In some cases, values can be selected other than 10 kPa and 5°, in particular when mechanical characteristics are very poor or very good.

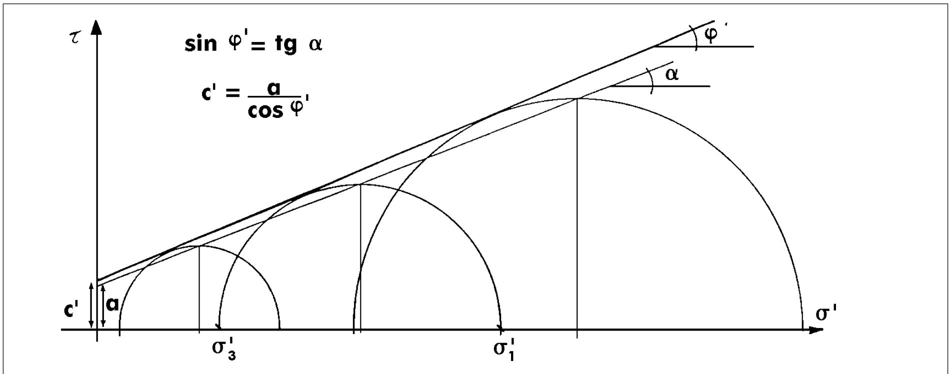

Fig. 5 - Relation between Mohr-Coulomb failure criteria and maximum shear strength

Short-term stability (practically permanent combination at the end of construction)

Problems of short-term stability for small earthfill dams are due either to relatively compressible materials in the foundation or to clayey materials in the fill compacted on the wet side of Protor Standard, or to both. These materials, saturated or close to saturation, have a shear strength that is limited to c_{uu} (φ_{uu} = 0)[1].

1. c_{uu} (cohesion) and φ_{uu} (internal friction angle) are what are known as undrained characteristics, obtained via the undrained unconsolidated triaxial test interpreted in total stress conditions.

Case of compressible foundations

For small dams on compressible foundations, the construction of side berms to a height of approximately H/2 (H = height of the fill) and several metres to several times H wide can be a more economical solution than stripping off the weak layers of the foundation.

Three types of failure must be considered: non-circular, circular and puncturing. In the case of a thin weak layer, only a non-circular failure calculation is correct; however, if a weak soft foundation thickness of at least H/3 is considered, roughly similar results are generally achieved for the three types of failure.

The advantage of calculating puncture failure[1] is that only shear strength in the foundation is considered, which is logical when the fill is considerably stronger than the foundation. The safety factor is:

$F = N_c . c_{uu} / \gamma . H$

where

γ and H, are the unit weight and height of the fill (even though crest width is relatively limited)

c_{uu} is the undrained cohesion of the foundation

N_c is a factor approximately equal to 4 + 0.5 L/D where L is average width of the fill (halfway up its height) and D the thickness of the soft foundation (minimum value of $Nc = \pi + 2$, i.e. approximately 5).

A calculation in effective stress conditions is only warranted for relatively high embankments. The work of LEROUEIL, MAGNAN and TAVENAS[2] has shown that pore pressure u increases only slightly in the foundation as long as effective vertical stress is less than preconsolidation stress (partial consolidation during the initial construction phase).

For the great majority of small dams on soft foundations with little strength in the short term, a stability calculation with c_{uu} is satisfactory: it is a fairly pessimistic calculation, provided that the weakest layer has been detected during investigations.

Case of fills consisting of wet clayey materials

Compacting materials close to saturation can induce the development of high pore pressures at the end of construction, even in the field of low stresses.
In addition to the calculation of total stresses using c_{uu}, a calculation under effective stress conditions using c', φ' and u must be done.

Pore pressure, whose role can be crucial even if it is difficult to precisely assess, can be expressed by $u = r_u . \gamma . h$ where r_u is a factor less than 1 and where $\gamma . h$ represents the total vertical stress due to the weight of the column of earth above the point considered. When r_u is close to 1, shear strength is reduced practically to c'.

1. See *Bibliography*, reference 8, p. 111.
2. See *Bibliography*, reference 9, p. 111.

The intensity of the pore pressures that are likely to develop can be evaluated in the laboratory from isotropic compression tests σ with measurement of u, which makes it possible to determine the factor B = u/σ. It should be noted that in the stress path equivalent results are achieved, σ being average stress $(\sigma_1 + 2\ \sigma_3)/3$.

Experimental tests have shown that slight increases in the degree of saturation (problems of overcompacting) or in moisture content w can result in significant increases in pore pressures. On the other hand, the value of c_{uu} decreases regularly when w increases.

It is noteworthy that near the face of a slope (especially at the dam base, at a berm or at a change in slope), the value of r_u is significantly higher than that of B, as the vertical stress is quite close to the minor principal stress σ_3. For example, if horizontal stress $\sigma_1 = 2\ \sigma_3$ and $\sigma_2 = \sigma_3$, the result is $r_u = 1.33$ B. While any development of pore pressure due to construction would be adverse for the downstream slope, for the upstream slope of a homogeneous dam values of B up to 0.35 are acceptable (the FELLENIUS method corresponds to rapid reservoir emptying at an r_u of approximately 0.40 to 0.45). The cross-section and structure of the embankment must be adapted to the pore pressures that are likely to develop.

It is interesting to carry out stability tests and calculations with materials compacted in the laboratory, first at a higher water content than that planned for placement and second at a high level of saturation, to measure the consequences. It is also possible to do a sensitivity study by varying the factor r_u, which would make it possible during the work to check if there is any risk of failure, using the pore pressure cells installed.

Accounting for seismic activity

Seismic activity on the site plays a role on three main levels:
- dam stability under the effect of an earthquake, possibly with an estimation of strains,
- special construction techniques for the foundation and the embankment,
- behaviour of ancillary works (spillway, intake tower, etc.).

Design earthquake and methods for dam stability calculation

The reference earthquake for a dam project is defined in principle in the course of the geological study according to seismic activity in the region (seismo-tectonic provinces, historic earthquakes, micro-seismic intensity, laws governing attenuation with distance, etc.). It is characterised by a maximum horizontal acceleration at the surface of the natural ground, which can be written in the form α.g (where g is acceleration due to gravity).

In present practice, the methods used to evaluate the stability of structures under earthquake conditions depend on the value of and the assumed sensitivity of the dam: height, foundation type (presence of saturated sand layers for instance), and make-up of the structure.

For small dams in areas with low to moderate seismic activity ($\alpha < 0.15$ to 0.20), it is generally enough to use the pseudo-static method. When the dam is more vulnerable and the earthquake greater, it is recommended to combine that method with more representative ones taking into account the dynamic behaviour of the soil (foundation and fill).

Pseudo-static method

This is the most widely used method for small and medium-sized dams.

The influence of the earthquake is represented by a horizontal seismic coefficient K_h that amounts to applying an additional horizontal driving force $K_h.P$ at the centre of gravity of the volume of earth with potential to slide and with a total weight P. The resisting forces mobilised are those estimated from static strength (as defined in the stability study with no consideration of seismic activity).

This notion is well suited to usual calculation methods that divide the volume of earth into vertical layers. In most cases no vertical seismic coefficient K_v is used, as the introduction of an additional force $K_v.P$ leads to an additional driving force in the case of acceleration directed downwards.

This method requires the choice of a seismic coefficient, which is always empirical. In principle, in calculating the stability of a slope K_h is taken as equal to $\alpha.\beta$ where β is a reducing factor defined from experience ($\beta = 1/2$ to $2/3$ in American and Japanese practice).

As a general rule, β can be set at 2/3 to 1 for frequent operating cases such as the permanent state, and at 1/2 to 2/3 for other cases (end of construction and drawdown)[1].

Recommended minimum rules for the factor of safety F_{st} in earthquake conditions are generally $F_{st} > 1.1$ in the permanent state and $F_{st} > 1.0$ in other cases.

Note:
The influence of an earthquake on the static pressure due to the water in the reservoir is often calculated according to the Westergaard theory (relative to a vertical barrier, and therefore more appropriate for concrete dams) with a reduction to take into account the sloped face. In the case of usual slopes (1V/2H to 1V/4H) and with the shallow depth of water stored behind the dams concerned here, this effect is practically negligible *(see Chapter 5: Stability analysis – p. 125)*.

1. See *Bibliography*, reference 10, p. 111.

METHODS USING THE DYNAMIC BEHAVIOUR OF SOILS

The pseudo-static method is not satisfactory on the theoretical level and does not give a correct understanding of problems when seismic activity is high.

The most suitable method in this case is that developed by H.-B. Seed, which consists in doing a study of the dynamic behaviour of the dam and its foundation during an earthquake. The dam is then considered stable when strains are limited and compatible with the properties of the materials used.

This study can be necessary if $\alpha > 0.2$ to 0.25 and for most important dams.

SPECIAL CONSTRUCTION TECHNIQUES FOR THE FOUNDATION AND THE FILL

The foundation can cause problems in the event of a strong earthquake when it contains materials that are likely to present a significant decrease in strength in relation with strong development of pore pressures (phenomenon of liquefaction of saturated loose sands) or high distortion (soft clays). As a general rule, these materials also pose problems in "static" design of the dam and may have been replaced or treated within the dam area. However, in this case it is still necessary to check the influence of whatever material has been left in place or has not been treated beyond the dam's upstream and downstream toes. In the case of sandy layers, classic treatment consists in building a mesh of vertical drains (to drain overpressures generated by the earthquake) or in improving the ground by densifying it (vibratory compaction, weighted columns, solid grouts, etc.).

For the fill, modifications may be necessary to deal with any internal problems and strains that may occur. Generally, a probable consequence of a strong earthquake is the appearance of concentrated leakage through the dam. Such modifications concern zoning of the fill, resistance of the materials to retrogressive erosion, and design of a crest. The following precautions are worth mentioning:

- avoid fine, cohesionless and uniform soils in saturated zones;
- provide a chimney drain or enlarge it in the case of homogeneous fills;
- design transition zones that are as wide as possible;
- be especially prudent with the thickness of filters (place a layer of sand upstream from the core to plug any cracks that may be caused by an earthquake);
- build the dam or its core with materials offering good resistance to internal erosion (plastic clays, sandy gravel with clay fines of a very continuous grading).

In addition, if the design earthquake is very strong, it may be wise to increase freeboard and crest width.

Monitoring system[1]

The present chapter only looks at aspects related to the design of a monitoring system for small fill dams, both new constructions and operating dams with no such system. The frequency of measurements is addressed in Chapter VII.

However, it must always be remembered that visual inspection is the most important part of dam surveillance: it is often the means to detect problems and anomalies that affect the dam. On the other hand, monitoring is a quantitative method based on the use of measuring instruments, selected and positioned to give information on how the dam's behaviour changes. The monitoring system must therefore be designed according to the type, the dimensions and the specific technical features of the dam. The system will be very simple for small dams and denser for medium sized ones.

For small dams, the monitoring system should consist of simple, robust and easy to read instruments.

Monitoring devices for small dams

Measurement of reservoir water level

This measurement helps in meeting three objectives:
- improving reservoir management through continuous knowledge of the volumes of water that are available;
- participating in dam monitoring by allowing examination of the influence of reservoir water level on measurements taken by certain instruments (in particular flows and ground water level);
- enriching hydrological data through measurement of flood flows.

Operation of a gated dam and improvement of hydrological data will sometimes justify installation of a recording water level gauge. In all other cases, in particular for the needs of monitoring, a staff gauge is perfectly suitable to measure reservoir water level.

Leakage measurement

Inspection of leaks, seepage and wet areas is mainly visual. Measurement of flows assumes that they be collected in a trench at the dam base or by placing outlets. Two ways of measuring flows can be considered:
- by capacity (measurement of the volume gathered in a gauged vessel during a given period);
- by measurement of the level upstream from a calibrated weir.

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1. See *Bibliography*, reference 11, p. 111.

These systems are installed on new dams at the outlet from drainage elements and on operating dams in areas where leakage is observed. Every effort must be made to ensure that the point of measurement collects all leakage flows as well as possible with no flow around the measuring point and, if possible, free of any influence of rainfall.

Weirs must be kept clean and the approach channels to weirs must be regularly cleared of any material deposited. If granular materials are observed, a specialised engineering firm should be alerted in order to study whether there is any risk of internal erosion.

Ground water and pore pressure measurements

It is important to check the position of the water table and how pore pressures are evolving in the foundation and in the embankment.

Measurement devices can be classified in two types:
- open tube piezometers with a length of slotted tube from a few decimetres to several metres;
- pore pressure cells (vibrating wire or counter pressure).

Pore pressure cells give precise localised measurement and offer faster response times than piezometers.

The open tube piezometer is cheap and easy to read and allows anomalies to be detected in the foundation (by revealing insufficient pressure drop), or in the downstream slope (problems of saturation). Piezometers are therefore installed in bank to bank profiles (being careful at the contact with the drainage blanket). Given the response time, the open tube piezometer is more suitable for permeable ground.

On the other hand, if more complex phenomena are to be monitored in detail (consolidation of wet clay materials in the foundation or the fill, progression of saturation, efficiency of a chimney drain, etc.), localised measurements of pore pressure grouped in a few upstream-downstream profiles will be preferred.

Displacement measurements

Several types of displacements can be measured:
- measurement of the absolute displacements of the dam's survey targets with respect to fixed benchmarks set up in zones that are not likely to be affected by any movement; altimetric measurements (settlement) and planimetric measurements (upstream-downstream and bank to bank) can be taken;
- measurement of internal displacements in the fill, vertical ones using settlement meters and horizontal ones using inclinometers or strain gauges; these devices are generally only used for large dams;
- measurements of relative displacements of concrete structures, with pendula in the intake tower or VINCHON gauges at joints in the inspection gallery.

The most common measurement is settlement, which in general changes very little after a few years. It is important to start settlement measurements as soon as the last layers of fill are compacted. The survey targets sealed into the dam body are positioned on the crest of the fill or near it and also on the downstream slope for relatively large dams.

CHOICE OF A MONITORING SYSTEM

It is obviously impossible to give strict rules for design of monitoring systems for small dams. The system must be adapted to the special case of each dam.

The monitoring system for a new dam should be planned in the preliminary design stage and placed during construction. It will necessarily evolve, and some devices may be deliberately abandoned after a few years while others may be added if any problem is revealed by visual observation or if there are any concerns about the dam's behaviour.

Instruments can also be installed on older dams that did not have such a system at the origin, but obviously installing a system during construction is preferable: it is of course much easier to install piezometer cells in fill during construction than in holes drilled afterwards.

In any case, a dam's monitoring system must be determined with the two following questions in mind:

- what are the important phenomena in dam behaviour and how do they evolve?
- how should these phenomena be measured?

The main changes that are likely to result in problems or in failures are globally of three types:

- settlement that is considerably greater than what was foreseen at the crest of the dam, which also means a decrease in freeboard;
- development of abnormally high pore pressures;
- leakage through the embankment or the foundation that is not controlled by the drainage system (too small or plugged) and which may, due to a sudden or progressive worsening, lead to piping or saturation in the downstream slope.

The flow of leakage must be measurable at the downstream toe of any dam. A ground water level inspection system should be planned for any reservoirs where $H^2\sqrt{V}$ is greater than 100 and surveying benchmarks must be installed for any where $H^2\sqrt{V}$ is greater than 300.

Water intake and outlet

For large earthfill dams, the water intake and release system is generally separate from the reservoir drawdown system. However, small and medium-sized dams often have a single pipe, made of steel (see phot. 16 to 18) or steel lined concrete in most cases, ensuring the following functions:

- discharge of water as the fill is placed, and discharge of the construction period flood;
- release of stored water, which corresponds to the main objective of this type of dam (low flow support, irrigation);
- emptying the reservoir for maintenance and especially the possibility of rapid emptying in a few days in case of hazard (for example sliding in a part of the downstream slope).

Relatively numerous incidents have affected drawdown systems at small earthfill dams. The main ones that have been observed, excluding problems with gates, are the following:

- significant scouring downstream (insufficient energy dissipation);
- corrosion of the steel (no protective lining);
- piping along large diameter reinforced concrete pipes (poor compacting around the pipe);
- failure at a joint (faulty connection between two sections) due to elongation of the pipe caused by a high strain in the fill.

This last case is by far the most frequent and concerns both steel pipes and PVC pipes with no concrete encasement.

Two types of recommendations are proposed in this chapter: first, recommendations that are common to all drawdown systems consisting of a pipe, and second, recommendations on a minimum set-up to be installed according to $H^2\sqrt{V}$. In certain cases, the selected system will therefore be larger (larger diameter to make the pipe inspectable, reinforced concrete gallery, additional gate, pipe in the inspection gallery, etc.).

These recommendations do not concern reinforced concrete galleries that are built in situ and with a cross-section that is larger than shop manufactured pipes; such galleries should be founded on relatively undeformable ground with Water-Stop joints between the various sections and the downstream part surrounded by draining materials.

Recommendations common to systems consisting of a pipe

For all drawdown systems, both an upstream protection to avoid any problem due to solid transport (raised strainer or tube, or one installed in a shaft, or a grate and rockfill) and an energy dissipation system downstream to avoid scouring will be required.

When $H^2\sqrt{V} > 30$, the selected diameter must be such that the volume of water can be halved in less than 8 days, calculated with no consideration of natural inflow[1]. In the case of large catchment areas, it should be checked that the pipe dimensioned according to this condition is capable of discharging twice the highest average monthly flow with the reservoir full. The selected construction period flood may also result in increasing the diameter. The pipe may be of PVC (water collector, maximum pressure in operation 1 or 1.6 MPa, diameter 160 or 200 mm), or steel (continuous protective lining inside and outside to withstand corrosion), or steel lined concrete (diameter $\geqslant$ 600 mm); reinforced concrete pipes or cast-iron pipes are not recommended. The pipe should have an upstream-downstream slope $\geqslant$1% and should be placed in a trench dug in the foundation at the lowest point of the valley for small diameters ($\leqslant$ 400 mm) or in sufficiently stiff ground in place (the foot of a slope in general) for larger diameters ($\geqslant$ 600 mm). The possibility of significant settlement should be avoided, especially differential settlement (if not, special joint systems must be designed). Special attention will therefore be paid to checking the pipe's foundation (no localised hard points).

The pipe must be totally encased in concrete dosed at 200 - 250 kg of Portland cement per m^3 poured in the entire excavation to protect it and guarantee good hold to the ground *(see photos 15 to 18 p. VI)*. The recommended concrete thickness[2] is about fifteen cm. It is not necessary to exceed that figure, which makes it advantageous to dig a trench with vertical walls to minimise the concrete volume. Embedding the pipe in concrete in this way has two functions:

- to improve the contact between the pipe and the ground, especially in areas where compacting is difficult;
- to avoid introducing pressurised water into the fill if, for any reason, the pipe should be punctured.

But in the case of a steel lined concrete pipe, the second risk is excluded.

Finally, one last precaution consists in placing draining or filtering granular material on either side of the pipe over its downstream third, in order to block any piping. The simplest technique is to use a drainage blanket or drainage strips in the fill *(see above)*.

When $H^2\sqrt{V} > 100$, it is recommended to do a watertightness test before embedding in concrete by plugging the two ends of the pipe and raising pressure up to twice the reservoir depth + 0.2 MPa, and then maintaining this pressure for 8 hours (the weak point will be at the joints).

Case-by-case recommendations (minimum system)

The minimum system concerns the type of pipe, its diameter, and the number and position of gates or valves. The recommended minimum measures are grouped in table 7 below.

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1. Calculation considering the possibility of discharge through turbines where appropriate.
2. See *Bibliography*, reference 12, p. 111.

The most widely used system for large dams consists in installing upstream two valves or gates. This avoids pressure in the pipe or gallery[1], which helps in inspection and reduces the risks of problems in the case where watertightness is not perfect. The guard gate generally remains open.

For dams of more modest dimensions ($100 < H^2\sqrt{V} < 1500$) it is acceptable to install a pressure pipe as long as it is embedded in concrete. In this case, a single guard gate is placed upstream and the regulating valve is placed downstream where it is easy to access and operate.

For smaller dams ($H^2\sqrt{V} < 100$) it is acceptable to have only one valve downstream. If any incident should occur on that valve, it will still be possible to take action by having a diver install an inflatable plug upstream or a membrane to obstruct the strainer.

$H^2\sqrt{V}$	Type of pipe	Diameter of the pipe in mm	Number and position of the valves
< 30	PVC or steel	160 or 200 PVC 200 to 300 in steel	one downstream valve
30 to 100	steel	300 to 400	
100 to 300	steel or steell lined concrete	400 to 600	one upstream guard valve and one downstream valve
300 to 700		600 to 800	
700 to 1 500		800 to 1 200	
> 1 500	reinforced concrete gallery		one upstream guard valve and one ups stream regulating valve

Table 7 - Minimum outlet works

Let us recall that the selected diameter should in particular permit rapid emptying when $H^2\sqrt{V} > 30$ *(see p. 94)*. Finally, whenever the designer wants the pipe to be inspectable, he should select a minimum diameter of 800 mm. This should be the case for strong design earthquakes.

Spillway

For small dams, the spillway often consists of a channel (or chute) with a free overflow sill upstream and an energy dissipator downstream in the valley bottom. For some larger dams, it may be more economical to opt for a tower at the upstream toe connected to a gallery under the fill, which by compartmenting that structure ensures the three functions of a shaft (or morning glory) spillway, water intake at several levels, and bottom outlet.

Calculation of the design flood and safety flood is addressed in Chapter II.

1. Whenever this is possible at negligible extra cost, it is better to avoid pressure pipes in fill.

Design of the free overflow sill

For a given flow, there are infinite solutions between:
- a very long sill, resulting in very low hydraulic head;
- a very short sill with high hydraulic head.

The general principle consists in assuming a given overspill length, calculating the head on the sill with account taken of attenuation in the reservoir, and possibly increasing or decreasing the length of the sill.

A maximum head on the sill of 0.50 to 3 metres, and more generally 1 to 2 metres, is selected.

Front or side inlet for a overflow spillway

An overflow spillway is called straight drop spillway if the direction of flow is upstream-downstream, and side channel spillway if the flow changes direction by 90° at the spillway *(see photos 19 and 20 p. VII).*

When the reservoir has a large surface area, it can provide good attenuation. It is then advantageous to store as much of the volume as possible temporarily and therefore have higher head and the smallest overspill length possible. In this case, the inflow is generally frontal (i.e upstream - downstream).

In contrast, a long overspill length will decrease the surface area that must be purchased or expropriated since the highest water level will be low. The cost of the fill is then reduced because the crest is lower, but of course the cost of the spillway will be increased. Freeboard gives greater safety for a flood greater than the design flood. Inflow is generally lateral, which often helps to reduce earthworks.

Discharge of floating debris

It is vital to have a safety margin for discharge of floating debris, especially for shaft or sluice spillways. If possible, this means avoiding having a walkway above a spillway with frontal inflow. If such a structure is selected, care must be taken to keep sufficient air gap during the design flood, which may mean moving the walkway towards the downstream side of the sill (or the contrary). Finally, as a last recourse, the walkway could be designed to be washed away.

When the catchment area is wooded, trees may be torn away from the banks during strong floods. This possibility becomes a certainty during exceptional floods. But experience has shown that a lot of other floating debris can arrive in front of the spillway, in particular trailers!

The minimum dimensions that can be recommended to discharge such debris are as follows:
- sill length of 10 - 15 metres between piers;
- air gap 1.5 to 2 metres under a walkway or a bridge;
- shaft 6 to 8 metres in diameter.

If the spillway is smaller, a protective system must be envisaged to trap floating debris far enough from the area of inflow to the spillway to avoid any disturbance in flow conditions and heightening of the water level (trash rack with bars spaced wide apart). It is of course vital to design an easy means of access to come and recover floating debris after the flood. The solution consisting in placing a floating boom upstream from the inlet to the spillway is not satisfactory. Such a system is in fact difficult to handle if it is expected to function at various water levels. In case of a very strong flood carrying a large number of logs, there is a risk that the cables will break and the spillway will be blocked. Booms should be reserved for the case of a very large spillway where they serve to channel floating debris.

Operation of moving parts

When a spillway is gated, which is not common at small earthfill dams, it must be guaranteed that they will open during a flood. A reliable automatic system should therefore be designed, coupled with an alarm to the operator in order to rapidly have human presence during the flood or in case of spurious opening of the gates.

Fuse gates[1]

Hydroplus fuse gates installed on a specially designed free overflow sill can increase the volume of water stored and/or discharge a re-evaluated flood.
These are prefabricated jointing elements 0.50 to several metres high, corresponding to about three-quarters of maximum head without the fuse gates, which tip one after another in case of an exceptional flood so that the design freeboard is preserved.

This system can be applied both to existing dams and to new constructions.

Flexible sills[2]

When the overflow sill is very long, another interesting solution consists in placing a water inflatable system 1 to 3 metres high that will automatically deflate as the water level rises.

Like fuse gates, this system offers advantages as it requires no outside source of energy to be lowered (*see Chapter V, p. 136*).

Location and dimension

A surface spillway is normally built on one of the two abutments as it will then be founded on materials that are unlikely to settle. The designer will choose either the abutment that provides the shortest route to the downstream side of the dam, or whichever abutment is stiffer in order to provide a better foundation, or whichever abutment is less stepp to reduce problems with earthworks.

1. See *Bibliography*, reference 13, p. 111.
2. See *Bibliography*, reference 14, p. 111.

In the case of very wide and symmetrical valleys, the distance will nevertheless be very long, which gives rise to the idea of placing the spillway on the fill along the thalweg . For low embankments that are well compacted with a relatively incompressible foundation, this solution can be suitable. The concrete structure is built with articulated joints and can absorb the little settlement that is observed with no damage. Such a design is now classic for dams up to about 20 metres high and even higher as long as the length of the overspill sill does not exceed approximately 15 metres, to avoid building construction joints between the two banks. It is not excluded that a longitudinal joint may be built, but then the structure becomes more complex.

When there is a rock dam abutment or a rock saddle[1] at an altitude close to that of the reservoir, an economical solution consists in digging out an unlined channel[2]. If the rock in the channel walls is crumbly or vulnerable to frost, prudence is required because of the risk of obstruction by rockfall. Given the possibility of slow retrogressive erosion in the channel, it will also be necessary to place a sill at its upstream end anchored down to the unaltered rock.

In the case of a classic reinforced concrete channel with Water-Stop joints between sections, the action of water outside the channel must be taken into account, in particular by placing:

- a cut-off to reduce circulation along the channel walls and bottom (risk of internal erosion);
- a system to combat uplift that could cause heave in some parts of the channel, in particular the sill and energy dissipation areas (drain, weepholes, anchor bars, horizontal footing). Drains or weepholes must be bent towards the downstream to avoid generating any uplift due to the kinetic term in water pressure ($V^2/2g$).

The channel should be set out as straight as possible and changes in cross-section or slope should be as regular as possible. If not, stationary waves may develop at such discontinuities at the upstream part of supercritical flow. They will rebound back downstream on the side walls of the chute channel. The rise in the water line that results can cause overspill during floods, which could damage the embankment's abutments or the embankment itself. To guard against such a problem, the simplest technique consists in setting out a spillway with a perfectly straight frontal inlet on the fill or on an abutment, or in setting out a lateral inflow spillway followed by a narrowing structure with a gentle slope and a straight chute aligned with that structure.

Calculation of the water line (in a steady state for the attenuated design flood flow) is used to dimension the elevation of the walls, selecting a minimum freeboard of 0.50 metre. It is recommended to do a second calculation for flow with the safety flood (*see Chapter II, p. 25*) in order to check that that flood will not cause overspill. In this way, it can be effectively considered that the safety flood is reached when the reservoir reaches the crest of the embankment (or if the embankment has a watertight core, the crest of the latter when the fill above it is permeable).

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1. In the case of a saddle at an altitude close to that of the reservoir, see chapter III, Feasibility study, stage 2, p. 54.
2. Unlined channels require greater surveillance. They are not recommanded if the rock is cracked.

For a shaft spillway, the tower should be founded on stiff ground and be sufficiently heavy to avoid any risk of heave (taking a safety factor against uplift of the order of 1.2).

Solutions tailored to very small dams

Very small dams are earthfill dams where $H^2 \sqrt{V} < 30$ approximately.

Rockfill bonded with concrete

It is possible to make the central area of the embankment an overflow area lined with a transition layer and a rockfill layer bonded with concrete, with no joints but with ejector tubes to combat any uplift and a cut-off wall at the overflow sill to avoid risks of flow around it. Of course, such a solution requires surveillance by the operator after every flood, as well as regular maintenance[1].

Rockfill bonded with bituminous mastic

Such a strong and flexible lining is more reliable than the rockfill lining described above, but it is also more costly. This technique is of interest for very small dams on rivers with soft foundations that have to handle very high flood flows[2].

Behaviour of ancillary structures in earthquakes

The influence of earthquakes on ancillary structures should be analysed, at least by simplified methods, whenever earthquake activity is high (α > 0,2 à 0,25).

In case of a strong earthquake, design may be modified. For example, it is prudent to avoid a surface spillway installed on the fill. In the case of an intake tower at the upstream toe in the valley bottom, verifications should be made with the reservoir full (Westergaard type overpressure) and with the reservoir empty, given the height of this type of structure.

1. See *Bibliography*, reference 15, p. 111.
2. See *Bibliography*, reference 16, p. 111.

Tendering and dam construction

Tendering

After the various studies required to design the dam, tender documents must be prepared, including the administrative and technical documents that will help contractors to offer a price for dam construction under the established conditions. Restricted tendering is recommended so that only contractors with a level of competence that corresponds strictly to the planned works are invited to tender.

Except for very small dams where the earthworks play the predominant role, it is recommended that the works be divided into packages and the procedure of separate contracts be followed, with each package corresponding to a technical speciality (earthworks, concrete, watertightening, etc.) with the main contractor (earthworks) in charge of co-ordination. In this case, the co-ordination must be costed in the main contractor's bid.

The works will generally be planned in a single summer. However, for larger dams it may be necessary to plan on two seasons.

The preferred procedure is tendering with no possibility of alternatives, or in some cases with limited possibility of alternatives, which of course assumes that preliminary studies will be very thorough. It is recommended that payment on a unit cost basis be chosen.

The tender documents will include the particular specifications for the tender defining the conditions of tendering, of submission of bids, and of bid analysis and contract award. The tender documents that will be part of the contract after being filled in or signed by the chosen contractor are as follows, in addition to the General Administrative Clauses and General Technical Clauses:
- *Contractor's commitment;*
- *Particular Administrative Clauses*, a document established by the agency responsible for the operation, which may not be modified or filled in by the contractor;
- *Particular Technical Clauses*, a document established by the engineer that modifies and complements the general technical clauses;
- *the unit price schedule*, which should take into account all the technical requirements and operations defined in the particular technical clauses to minimise any source of dispute during the works;
- *the cost estimate*, including the foreseen quantities in the parts of the project to which each unit price applies;
- *drawings and technical data sheets*, designated in the particular administrative clauses as integral parts of the contract.

The tender documents may also include data sheets and drawings that are only of indicative value and for which the owner is therefore not liable.

CONSTRUCTION PRINCIPLES TO BE STIPULATED IN THE PARTICULAR TECHNICAL CLAUSES

Protection of the construction site from water

This addresses the following problems:

- collecting springs and infiltration, and release outside the various excavations; temporary or permanent structures may be required;
- protection of the fill and the borrow areas from runoff;
- the particular technical clauses should stipulate up to what flow the construction site should be protected, for example the 10-year flood calculated over the period of construction, but for practical reasons, this flow will be translated into an elevation in the tender documents and contract. That elevation will be referenced to a fixed point downstream from the construction site. The contractor bears the sole liability for protection of the construction site and his equipment as long as that elevation is not exceeded.

Conditions for quarrying in borrow areas

The preliminary studies will have defined the various types of materials encountered in the borrow areas and, according to their intrinsic and in situ characteristics, will have assigned them a function and a place in the structure, or for some of them, will have decided to eliminate them.

Additional site investigations and tests must be done to detail the results of the study as soon as the test section is begun.

After stripping to eliminate and store topsoil and plant debris, a plan for operation of the borrow area defining what part of the structure the various materials are destined for should be set up.

Materials are extracted either in horizontal layers, which would permit some drying by the sun and the wind, or in vertical layers, which would permit mixing of the various layers.

The extracted materials can only be placed directly if their water content is within the prescribed range. If not, drainage of the borrow areas or temporary storage for drying or wetting can be necessary, and a cost for this work should be given in the price schedule.

Preparation of excavations

After examination of the base of the embankment and the cut-off, it may prove necessary to excavate deeper.

Before placing any fill or concrete:

- the surface of the excavation bottom must be cleaned and strengthened;
- a detailed survey must be done;
- the excavations must be formally accepted (*see Chapter III, p. 62*).

In the case of a rock foundation, special attention must be paid to identifying any geological features such as cracks (empty or filled in with erodable materials), fossil karsts, or soluble materials. Appropriate treatment must then be undertaken.

Fill placement

To build a dam with the dimensions of the theoretical cross-section indicated on the final design drawings, fill must be placed with extra thicknesses on all the slopes. These extra thicknesses, which allow effective compacting down to the theoretical limit of the slope, are removed at the end of construction.

To guarantee a good bond between the fill and the soft foundation, the latter must be compacted with the same equipment as the fill and then scarified before the first layer of fill is placed (*see photos 7 and 8 p. III*). When the slopes of the cut-off are loose ground, it is recommended that the compactor should push up onto the slopes to ensure good bond between the fill and the ground.

The material to be compacted is levelled along a flat layer of uniform thickness over a base with no hollows or bumps that has been scarified in advance to a minimum depth of 5 cm to assure a good bond between layers. Compacting standards are defined in the particular technical clauses, as are the methods of carrying out test sections (*see Works supervision hereafter, p. 104*).

When the construction site must be shut down, in particular at the end of the day, the last layer placed must be smoothed with a slope to permit any rainfall to run off naturally. Wetted fill may be kept, if it is possible to bring water content down to an acceptable level before compacting. If not, it must be removed. Frozen fill must be removed.

Near rigid structures or in certain special points, normal compacting equipment cannot be used. In this case, it is replaced with equipment suitable for the circumstances at hand that can compact the fill according to the defined requirements.

Stocks of draining and filtering materials must be protected from runoff to avoid any contamination. They must be placed as quickly as possible and be protected immediately with a layer of compacted materials.

In the case where a zone is stripped off, it must always be connected by a fairly gentle slope to the neighbouring zones of the fill, in order to permit normal use of machinery and avoid discontinuities in the mass.

Test section

After the contractor has been chosen, a test section must be built.

Role of a test section (see photo 3 p. II)

The test section is intended to establish the conditions for fill placement and determine the best methods for construction.

It is systematically planned before construction of the fill and can be used to advantage for construction of a protective cofferdam or access road. The test section should not be incorporated into the embankment unless it can be guaranteed that it will not later be a zone of weakness. It must be built with the equipment proposed by the contractor, in particular with the equipment to smooth, level off, scarify, and compact the fill, in order to check that it is appropriate.

If difficulty in placing the materials requires a search for an appropriate compactor in particular (for example for hard soils or soft rock), the particular technical clauses stipulate what types of machinery the contractor must test during construction of the test section.

Finally, for some crumbly materials, it is recommended to compare grading before and after compacting and check that the resulting grain size ranges are acceptable.

Dimensions

After finalising any treatment operations required, the compacting test is carried out on a test section with the following minimum dimensions:
- length: 30 to 40 metres;
- width: 4 to 6 metres.

The surface area must be properly prepared in advance by stripping off top soil, eliminating lumps, drying, scarifying to a depth of 0.15 metre, placing a 0.20 metre layer of material, and compacting (10 passes).

It is recommended to then place at least three layers of fill in order to:
- eliminate any effect of the base;
- check bond between layers;
- test two or three different thicknesses of the layer.

If the tests involve several compactors and/or several different materials, the number of sections or their surface area must be increased accordingly.

Measures concerning the effectiveness of the compactor

Here the question is to determine the appropriate thickness of the layers and the corresponding number of passes of the compactor.

In general, for a given thickness the entire section will receive what is considered minimum compacting, for example 6 passes, and then is divided into 3, 4 or 5 parts which each receive additional compacting in order to obtain, for example, one part with 6 passes, another with 8 passes, another with 10 passes, and the last with 12 passes (a pass corresponds to one way travel by the compactor over the area).

Measures of water content and dry density are then taken in the central part of each of the sections (where there is no edge or overlap effect). These inspections are done by the same agency with the same equipment and the same operator as will be used on the construction site itself.

However, when the material is not very homogeneous, it is preferable that each layer be compacted with an increasing number of passes, and successive measurements be taken at the same place.

The optimal number of passes to reach the desired dry density must generally be between 6 and 12, in order to achieve fill of sufficiently homogeneous compactness and to optimise use of the machinery. The thickness of the layers will be increased or reduced depending on whether the number of passes required is less than 6 or greater than 12.

That thickness, after compacting, must nevertheless always be between 0.20 and 0.50 metre. It should be checked that the entire thickness of the layer is properly compacted. Digging a trench is one way to observe the homogeneity of the compacted material.

Equipment

There are three main categories of compactors:

- *pneumatic tyred rollers* are suitable for compacting almost all soils, but the use of heavy rollers means a risk of layers being foliated, while less heavy rollers may be ineffective at greater depths;
- *tamping rollers*, preferably self-propelled, are suitable for compacting fine soils;
- *smooth vibrating rollers*, generally self-propelled, are preferably reserved for granular soils (sand, dry gravel) and rock materials. They are very effective at depth but not on the surface, at least for the first 2 to 5 centimetres.

As concerns excavation and transport of materials, the two most commonly used methods are:

- the motorscraper;
- the mechanical shovel along with trucks or dumpers, which helps in mixing several different layers and is most suitable when the borrow area is far from the dam or has a pronounced relief.

Works supervision

After drawing the conclusions from the test section, the objective is to build, under the best possible conditions, a structure that will be used for several decades, with the least possible maintenance.

General philosophy of works supervision

Whatever the quality of the geotechnical studies done on the project, they are always limited in scope, both as concerns the investigations on the foundation and those in the borrow areas.

Construction will therefore often reveal situations that were not foreseen during the studies and to which the works must adapt very quickly.

Another fundamental aspect of homogeneous earthfill dams and some zoned structures is their sensitivity to heterogeneity. A few percent of the volume of fill placed in poor conditions (either because of their inherent quality or because of outside conditions) and in the wrong place can cause the failure of an earthfill dam.

These two aspects incontrovertibly reveal the necessity for constant supervision in the widest sense of the term.

The only participant in the project who is always present on the construction site is the contractor, so it is obvious that he will play a crucial role in the quality of the works he is carrying out; this means self-inspection, which can only be envisaged with competent contractors possessing good experience with this type of structure.

This aspect is very important and must be properly understood when the contractor is selected (beware of the owner's temptation to take the *least cost* rather than the *best proposal*), especially when the structure is small; as further along we will see that the engineer cannot provide permanent supervision on small projects.

As the role of the contractor is so essential, it is recommended that he be encouraged to apply a quality assurance plan. This precaution does not, in any way, reduce the need for careful inspection of construction, but it can help that inspection to be carried out in an easier context.

Objectives of inspection

The objective of inspection is, *ultimately*, to check that the dam is built in compliance with the design, or at least with good practice in the field. Materials must therefore conform to specifications, as must placement.

As concerns the compacted materials in any zone of the fill (or the entire embankment if it is homogeneous), they must:

- be of the foreseen inherent quality, meaning that their grading, their ATTERBERG limits, their organic material content and their mineralogy must comply with the prescribed limits;
- be in a state of wetness conforming to the acceptable range;
- be sufficiently compacted to avoid any later problem, but with no adverse overcompaction.

As concerns the other materials in the fill, whether filters, drains, or rip-rap, their quality is the most important point to check: grading, block size range, fines content, presence of clay, friability, abrasion resistance, mineralogy.

In addition to the quality and placement of the materials, there is also a need to inspect the setting out elevations and dimensions of the various parts of the embankment, starting with acceptance of the excavations.

Finally, inspection will look at special systems (grouting, diaphragm wall, monitoring devices, etc.) and at ancillary structures (spillway, intake and outlet).

All of the inspections mentioned above are first and foremost visual, but in general they will be combined with measurements and tests that may contradict the impressions gained by visual inspection, at a frequency that depends in particular on the inspector's experience.

Inspection of compaction

This consists systematically for all embankments in measuring water content w and dry unit weight γd, using a gammadensimeter or a membrane density meter (*photos 4 and 5 p. II*). The measurements must be compared to the relevant results of the studies and the test section. Except in very special cases, the heterogeneity of materials will make any comparison very difficult, even if the study has clearly defined the various types of materials.

In fact, the points that must be determined with precision are the following:
- *Difference between w and optimum moisture content* (OMC) (for example if w = 20% and OMC = 18%, the material will be said to be 2 points on the wet side) and *compaction* $\gamma d/\gamma d$ optimum (e.g. if $\gamma d/\gamma w = 1.70$ with γw = unit weight of water, and if γd optimum/ $\gamma w = 1.73$, compaction is 98%); the most reliable evaluation therefore consists in combining each measurement (w, γd) with a standard Proctor compaction test (material taken at the exact point of measurement), which can be a quick test according to what is known as the *Hilf* method. Experience has shown that water contents that are more than 2 points from OMC on the dry side or the wet side often correspond to materials that are difficult to use.
- *Degree of saturation S of the compacted material;* this is easy when the specific unit weight of the particles γs is relatively invariable; it is of interest to plot all the measured results (w, γd) on a chart; values of S that are less than 70% in general reflect an insufficiently compacted dry material; values over 90% generally correspond to a wet material where pore pressures are likely to develop; values for S over 100% reveal an error in w, γd or γs; and of course, values of S between 70 and 90% do not necessarily mean that the material is correctly compacted;
- Dry unit weight γd of the material in the borrow area (if it is a material with constant S, we only need to know w to obtain γd); this is important if this value is notably greater than γd optimum, which is often the case when hard soils, soft rock or weathered rock are quarried; satisfactory compacting with no wetting as developed on the test section then generally corresponds to a value of gd between γd in the borrow area and γd optimum (for example if γd in the borrow area = 108% of γd optimum, the suitable compaction will be of the order of 104%)[1].

In addition to the measurements and tests described above, inspection of compaction can also be based on simpler laboratory tests than the Proctor standard test but related to it by correlation and also on tests on the fill using light dynamic penetrometers in particular, with the precision achieved then being variable.

Finally, visual inspection of fill placement usually makes it possible to detect excessive water content (moving surface when the machinery passes over the fill) or insufficient water content, especially if the visual observations can be linked to previous quantitative inspections (*photo 6 p. III*).

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1. See *bibliography*, reference 17, p. 111.

Extent of inspection concerning compaction

Reliable inspection of compaction requires the constant presence of a competent geotechnician whose judgement will be based on a number of PROCTOR tests. Since this constant inspection is in addition to general works supervision, which is part of the engineer's services in the case of a standard mission, it will mean an extra cost for the owner. This gives rise to the following recommendations:

- a competent representative of the engineer, with complete laboratory equipment, should ensure constant inspection of compacting for all dams where $H^2\sqrt{V}$ is greater than 300;
- when $H^2\sqrt{V}$ is greater than 700, a laboratory technician (or more than one if the rate of placement so requires) will back him up;
- when $H^2\sqrt{V}$ is less than 300, constant inspection is also recommendable but, if this is not possible, the engineer should conduct a standard inspection of compaction during each of his visits. He will also carry out global inspection of all the compacted layers using a light dynamic penetrometer, which will in particular detect the presence of soft materials.

The frequency of the measurements and tests will depend on construction site conditions; recommendations for all dams are:

- at least one measurement (w, γd) per 1 000 m^3 of materials compacted;
- one Proctor standard test for one to ten measurements (w, γd).

The particular technical clauses *(see p. 100 and 101)* should stipulate what method to follow in case of non compliance with specifications; in particular alarm points should be defined concerning the suitability of materials (grading, water content) and hold points for their placement (thickness of layers, density, water content).

Set of record drawing documents

This step is often neglected for the smallest structures, but grouping all the documents concerning construction in an as-built file is very important to prepare and facilitate maintenance. When this file is set up, special attention should be paid to all elements that will no longer be accessible after construction (surveys of excavations, position of the drainage system, fill inspection, type and layout of monitoring devices) as well the description of any incidents that occurred during construction such as floods, frost, or holds due to rain.

Specific features of very long dykes

Dykes are fill structures built lengthways along a store of water, to protect the river valley from penetration of that water. Such developments may have one or several objectives: power generation, navigation, flood protection, irrigation, recreation, etc.

The cross-section of a dyke is similar to that of a small dam, but the structure is of course much longer - even up to several kilometres. The volumes of water held back by a dyke are generally very great in comparison to their height.

A single structure will generally encounter a wide variety of site conditions as concerns topography, geotechnics, hydrogeology, etc., as well as widely differing constraints according to the characteristics of the areas they cross, whether urban, agricultural, wild or other, and the combination of human activities in the form of industries, communications, irrigation, etc.

Special features in the site investigation phase

Given the vast surface areas covered by the investigations, it is generally necessary to organise them in two stages:

- a preliminary investigation campaign in the planning stage:

- surveying at scale 1:10 000 or 1:5 000,
- geotechnical studies with mechanical augers and a few coreholes,
- hydrogeological studies with piezometers.

- a detailed campaign in the final design stage

- surveying at scale 1:2 000, 1:1 000 or 1:500,
- geotechnical studies with coreholes, test pits and laboratory or in situ tests,
- hydrogeological studies with water tests (LEFRANC) and pumping.

Specific features in the design phase

The great length of such projects, sometimes with only a limited surface area occupied, means major earthworks and exchanges between excavations and fill or deposit areas.

Project optimisation aims at balancing excavations and fill by minimising the borrow areas and deposit areas required as well as transport (especially by trucks on roads).

When the dyke is designed with a side channel on the downstream toe, absolute watertightness is not a goal and the drainage structure allows infiltrating water to flow off naturally, as well as regulating the watertable (draining, holding or replenishing it as the case may be).

The upstream toe should be protected from scouring.

Slopes often suffer from the various agents of erosion: wave action, drawdown, current

speed, wind action, frost, etc. The range of protective measures available is quite large:
- bituminous concrete linings,
- pre-cast concrete linings,
- rockfill,
- mixed seeded protections,
- gentle seeded protections.

The chosen solution will result from consideration of technical, economic, appearance and ecological concerns. Upkeep must also be taken into account.

Various criteria should be examined:
- stability of the downstream slope at the maximum gradient,
- stability of the upstream slope during rapid emptying,
- stability in the case of overtopping (for an overtoppable dyke),
- correct functioning of drainage via the side channel,
- 2D flow calculation and exchanges between the watertable, the reservoir and the drain,
- compliance with filter requirements between the components of the dyke and the ground on the site.

Impounding water behind the structures requires special attention and very strict supervision. Piezometers will be installed in cross-sections and monitored continuously as the water level progressively rises, along with infiltration flows in the side channel.

If the drainage system should not function satisfactorily, relief wells are drilled in the side channel to connect up the deep-lying aquifers under pressure with the drain.

Elements on cost

The total cost of a dam depends on the specific construction conditions on the site. It may be considered too high when the objective is irrigation, resulting in abandonment of the project by the owner, whereas a similar cost may be considered acceptable for drinking water supply or for a recreational facility. The level of public subsidies also has an influence on the owner's decision.

Capital costs

Capital costs include three parts:
- *the cost of dam construction.* Earthworks represent on average more than half of the cost while the rest is due to ancillary structures (spillway, intake, outlet), installation of plant, watertightening measures and monitoring systems; an order of magnitude of how construction costs are divided up is given in table 8 for homogeneous and pseudo-zoned dams;

◆ *engineering costs.* These include surveying, environmental impact assessment, land ownership investigations, hydrological studies, geological and geotechnical investigations, establishment of the design and the tender documents, and permanent works supervision. These costs represent on average approximately 12% of the cost of the works, but in some cases may rise to 15 or 20%;
◆ *costs induced by creation of the dam,* including land purchase, compensation, remedial measures, restoration of communications and transport networks. This cost may be low or very high: beware of under-estimates.

Cost of maintenance and surveillance

Operation of maintenance and surveillance can be divided up into three categories:
◆ *routine operations,* generally carried out by the owner or operator: operation of equipment, visual inspection, measurements of groundwater levels or leakage flows, mowing seeded slopes, removing trees and shrubs, painting metal parts, etc.;
◆ *specialised operations,* generally contracted to an expert surveyor and to an engineering firm (inspections, interpretation of readings from monitoring systems, reports);
◆ *exceptional maintenance operations* such as replacement of antiquated or defective equipment, repair of concrete structures (resurfacing, treatment of cracks).

When structures are properly designed and constructed, and do not later require major reinforcement work, the total cost of surveillance and maintenance (including services provided by the owner himself) can be estimated annually at approximately 0.25 to 1% of the up-to-date cost of the works. The owner should therefore plan for this cost.

ITEM	Range of variation %	Typical values	
		without grouting	with grouting
Earthworks	25 to 65	60	50
(including fill)	(20 to 55)	(50)	(40)
Grouting / cut off	0 to 20	0	15
Civil works	15 to 25	20	17
Équipement	5 to 20	10	8
Construction site plan	5 to 15	10	10
TOTAL	100	100	100

Table 8 - Approximate breakdown of cost items for an earthfill dam

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▲ **1 -** Compacting clay in the cut-off trench (text p. 71).

◀ **2 -** Vertical drain dug back out of the embankment and filled in with a draining, self-filtering sand (text p. 78).

Test section and compaction inspection

▲ **3 -** Test section for a very small dam (at the bottom are the smooth roller compactor and a motorscaper conveying earth from the borrow area) (text p. 102).

▲ **4 -** Compaction inspection with a membrane densimeter…

▲ **5 -** … or a gammadensimeter (text p. 106)

6 - Clay compacted at ▶ a water content considerably higher than the Optimum Proctor. This layer must be removed. (text p. 106).

◀ **7 -** Scarification after passage of a smooth roller compactor. (text p. 102).

◀ **8 -** Compacting with a vibrating tamping roller (scarification as shown above is pointless with this roller). (text 102).

III

▲ **9 -** Anti-contaminant geotextile under a horizontal gravel drain. (text p. 79).

▲ **10 -** Placing a bituminous geomembrane. (text p. 81).

Facing protection

◀ **11 -** Placing a rip-rap at the top of a dam with a constant water level. (text p. 76).

◀ **12 -** Rip-rap on a dam built for irrigation. Note the row of gabions that is used to decrease the height of the embankment while preserving freeboard. At the bottom is the spillway shown in photo 20.

13 - Topsoil covering the downstream face of the same dam. Note the berm. (texe p. 76). ▶

14 - Seeded downstream slope. (text p. 76-77). ▶

SUCCESSIVE STAGES IN LAYONG A STEEL OUTLET PIPE (text p. 93-95).

◀ **15 -** Installing supports at the bottom of a vertical-sided trench.

16 - Bringing in the pipe and welding sections together. ▶

◀ **17 -** Filling the trench with concrete.

18 - Concreting is completed. ▶

Spillway

◀ **19 -** Frontal inlet spillway, chute set on fill and energy dissipator. (text p.96).

▼ **20 -** Overspill part of a side inlet spillway.

Concrete dams

◀ **21 -** Concrete gravity dam.
(text p. 116).

22 - Single curvature arch dam (Chapeauroux water intake). Note the standard crest shape of the overflow portion. (text p.115). ▶

VIII

▼ **23 -** Riou RCC dam. (text p.118 and cross-section p.130).

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CHAPTER V

Small concrete dams

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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 ($\sigma = p.R/e$) 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 ($\sigma = p.R/e$), 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.

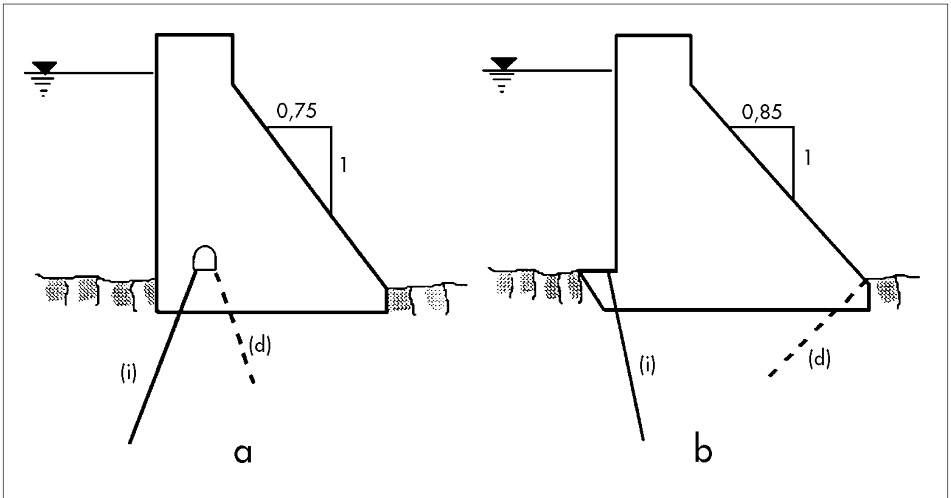

Fig. 1 - Layout of the grout curtain (i) and the drainage curtain (d): a - with gallery; b - without gallery

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,

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).*

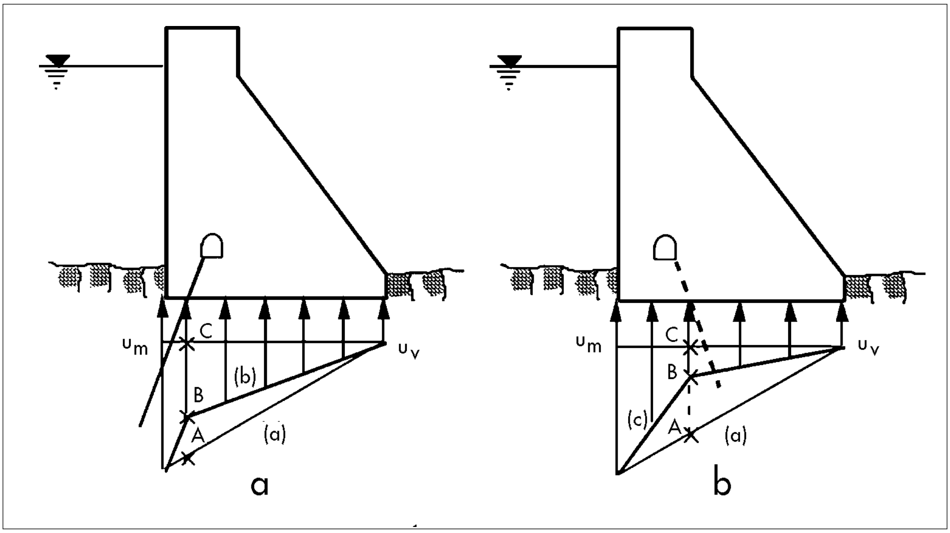

Fig. 2 - Indicative diagram of uplift:
(a) - with no grouting or drainage
(b) - with a grout curtain
(c) - with drainage

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].

Accidental actions: earthquake[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/m³).
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. φ = 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.

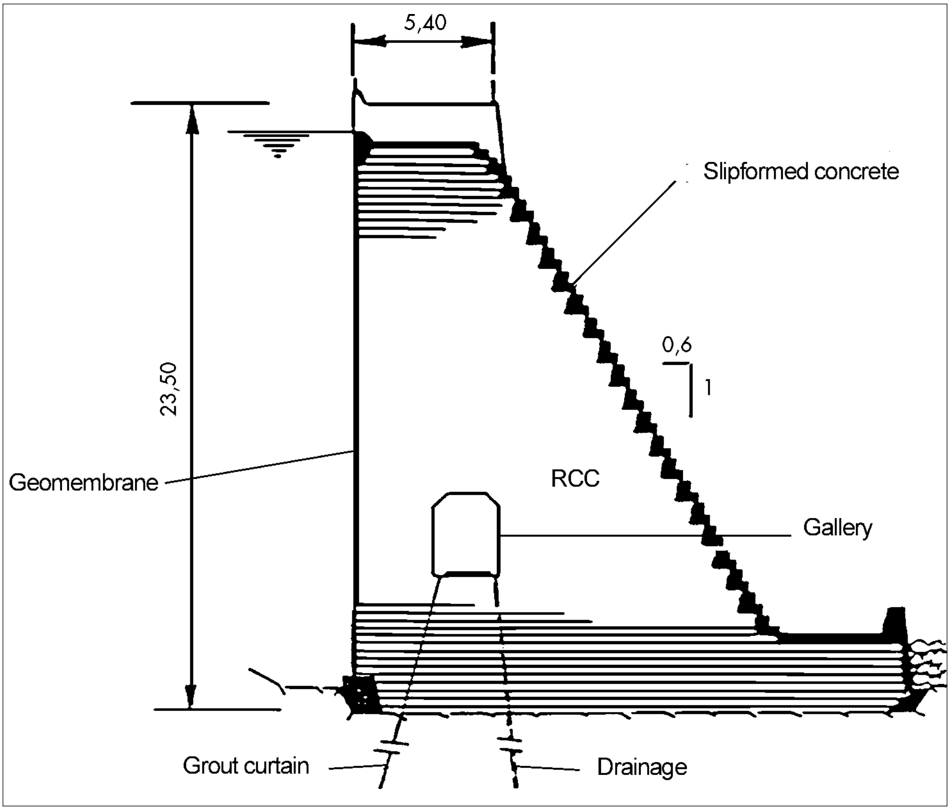

Fig. 3 - Typical cross-section of the Riou RCC dam - Faced with impervious geomembrane

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.

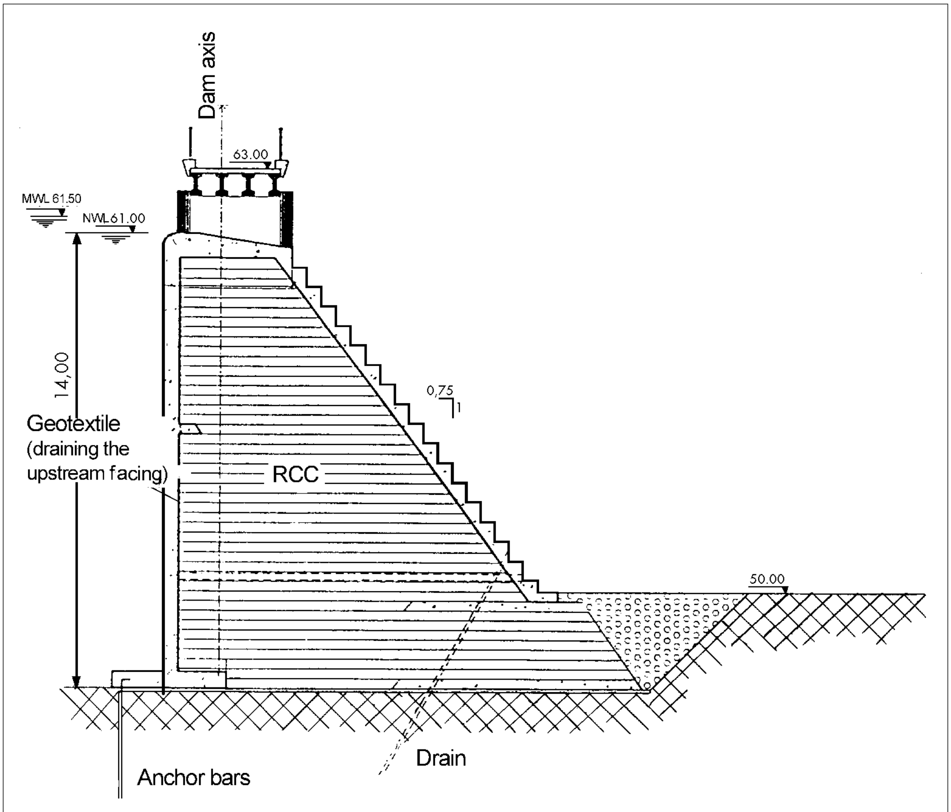

Fig. 4 - Typical cross-section of Villaumur RCC dam

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).

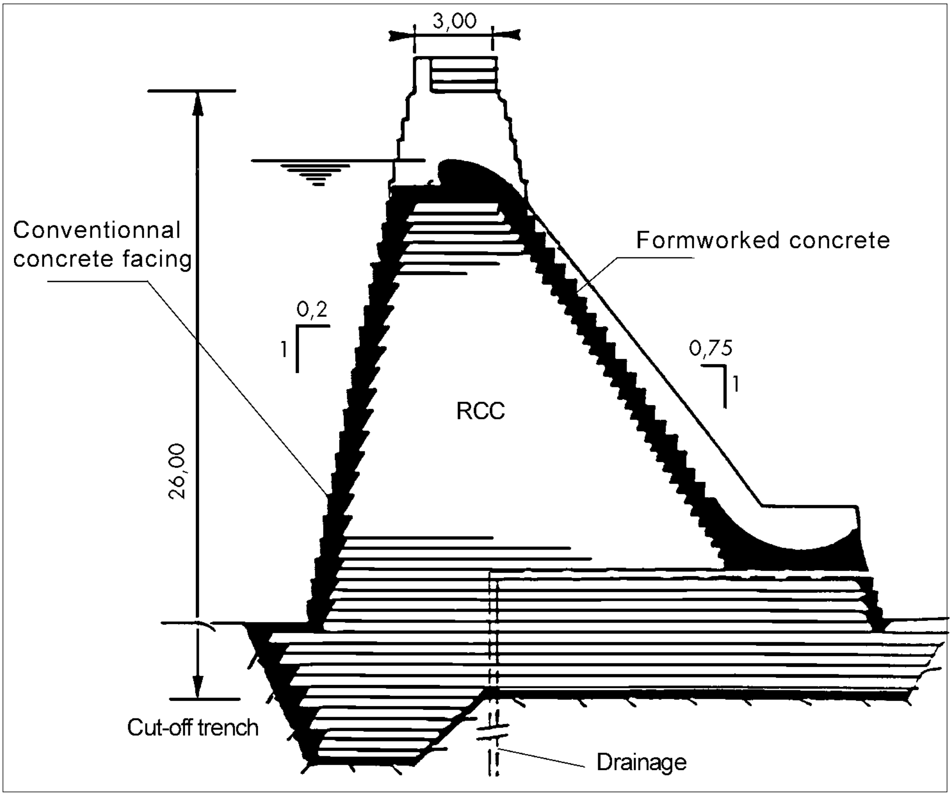

Fig. 5 - Typical cross-section of EL KOREIMA dam

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.

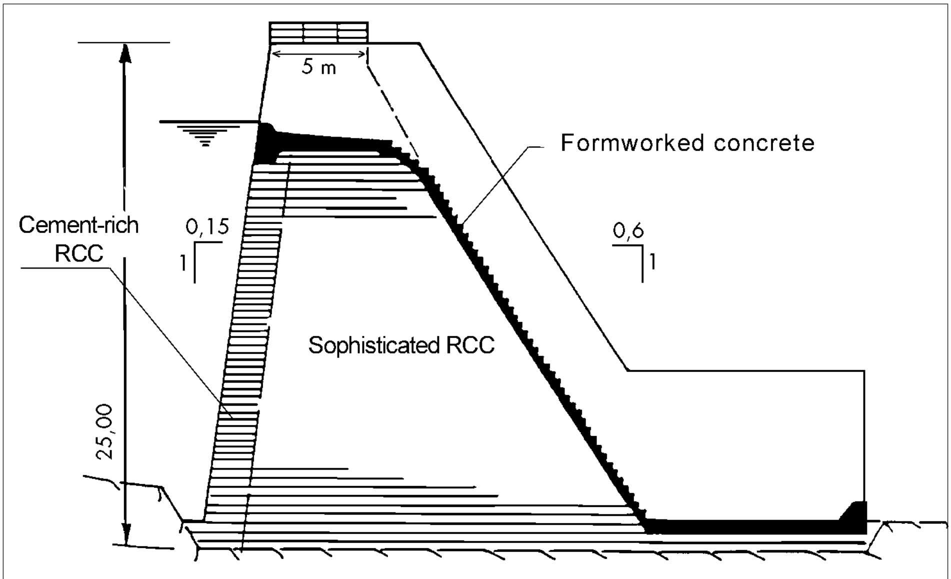

Fig. 6 - LOUBERIA dam

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.

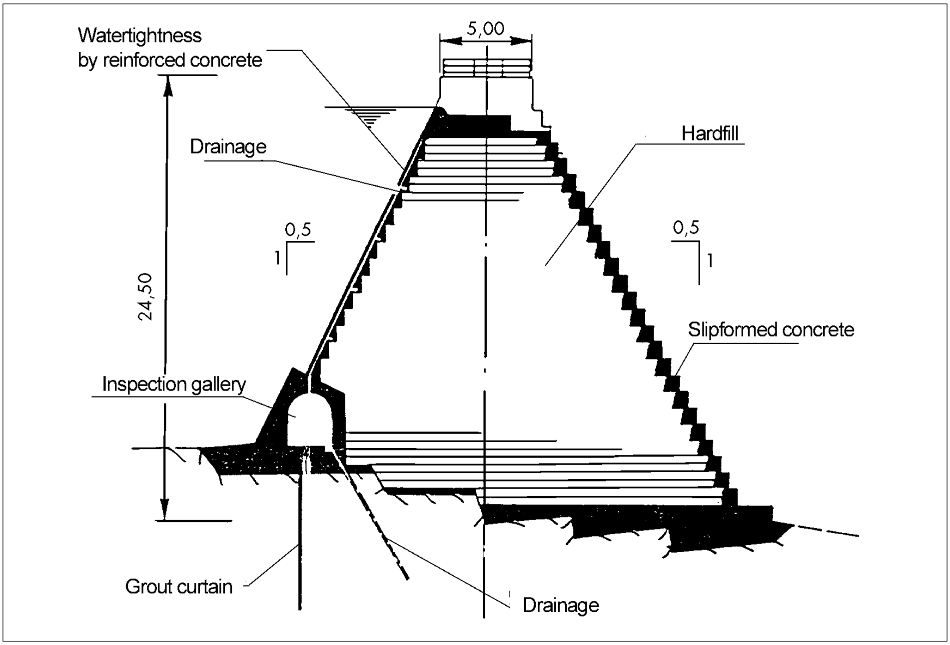

Fig. 7 - Typical cross-section of MYKONOS 1 dam with a symmetrical cross-section

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.

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

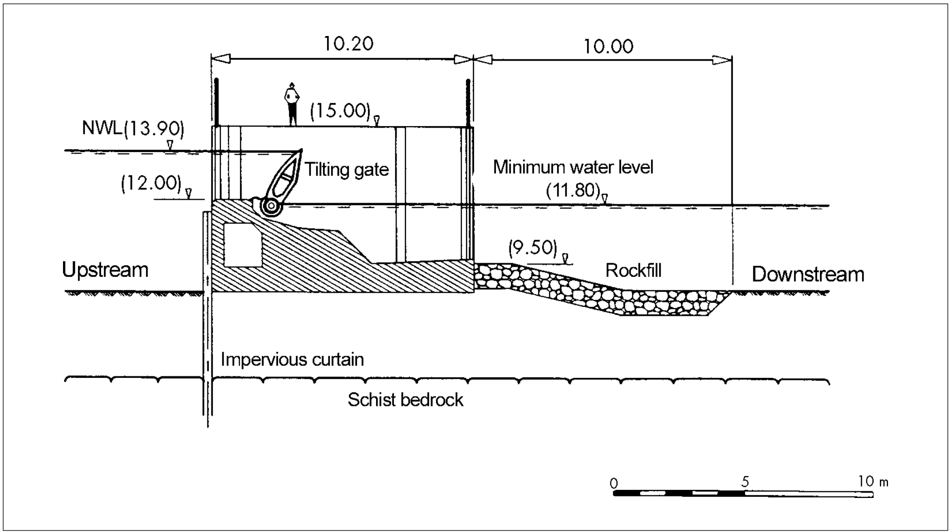

Fig. 8 - Dam with a tilting gate.

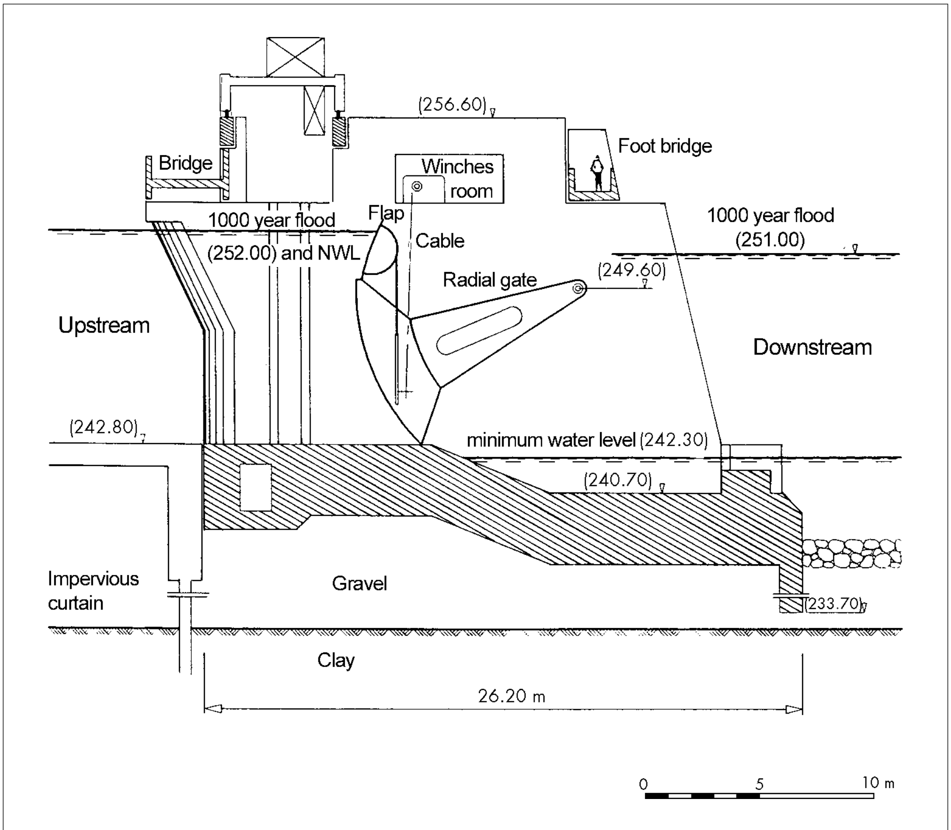

Fig. 9 - Dam with a radial gate.

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.

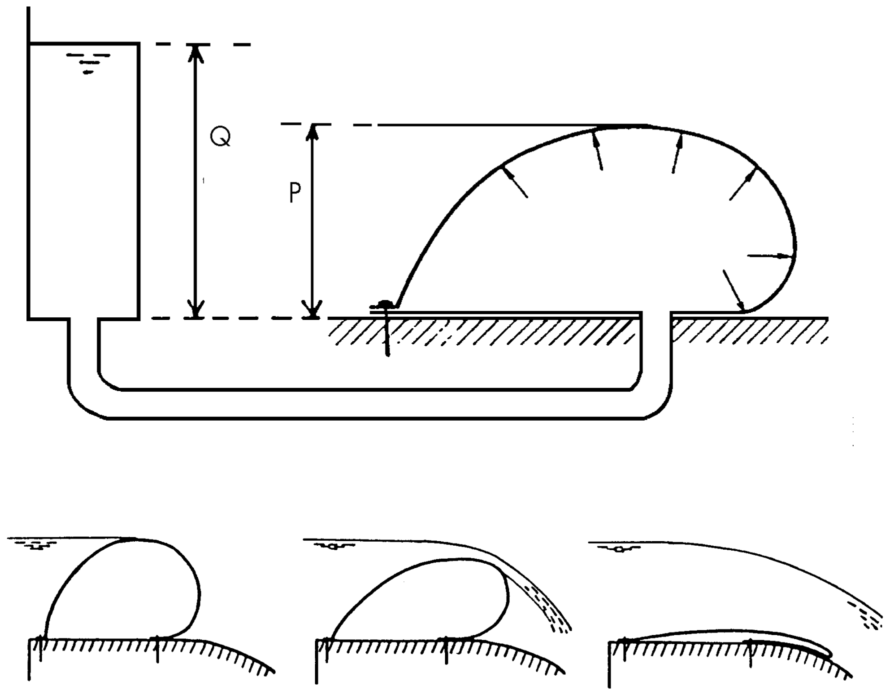

Fig. 10 - Schematic principle of a water inflated barrage.

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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Table of contents

CHAPTER VI

Management of water quality

This chapter first looks at water quality in reservoirs and, in a second part (p. 150 ...), at the consequences of compensation flow.

MANAGEMENT OF WATER QUALITY RESERVOIRS

From an article published by Henri BEUFFE (Unité de recherche Qualité des Eaux, Cemagref-Bordeaux), Alain DUTARTRE (Unité de recherche Qualité des Eaux, Cemagref-Bordeaux), Alain GREGOIRE (Centre National d'Équipements Hydrauliques Électricité De France), Antoine HETIER (Compagnie d'Aménagement des Coteaux de Gascogne), and Michel LAFFORGUE (Aquatechnique).

(Taken from "Management of water quality, flora and fauna: some conclusions and techniques for restauration", published in the transactions of the 18th ICOLD convention in DURBAN, with the kinf authorisation of ICOLD).

In comparison to natural ecosystems, man-made lakes are marked by the major influence of human beings in their management. The hydraulic regime is fundamentally modified, transforming a running water milieu to a still water milieu, thus generating vital ecological changes.

The present section will first attempt to inventory the ecological changes consecutive to construction, and then to give some initial ideas on methods to restore and manage this artificial environment.

Eutrophication in man made reservoirs

Preamble

Amongst the problems suffered by fresh water aquatic ecosystems, eutrophication is very often mentioned. The meaning of that term, which is often used inappropriately, warrants some explanation.

Fresh water naturally contains dissolved nutrient salts that enable vegetation to grow thanks to solar energy.

As they are the primary producers in the system, those plants are the first link in the food chain of aquatic animals, the consumers (zooplancton, fish, etc.).

Producers and consumers both die after reproducing, which creates a source of inert organic material that settles in the water. That matter is in turn mineralised by bacteria and then generates more nutrient salts that become available for primary producers. Thus, from one generation to the next, the ecosystem is slowly enriched by the natural succession of external inputs and internal recycling of its own production. This natural process is called eutrophication.

Aquatic ecosystems go through intermediate trophic levels and change from oligotrophic milieus (with little production), to mesotrophic milieus (with average production), until they reach a state of equilibrium, the eutrophic state, characterised by maximum yield with:

- a moderate nutritive element content;
- a maximum diversity of specific plant and animal life;
- a balanced production of plant life;
- an optimal production of consumers.

Because of the intensification of human activities around catchment areas (industries, agriculture, urban waste), the increase in nutrient inputs causes an acceleration of the natural process of eutrophication and often rapidly surpasses this state of equilibrium. There then occurs a simplification of the biocenoses, in particular to the benefit of algae, which, if allowed to develop in an uncontrolled manner, would cause major nuisances (colour, flavour, deficit of oxygen in the bottom layers of water in the summer time, deposit of sediments, etc.).

Processes that contribute to eutrophication in man-made reservoirs

Early resettlement of nutritive elements in submerged soils

Reservoirs may be enriched spontaneously when filled, by resettlement of nutritive elements from ground that has dried out. Such resettlement is related to the decomposition of the organic material in or on those soils.

A study of several reservoirs used for irrigation shows that such self-enrichment is only of limited intensity[1].

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1. See *Bibliography*, reference 3, p. 157.

To the extent that it does occur, experience has shown that in countries of temperate climate, the process is limited in time as the nutritive elements in the mass of waste are exhausted.

Variations in water level and redistribution of phosphorus[1]

Of the various types of reservoirs, those used for irrigation are subject to significant variations in water level, especially as the optimal period in terms of demand for water resources corresponds to low flow in rivers.

As a consequence of these variations, desiccation of the sediment and soil exposed to the air causes modifications in speciation of the phosphorus towards the most unstable forms, i.e. those that are readily usable by algae or higher plant forms[2]. While the processes used to explain this vary (dehydration of ferric hydroxides, destruction of the bacterial and vegetable biomass), it would seem that, among the factors playing a role in making phosphates soluble, two have an effect that can be directly connected to water level variations: the time during which sediment is returned to suspension, and its intensity as the water level falls and rises again[3]. This state will be encouraged even more if the reservoir is shallow and the bottom will be significantly dewatered in comparison to the volume discharged. The frequency and duration of such episodes will also be decisive.

Inflows from the catchment area

Inflows from the catchment area, which provide most of the nutritive load coming into the reservoir, determine the trophic level of the aquatic ecosystems over time.

In the long term, the cumulative effects of external and internal loading (resettlement by the soil and sediments) can contribute to a deterioration in water quality, accompanied by, in the worst cases, the appearance of sickening smells, proliferation of algae or some species of macrophytes, and death of fish populations.

These effects will be aggravated if the rate of renewal of the reservoir water is high. Frequent tapping of the water at the bottom is therefore a favourable factor.

Aquatic fauna

Inventories of fish life upstream and downstream of certain hydro-agricultural reservoirs have shown that[4]:

- the fish population in the reservoir has an undeniable influence that is exercised systematically on the watercourse downstream, including a decrease or even the disappearance of species that can live only in a cool, well oxygenated running water environment;

1. See *Bibliography*, reference 4, p. 158.
2. See *Bibliography*, reference 5, p. 158.
3. See *Bibliography*, reference 5, p. 158.
4. See *Bibliography*, reference 3, p. 157.

- the modification of the fish population in the watercourse upstream of the reservoi is also influenced by the fish living in the reservoir, but this type of modification seems to be slower.

The capability of indigenous species to survive in a reservoir is compounded by the consequences of introduction of foreign species. When this occurs, the type and size of the latter determine the structure of the population. Among the lake species that prove to be the fittest in reservoirs, roach, perch and bream are the most commonly mentioned. Appropriate management of fish populations would seem to be necessary to avoid the proliferation of undesirable species (especially bream).

In general, the amplitude and frequency of drawdowns have a considerable impact on fish life, through the limitation of submerged or running water aquatic flora, the basis of the life support system (insect larva, molluscs, shellfish). and the fish spawn, as well as the loss of living space.

Management and restoration

Growing needs for leisure activities have been added to traditional uses of water resources. Such an increase in uses of aquatic ecosystems generally occurs with no analysis of their capacity to satisfy them.

Each use includes a certain number of requirements and obligations, which usually lead managers to modify, where necessary, the characteristics of the milieu to meet needs as best they can. The table summarises the quality requirements for certain uses of aquatic milieus[1]:

USES	REQUIREMENTS
Drinking water	Physico-chemical and bacteriological
Swimming	Appearance and bacteriological quality
Fishing	Fish breeding potential of the milieu (a consequence of its quality: water quality, breeding and feeding zones for fish)
Canoeing and similar	Free surface: little or no vegetation on the surface
Shooting	Presence of hydrophytes (to serve as fodder for birds), sufficient free water surface

One recent movement, in terms of use of aquatic milieus, has been the effort to satisfy the greatest number of uses in a single milieu. This kind of inter-relation leads to many difficulties in cohabitation, which can often only be solved by distinguishing different geographical areas. In parallel, this has led many dam owners to build dams at the upstream end of the reservoir.

1. See *Bibliography*, reference 6, p. 158.

Finally, analysis of nuisances is rarely conducted in global terms, which often means setting up parallel actions for resources management, lacking in consistency and sometimes resulting in other nuisances.

At the same time, progressive ageing of the ecosystems created in this way along with a constant supply of nutritional matter from the outside (especially phosphorus), usually increased pollution, accelerate eutrophication and cause often adverse changes in their quality, with pollution by phosphates causing an explosion of plant growth[1].

Before first reservoir filling, it is recommended to completely clear the land to be inundated, but also to strip any earth with an organic matter content greater than 2%.

In addition, prevention of accelerated eutrophication in aquatic milieus requires treatment of the problem of phosphorus and nitrogen pollution at the source:

- through control of urban and industrial pollution,
- through creation and rehabilitation of sanitation networks and water treatment stations,
- through infiltration of waste water in sandy substrates,
- through limitation of non-point pollution sources (fertilisers of all kinds, etc.).

We will not go into the detail of these operations, but rather consider curative actions that can be undertaken when actions in the catchment areas are insufficient or too late.

Curative action for a reservoir: reduction in nutritional matter

We will briefly describe the most frequently used techniques that are cited in the literature[2].

Techniques that consist in extracting and removing materials loaded with phosphorus (sediment, vegetal matter) are efficient over the long term as they make it possible to balance phosphorus contents and slow infilling in the basin. However, such techniques are often very expensive (in particular dredging) and pose the problem of how to manage the materials extracted: sediment that may be more or less polluted, great quantities of vegetable biomass, putrid water.

It is also possible to remove deep-lying water that is rich in phosphorus, iron and manganese, via a siphon or hypolimnion diversion.

Designing the dam project from the beginning with an intake tower letting in water at several levels can make it possible to avoid such stratagems by allowing some parts of the water mass in the reservoir to be abstracted as it undergoes stratification in the summer. Early, progressive elimination of the algae production in the epilimnion, where sedimentation and decomposition often result in high consumption of the reservoir's oxygen[3], will have a beneficial effect on water quality, especially when it is intended for drinking water supply. In addition, replenishing rivers with this warmer water will in certain cases be more suitable to previous characteristics and the original aquatic biocenoses.

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1. See *Bibliography*, reference 7, p. 158.
2. See *Bibliography*, references 7 and 10, p. 158.
3. See *Bibliography*, reference 1, p. 157.

Processes that are intended to immobilise initial phosphorus levels, either by precipitating it in the water column or by blocking it in the sediment, can help to avoid requirements related to removal of the extracted products.

Among those processes, aeration of the hypolimnion can be used when there is clear stratification of the water in the natural state and when it is not advisable to mix the water column. In fact, mixing the water column vertically causes deep-lying water to heat up, with adverse effects on drinking water production and the equilibrium of fish life[1].

In stratified bodies of water, aerating the hypolimnion by injecting compressed air can re-establish oxidation conditions (by formation of ferric oxides) in the sediment and thus avoid the release of undesirable composites such as ammonia, phosphorus, hydrogen sulphate and manganese (*see figure 1*).

It should be mentioned that making the necessary devices too small can mean that organic matter is first oxidised and mineralised[2].

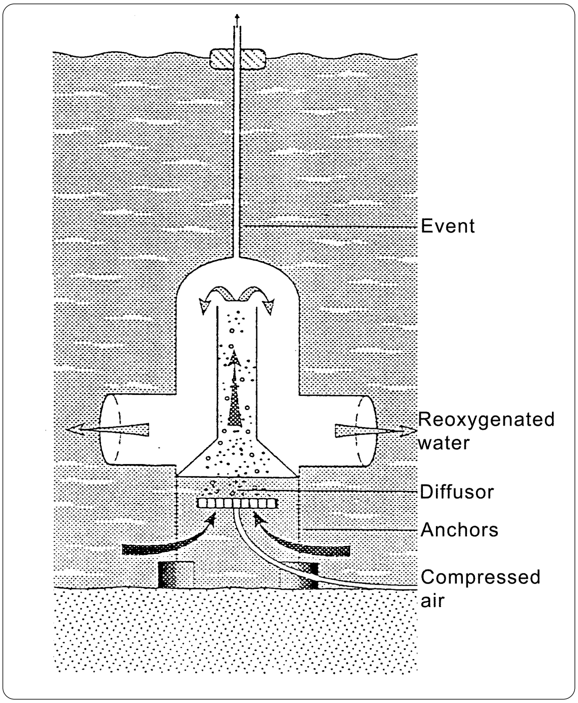

Fig. 1 - LIMNO process

1. See *Bibliography*, references 8 and 9, p. 158.
2. See *Bibliography*, reference 10, p. 158.

For reservoirs with a depth that will not permit the development of clear thermal stratification in hot weather, diffuse aeration is a good technique to combat the development of anoxia. Compressed air, supplied by a compressor on the bank, feeds polyethylene bubbler pipes floating above the reservoir bottom. Oxygenation of the water column is achieved by:
- transferring to the water column a part of the oxygen in the injected air,
- re-aerating the surface of the bottom waters that are subjected to convection where the air is injected.

Finally, treatment of sediment *in situ* by injection of calcium nitrate, ferric chloride and lime will oxidise that sediment and thus remove sulphurs, trap phosphorus and stimulate denitrification.

Whatever type of treatment is envisaged, it must be:
- tailored to the limnological problem at hand,
- effective for the technical and financial investments made,
- free of any irreversible adverse reactions.

The last two conditions are not always met, and precautions must be taken by presenting the treatment as experimental, with "post-operative" monitoring of the ecosystems reactions[1].

Management of aquatic plants

Aquatic plants can cause significant nuisances in shallow man-made reservoirs with little drawdown.

In the first few moments after their creation, man-made reservoirs are normally free of any plant life. After a few years, a considerable portion of the species in the reservoirs is the result of arrivals from outside. Such arrivals may be natural or induced, deliberately or not, by the users of the milieu. Seeds of many plants can be carried by the wind, tributaries, or aquatic birds.

The main factor in distribution of the macrophytes in lakes is depth, due to the consequences it has on attenuating light.

Thus, from the bank towards the centre of the lake a distinction can be made between helophytes with their feet more or less in the water and hydrophytes that reach the water's surface only when flowering.

The physico-chemical composition of the water and the sediment will also be an essential parameter because it is a source of nutrients, nutrients that are assimilated either through root systems or by diffusion through stems and leaves (hydrophytes).

Some species will flourish in milieus that are rich in nutrients while others prefer poorer environments; their presence is a means of estimating the trophic level in aquatic ecosystems.

1. See *Bibliography*, reference 10, p. 158.

Although there are preventive methods for management of macrophytes, such as the reduction of nutrient inflow from the catchment area, increasing shade and varying the water level, in many cases it will be necessary to make use of curative techniques. Here a brief overview of such methods is given[1].

Mechanical control

Devices for mechanical control are essentially composed of blades derived from farm machinery loaded on boats. While older models only cut the plants, more recent ones can achieve a real harvesting effect, limiting propagation from cuttings and avoiding the oxygen deficit due to rotting vegetation.

The type of equipment used to maintain the vegetation along highways can be adapted to control growth in areas of limited width.

In general, the effectiveness of such work rarely exceeds two summers, and varies according to the type of plants at hand and how readily they regenerate.

Curing and dredging are other techniques to control aquatic plant life, since they act on layers that are richer in sediment and on the roots, stolons or rhizomes of aquatic plants.

It must be remembered, as pointed out above, that these techniques involve considerable cost and also pose the problem of what to do with the material extracted. In the case of aquatic vegetation, simple uses can be envisaged such as environmentally-friendly fertiliser or compost.

Chemical control

Herbicides began to be used in aquatic milieus after the Second World War. Certain active substances have been authorised for use in France for "destruction of aquatic and semi-aquatic weeds". Some are intended to control hydrophytes, and others for helophytes. An updated list of such products is given in the *Plant Health Index*, published yearly by the french Technical Association for Agriculture.

This kind of technique is routinely employed in France. In a recent enquiry, out of 65 cases of control of aquatic vegetation in various regions, about one-fifth involved use of herbicides[1].

The use of such products, which contaminate the environment, has encountered some opposition and aroused arguments in most countries where it is authorised.

Above and beyond the toxicological hazards in the short and medium term for organisms that are not the target of the application, in particular fish, noteworthy secondary impacts are also decried, especially the risk of de-oxygenation of the milieu, related to the consumption of oxygen required for bacterial degradation of the dead plants and sometimes radical changes in the habitats in certain ecosystems.

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1. See *Bibliography*, reference 11, p. 158.

Later uses of the water in the treated milieus and those downstream from them must also be taken into account and a waiting period to permit dilution or break-down of the products must be observed.

Selecting certain plant species in this way also reduces the variety of aquatic ecosystems and generates new nuisances. Finally, the generally unpleasant appearance of areas treated in this way is another factor in rejecting these techniques.

Use of such products should therefore be considered with great prudence and not be systematically applied to the entire milieu requiring treatment, in order to limit risks for the environment and ward off secondary nuisances[1].

Biological control

In these methods organisms that consume the plants, induce diseases, or limit their growth are used.

The oldest and certainly the most widely documented example of biological control is grazing on emergent or amphibious plants in wet areas (reed beds, etc.).

Mammals and birds can also be considered as "control agents" for plant life but in most cases their impact will probably be very limited.

The grass carp (*Ctenopharyngodon idella Val.*) is one of the most promising means for biological control of macrophytes in tropical climates. It has been present in Europe for some thirty years, but it is illegal to introduce it into free water courses in France, which is supposed to keep it out of reservoirs. The effects of introducing this fish on aquatic ecosystems (in particular its adverse effect on native fish populations) have been studied in several countries and should inspire caution in the case of multipurpose milieus, even if the water system is closed.

Implementing biological control is generally cumbersome and costly and requires care to protect against undesirable secondary effects, by clearly establishing the specific effect of the control agent selected through extensive prior experimentation[2].

Conclusions on reservoir water quality

Inflow from catchment areas is the origin of most nutrient load coming into the reservoir and, over time, it determines the trophic level in man-made lakes.

The effects of that inflow, combined with internal load (release from sediments and soil that is regularly exposed), can contribute to deterioration in water quality.

This problem can take on serious proportions in reservoirs by accelerating eutrophication.

1. See *Bibliography*, reference 12, p. 158.
2. See *Bibliography*, reference 12, p. 158.

While the impact on fish populations downstream from reservoirs is practically immediate, upstream the repercussions will be much slower.

Maintaining water quality in man-made lakes depends on controlling the flows of pollutants in the catchment area, especially phosphorus, an element whose excessive presence in fresh water is responsible for phyto-plancton proliferation and all the resulting nuisances. If the problem of phosphorus is not treated at the source, it is simply transferred or even aggravated and any attempt to control it may ultimately fail despite short-term, local successes.

Finally, when action on the catchment area is insufficient or delayed, it becomes necessary to act directly on the reservoir, which implies an exact prior diagnosis and a global analysis of initial nuisances and secondary effects.

Consequences of compensation water on fish life

From an article published by Sylvie VALENTIN and Yves SOUCHON (Cemagref, Lyon) in a technical workshop on "small dams" at Bordeaux in 1993. This article mainly highlights the case of hydropower projects that short-circuit sections of rivers.

The adverse biological and ecological effects of the low flow levels guaranteed downstream from some small dams should be taken into account so that the measures necessary to limit them can be taken.

It is vital to quantify those effects according to flow. The microhabitat methodology explained herein is a tool for predicting those effects on the physical habitat of fish, and therefore provides assistance in determining the compensation water levels with the least adverse effect on fish populations. Even if it applies essentially to rivers that are habitat for salmonidae, the principle behind it can be usefully implemented to understand the consequences of a river where cyprinidae predominate.

Effects of reduced flows

There are many ecological effects due to dam management.

Reducing flows entrains a modification in morphodynamic parameters: the slope of the riverbed, sediment transport, depths, velocities, sinuosity, surface areas. These parameters define the spatial constraints on aquatic organisms, in particular the physical habitat of fish.

Morphology of rivers

The general morphology of a river is closely related to the flood discharge corresponding to bankful flow and to alternating flood and low flow periods.

In this situation, blocking sediment transport results in an overdeepening of the riverbed and increased erosion of the banks each time they are overtopped, which modifies both the longitudinal and transversal cross sections of the riverbed.

Substrate

In addition, the low flow conveyed in the river no longer ensures transport of suspended materials. Any addition of fine materials, especially from tributaries, results in clogging of the substrate. The consequence is a significant decrease in the areas where benthic invertebrates take shelter and in living areas suitable for fish populations.

Flow conditions

Reductions in flows also mean changes in the parameters that depend directly on flow, i.e. depth of water and velocity of the current. Flow conditions are therefore less varied, wetted surface areas are smaller (as the riverbed is narrower) and morphodynamic units or facies are modified.

These physical parameters play a crucial role in the quality of fish habitats. Tools are required to evaluate changes in them so that the potential impact of imposing a stable low flow on fish life can be assessed.

Measuring modifications in the physical habitat

Principle of the microhabitat methodology

To quantify the effects of reduced flow on fish habitats, the morphodynamic variables must be known at various flows so that they can be related to the requirements of fish species at various stages in their life cycles.

The microhabitat technique is a means of measuring the physical capacity of part of the watercourse.

This method of simulating the habitat was set up in the United States by STALNAKER[1] and BOVEE[2], and then tested in a number of studies. It has been further developed in France by EDF's Research and Development Division and by Cemagref[3] Quantitative Hydroecology Laboratory in Lyon. Experience is now available from biological validation in several types of rivers in France.

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1. See *Bibliography*, reference 18, p. 158.
2. See *Bibliography*, reference 13, p. 158.
3. See *Bibliography*, reference 16, p. 158.

It consists in coupling (*see figure 2)*:
- *a hydraulic model* to calculate depths of water and current velocities at various discharges from a campaign of measurements of physical variables (depths, velocities, substrate, surface);
- *a biological model* that translates those variables in terms of value as habitat via preference curves drawn up from literature and the results of fishing on French rivers.

For brown trout (salmo trutta fario) , preference functions could be validated in undisturbed rivers with brown trout populations, under non limiting conditions, especially as concerns trophic factors: a relation could be established between the biomass of the adult brown trout fished and the habitat variables that had been identified as critical.

The method is therefore preferably applicable to salmonidae habitats.

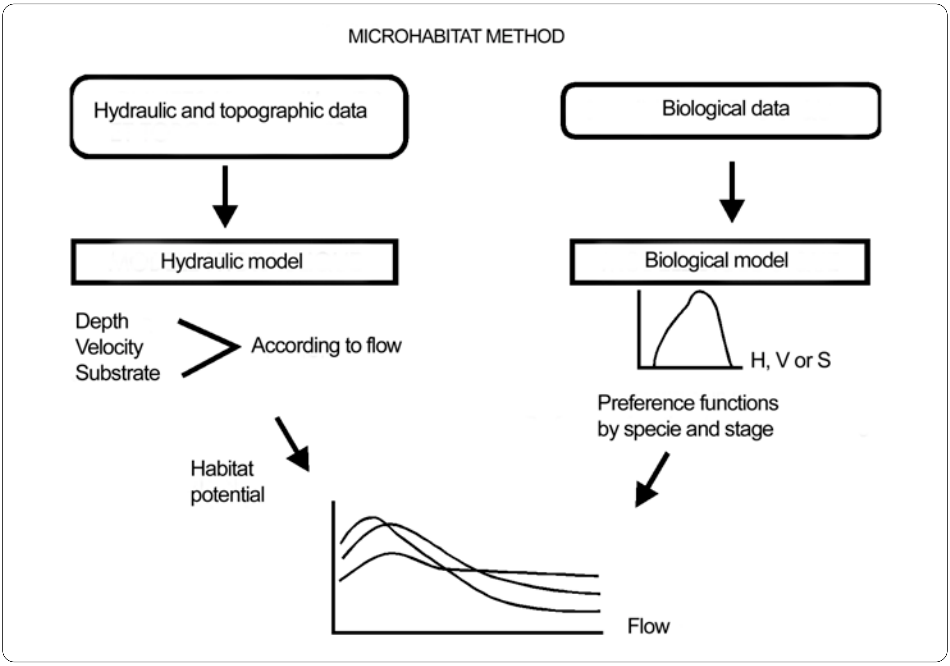

Fig. 2 - Principle behind the microhabitat method

Practical implementation

Four people are required to gather the necessary physical data, working one day per station.

Practical implementation involves several stages:
- a choice of representative stations for the morphodynamic units (facies) in the river sections under study, including a characteristic morphological alternation,
- a choice of transects representing homogeneous sectors (two or three transects per facies) that participate in hydraulic adjustment,

- surveying to map the stations and determine the slopes of the riverbed and the free surface levels,
- fine physical measurements of depths, velocities and substrates encountered along each transect.

Results

After modelling, the results are expressed, for each stage in the species under study, in the form of:

- a weighted usable area (WUA) calculated by adding the surface area of each cell weighted by the product of three preference functions (values between 0 and 1) corresponding to the values of the three variables velocity, depth and substrate, observed or calculated for any discharge,
- a habitat value, expressed as a percentage of the weighted usable area (WUA) versus the total wetted surface area.

These results make it possible to propose a tentative estimation of the effects of any modification in the hydrological regime and to identify the species that will suffer the most impact in their most sensitive habitat by detecting critical periods and stages.

Example of application of the microhabitat method

The following example is taken from the environmental impact analysis on the 8 metre high Naussac II dam on the Allier river, concerning the physical habitat of salmonidae and commissioned from Cemagref[1].

One of the selected stations was located in the first third of the 9.5 km of the river that were short-circuited by POUTÈS dam in the locality known as Sapet.

The microhabitat approach was applied for two species: atlantic salmon and brown trout (*see figure 3 p. 155*). The adult stage is not shown in the curves for the salmon habitat as the requirements of adult fish are essentially a question of crossing obstacles and the flows to attract them; those aspects are not addressed here.

Habitat curves (*see figure 3 p. 155*) for that station show that the atlantic salmon appears to be most sensitive to small variations in flow, with a maximum WUA at an optimal flow of 3 m^3/s. This flow corresponds to approximately $1/5^{th}$ of the mean flow[2] in this sector (16.7 m^3/s).

It must be verified that this flow also ensures a habitat for brown trout, which is the case in every stage. In fact, the trout has a maximum habitat between 1.5 and 3 m^3/s.

Existing compensation flow in the area is only 0.5 m^3/s, i.e. about $1/30^{th}$ of mean flow, which means a significant reduction in WUA (*see table 1 p. 156*) versus an optimal flow of 3 m^3/s.

1. See *Bibliography*, reference 17, p. 158.
2. Mean flow interannual.

It should be noted, however, that this flow is increased to 1 m^3/s in the daytime and 1.5 m^3/s at night during the period when the young fish are migrating, from 15 March to 15 June, in order to help them cross the dam (fish slide and lift). This considerably improves the sector's capacity in every stage of the two species' life cycle versus the existing compensation flow for the entire period of increased flow. Nonetheless, there is still a considerable loss of WUA for salmon and in the trout's spawning season versus optimal flow.

Taking ecological effects into account in studies of compensation flow

The context of the French law on fishing

The microhabitat approach was mainly developed in France to meet needs to quantify the effects of flow reductions within the context of the French law on fishing of 29 June 1984 (article L.232-5 of the rural code).

This legislation sets minimum flow at $1/10^{th}$ of mean flow in the river at the dam site, based on reflection conducted in the United States since 1965 and for practical reasons of definition of mean flow (which is easier to define than low flow).

It is therefore important to determine whether this figure is really suitable to meet the objectives of the law, which are to guarantee the survival, circulation and reproduction of fish species. In addition, as concerns dams built before the law was passed, where technical and economic reasons preclude its implementation, what measures must be planned to meet these objectives?

To answer all these questions, the physical capacity of the rivers must be expressed according to discharge, using a suitable tool.

Evaluation of flow levels established by the law on fishing[1]

The microhabitat method was used on eight rivers to measure the effect of a reduction in flow to $1/40^{th}$ and to $1/10^{th}$ on the capacity of the milieu to offer a habitat for brown trout.

Reductions in WUA were expressed versus two references (*see table 2, p. 157*):

- minimum WUA corresponding to average monthly flow in the driest month in the natural situation, flow that is assumed to limit the development of trout populations;
- maximum potential habitat (maximum WUA) achieved for an optimal compensation flow for trout habitat.

$1/10^{th}$ of the mean flow corresponds to a significantly smaller reduction than $1/40^{th}$, but it is still considerable and much less favourable than natural limiting conditions (table 2, p. 157).

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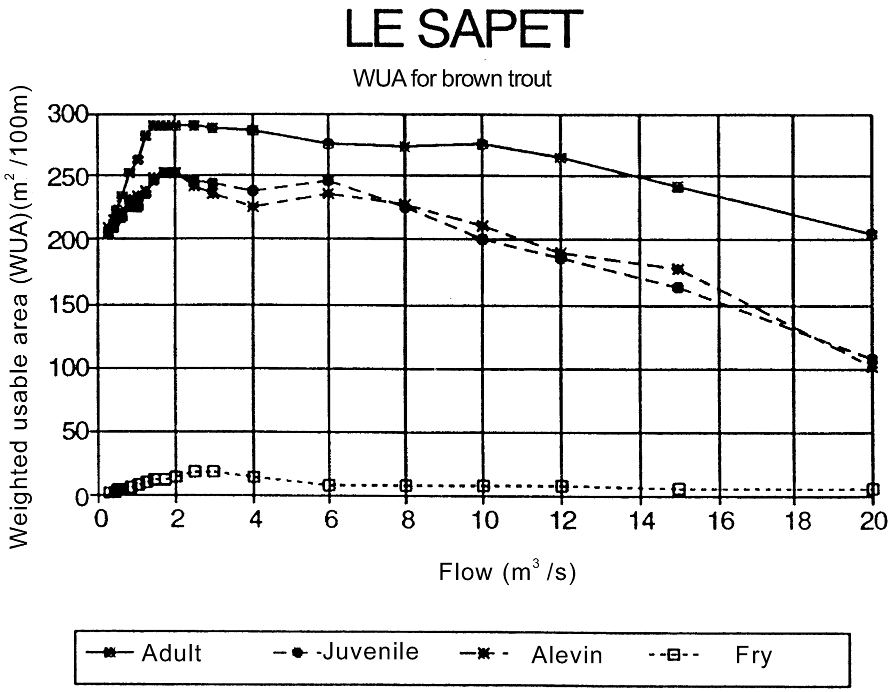
1. See *Bibliography*, reference 14, p. 158.



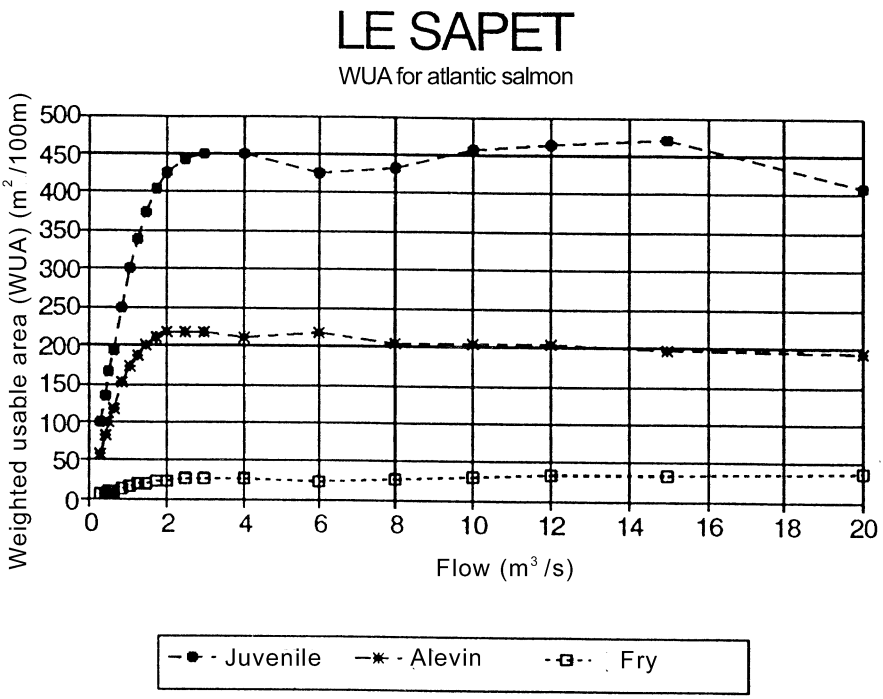


Fig. 3 - Changes in weighted usable areas according to flow for brown trout and salmon in the Allier river, in the section short-circuited by the Poutès dam at the station "le Sapet".

These results highlight the advantages to having precise studies like the microhabitat method to serve as tools in decision-making when establishing compensation flow levels in accordance with the spirit of the law on fishing; the objectives of that law were recently taken up again in the water resources legislation of 3 January 1992.

In addition to characterising habitat reductions at various low flows, it makes it possible to identify sensitive species and stages and therefore the periods that may be critical if flow is maintained too low.

Modulation of compensation flow

It may therefore be proposed in some cases that compensation flow be modulated to limit effects during such critical periods and to recreate a certain hydrological rhythm.

A study by Lignon du Velay was the opportunity to test various scenarios of modulated compensation flow management according to the season, with two or three different levels of flow in the course of the year[1]. Those different flow scenarios were then converted into records of useful weighted surface areas for brown trout in an average year and a dry year.

The main conclusion of that study concerned the summer period, which is the most critical for most of the stages in the existing compensation flow situation. Increasing compensation flow in the summertime would improve habitat potential in this case, especially for adult trout, and all the more so in dry years.

		Optimal Q	% reduction in WUA versus max. WUA at optimal flow		
			$Q=0.5 m^3/s$	$Q=1.5 m^3/s$	$Q=3 m^3/s$
Brown trout	Adult	2.5	23	0.5	1
	Juvenile	1.75	15	2	4
	Alevin	1.75	13	2	7
	Fry	3	79	40	0
Atlantic Salmon	Juvenile	4	63	17	0.5
	Alevin	2.5	54	9	0
	Fry	3	67	27	0

Table 1 - Reduction in weighted usable areas for brown trout and atlantic salmon at various flows at the Sapet station on the Allier river

1. See *Bibliography*, reference 15, p. 158.

	(A) versus the natural situation		(B) versus optimal flowoptimal	
Fraction of mean flow	1/40th	1/10th	1/40th	1/10th
Average % Range (%)	65 42 to 91	32 16 to 50	71 53 to 94	44 22 to 69

Table 2 - Percent reduction of WUA for brown trout in eight rivers according to two references:
(A) natural situation (average monthly flow in the driest month)
(B) optimal flow corresponding to maximum WUA.

Good knowledge of the natural hydrological regime and of limiting factors (low flows) is therefore crucial. In addition to possibly modulating flow, spillage could be planned to clean the substrate without impacting the riverbed.

CONCLUSION: MOVING TOWARDS INTEGRATED MANAGEMENT

It appears important to reason in terms of modifications to the habitat in a river according to:

- morphology;
- the natural hydrological regime (possibly by region) and any tributaries;
- existing populations (the most sensitive species).

Thorough analysis must be integrated into the impact studies very early in order to be taken into account when the structures are designed and the economic analyses are carried out.

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Chapter VII

Life of the dam

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This chapter is not intended to establish a rigid organisation, but to recall the main ideas and general principles that should guide participants in a dam's life.

Above all, the objective is to establish guidelines for planning that will help any dam owner to set up the most suitable surveillance and monitoring method for his dam. But this chapter is deliberately aimed at unspecialised dam owners looking after a small number of dams or even just one. The large dam owners like EDF[1] and CNR[2] have already developed an organisational structure taking into account the number of dams they own and their resources in terms of experienced personnel.

Small dams must be attentively monitored for two essential reasons:

- *to guarantee the safety of people and property;*
- *to maintain the roles fulfilled by the dam (water supply, power generation, recreation, fishing, etc.).*

1. Electricité de France
2. Compagnie nationale du Rhône.

Special features of "small dams"

The special features of low dams are in particular due either to their age or to their modest size, or to the lack of awareness of some dam managers about safety problems.

In particular, for the smallest of them, the following are noteworthy:
- knowledge of how the dam was built that is often too imprecise or too fragmented;
- poor knowledge of the dam's background and history;
- lack (in general) of monitoring systems;
- often rudimentary design and construction, for economic reasons.

Roles of the various participants

The owner (or the concession holder)

The owner is entirely responsible for his dam and for maintaining it in good condition. To this end, he must:

- create and keep up to date a brief containing all the documents concerning the dam;
- ensure surveillance and monitoring;
- maintain the dam and its various discharge structures in good operating condition.

He may award contracts for some of these tasks to an operator or a specialised engineering firm.

The operator

The operator works within the framework of a contract signed with the dam owner that clearly defines the scope of his activity.

As concerns monitoring and maintenance, the owner may consign the following tasks to the operator:
- regular visual inspection of the dam;
- periodic checks on proper functioning of control systems and operation of the discharge structures;
- routine maintenance of the dam and discharge structures, and clearing or cutting back vegetation around the dam;
- periodic measurements using the instrumentation, and checks that it is operating properly;
- drafting the annual operator's report.

The contract should precisely define the nature and frequency of all of these services.

The engineer

The participation of a specialist dam engineering firm is recommended whenever neither the owner nor the operator has the necessary skills. If the operator has those skills, the services described below can be included in his contract with the owner.

The engineer's services are generally the following, under a contract signed with the dam owner:
- report on interannual graphs plotted from the instrumental data;
- interpretation of that data and, where necessary, statistical analysis;
- drafting an annual monitoring report (*see Monitoring reports hereafter, p. 166*);
- participation in in-depth inspections of the dam (*see Technical inspections hereafter, p. 165*).

The inspection administration

For dams in France that are not under the responsibility of the Ministry of Industry, inspection of safety is carried out by the department in charge of the Water Police, the Local Agriculture and Forest Administration or the Local Public Works Department. In the exercise of their mission as the Water Police, these administrations report to the Ministry of the Environment. For dams under the authority of the Ministry of Industry, the inspection is carried out by the Regional Administration for Industry, Research and the Environment.

The role of the inspection department is as follows:
- to ensure that the owner has taken every necessary measure to provide appropriate monitoring and surveillance of his dam, including how the monitoring system is set up, how often measurements are taken, and what technical competence the participants have;
- organising annual inspections[1] and drafting a report on them;
- on these occasions, checking the proper operation of discharge elements with a role in safety and of monitoring instrumentation;
- verifying that the recommendations in the report on the previous inspection have been properly implemented;
- organising, where appropriate, 10-year inspections, in principle after complete emptying of the reservoir.

In no case should the inspection administration be in charge of taking measurements or interpreting them.

1. For dams with an impact on public safety, an annual inspection is obligatory, as are 10-year inspections, if possible with the reservoir empty. For other dams, inspections by the competent administration are recommended, but possibly less frequently. See *Inspection by the administration* hereafter, p. 167.

General surveillance principles

Surveillance of a dam is essentially intended to reveal, and if possible prevent, any deterioration, in order to keep the structure in good condition for safety, and also apt to fulfil its functions.

Efforts will mainly be made to detect any changes. Trends are generally very slow, but the risk of a rapid deterioration cannot be totally excluded, in particular for earthfill dams.

Monitoring should provide the means to detect anomalies, and evaluate how fast changes are occurring and how they will probably end, separating reversible trends from irreversible trends. This is intended to help the entity responsible for the dam to decide on the nature and urgency of the required work.

Precise and reliable measurements are needed. Dam surveillance includes two essential methods:
- *visual inspection,* which is a qualitative method, but nevertheless crucial as it embraces the greatest number of parameters;
- *instrumentation,* which is a quantitative method using specific devices for each dam.

It must be remembered that:
- in application of the French government circular 70/15 dated 14 August 1970, all dams with an impact on public safety, whatever their size or reservoir volume, must be inspected by the administration. This inspection is in particular intended to ensure that the concession holder (or owner) is correctly fulfilling his role in surveillance and monitoring;
- surveillance must include both gathering of information and analysis of behaviour;
- when instrumentation is used, it can only be complementary to visual inspection;
- the smallest dams ($H^2\sqrt{V} < 5$) may have no instrumentation, and in this case surveillance is limited to visual inspection.

However, a dam with no instrumentation should not be a dam with no surveillance.

Methodology for dam surveillance

To be effective, surveillance requires:
- good knowledge of the dam (data on construction, reports on all work and inspections, etc.);
- the guarantee of good maintenance;
- verification of the results of any work done.

Documents on construction

It is necessary to gather all known data about the dam concerning both its construction (surveys, design, construction, as-built documents) and its later life (operation, reports on inspections, reports on maintenance work, typical incidents, etc.).

It is obvious that the creation of this brief is fundamental.

Surveillance plan

For each dam, the special points that will be decisive for safety must be defined, focusing on particular features so that the operator's attention is drawn to the monitoring required for certain special points.

In addition, possible types of damage and their consequences must be inventoried.

Regular visual inspections must be carried out, along with implementation of simple means to evaluate changes as soon as they become apparent (marking, photography, locating cracks *in situ*, etc.). The type of observations will depend on the type of dam.

Monitoring includes two aspects:
- monitoring the general behaviour of the dam (for example, with piezometers for a gravity dam, pore pressure cells for a fill dam, etc.);
- special monitoring of zones that are judged in advance to be sensitive (for example, flow from springs percolating in an abutment, inspection of the stability of an arch dam abutment, etc.).

Whenever measurements are warranted, they should be regular, precise, reliable, and followed up by analyses.

Instrumentation may also be installed, whether temporarily or not, when a change is detected by visual inspection, and also when the stability condition of a dam for which insufficient data is available must be checked.

Organisation of surveillance by the owner

Dam surveillance is carried out by the owner or operator on the basis of inspections.

Visual inspection

Systematic inspections

Inspections should be done once a month if $H^2\sqrt{V} > 50$ and once every two months for smaller dams. They should be more frequent during first reservoir filling: weekly inspections may be recommended, or possibly fewer if the reservoir water level varies little. The contents of these inspections must be defined according to the structures being inspected, and the itinerary is determined to ensure regular inspection of all parts of the dam. For structures with instrumentation, the most simple measurements will be made on these occasions such as reservoir water level, leakage flows, piezometer readings, pore pressure cell readings, Vinchon vibrating wire readings, pendulum readings, etc.

Special inspections

This is the case when the reservoir is lowered or emptied exceptionally. The contents of such inspections must be adapted to the special points to be inspected (structures that are usually under water, river banks, upstream face, etc.).

Exceptional inspections

After a high flood, a storm or an earthquake, the owner or operator must examine any damage and make certain decisions.

If incidents or anomalies are noted on the dam:
- all anomalies should be marked out in order to determine the "starting point" with quantifiable data that can be compared to later data;
- if the anomaly is confirmed, the appropriate conduct should be defined according to levels of urgency;
- where necessary, the frequency of inspections and instrument readings should be modified, as should their contents.

Means required

It is not necessary that the people doing visual inspections be specialists in Civil Engineering. It is more important that they possess qualities of rigour and precision, as well as appropriate motivation.

However, it is important to specify that:
- the person in charge of inspections must have received sufficient information on the dam;

- if necessary, he should be given some minimum of technical training;
- when necessary, and according to the seriousness of what is observed, he should not hesitate to inform technically competent persons without delay (technical services, if any exist, or a specialised engineering firm).

Visual supports during inspections and monitoring measurements

The owner or operator must prepare or have prepared by an engineer:
- guidelines for the inspection, the itinerary to be followed, special points to be observed;
- reports or data sheets on previous inspections;
- a file of photographs;
- an empty sheet to be filled in (being attentive to keep it clear and simple) containing a section for recording monitoring measurements. For these, consistency with previous measurements should be easy to check.

The measurements should be analysed immediately, using graphs that help visualise changes. Technical departments or the consulting engineer can define upper and lower boundaries on the range of normal behaviour. It is essential that this raw information be analysed rapidly.

Any abnormal or doubtful measurement should be checked and taken again before being validated.

These inspection sheets should be filed by the owner or operator in a special place and be both clear and exhaustive, as part of the dam's history.

Technical inspections

A technical inspection is recommended whenever $H^2\sqrt{V} > 5$, by an engineering firm whenever the owner or operator does not have the necessary minimum technical skills. Recommended frequencies are:
- once a year when $H^2\sqrt{V} \geqslant 100$, or whenever the dam has an impact on public safety;
- once every two years whenever $50 \leqslant H^2\sqrt{V} < 100$;
- once every three to five years when $H^2\sqrt{V} < 50$.

Intermediate inspections may be necessary at the owner's request because of exceptional circumstances (high floods, storms, earthquakes).

Among points that require special surveillance, we can mention:
- *the state of dam faces:* cracks, traces of calcite for a concrete dam, collapse or swelling for an earthfill dam, inflows of water;
- *the state of drainage channels:* presence of materials that might come from the drain or the filters;
- *the state of the ground* downstream from the dam: springs;
- *the state of spillways:* no materials obstructing the chute;
- *the behaviour of reservoir banks,* in particular after rapid emptying of the reservoir.

During such periodic inspections, moving structures must be operated and proper conduct of measurements by the responsible person must be checked. A detailed report on the inspection, if necessary including recommendations, should be written, with a copy sent to the inspection department who will return it to the owner along with any comments.

MONITORING

Simple measurements, as indicated above under Visual inspection *(p. 164)*, are taken by the owner's or operator's personnel during regular systematic inspections (every month if $H^2\sqrt{V} > 50$; every two months otherwise, and more often during first reservoir filling).

Any surveying that might be necessary should be done at the owner's request by a specialised firm. No such work is recommended for dams where $H^2\sqrt{V} < 50$.

When $H^2\sqrt{V}$ is between 50 and 200, an annual survey is recommended (levelling only). When $H^2\sqrt{V}$ is over 200, an annual survey is recommended with levelling and planimetry. When the dam has an impact on public safety, two complete surveys per year are required, one with the reservoir at a high water level, and the other at a low water level. Finally, for an old dam with no instrumentation and no impact on public safety, it is not generally of any use to conduct surveying, unless some specific phenomenon is feared.

MONITORING REPORTS

The dam owner or operator writes a report, where necessary with the help of specialists, containing:
- the main data on operation;
- any incidents that have been observed;
- maintenance work done;
- a report on interannual graphs interpreting monitoring measurements and comments on trends in them;
- the inspections carried out.

For dams with an impact on public safety, every two years a statistical analysis of the measurements is required.

The report is then sent to the department in charge of inspection. It must be written with the same frequencies as the technical inspections, meaning once a year to once every five years (*see Technical inspections above, p. 165*).

Inspections by the administration

The annual and 10-year inspections described hereafter are obligatory in France for all dams with an impact on public safety. The inspection administration can take the initiative to inspect other dams when this is judged useful, with frequencies that may be different.

Inspection by the owner and any inspection by the administration will, if possible, be done jointly.

Annual inspection

This inspection is carried out at the initiative of the inspection administration, with the attendance of the owner and operator.

Other participants will be the owner's technical department, if any, or, if not, the consulting engineer.

The inspection includes:
- visual examination of the dam;
- a check on proper operation and proper maintenance of flood discharge and bottom outlet structures, as well as instrumentation;
- inspection of how the recommendations from the previous inspection have been carried out.

A report is drawn up by the inspection department describing the state of the dam and noting any actions to be undertaken. A copy of that report is sent to the owner.

Ten year inspections

For dams with an impact on public safety, the inspection administration organises what is called a "10-year" inspection, although the first takes place five years after first reservoir filling. The following inspections are spaced ten years apart. Each inspection in principle occurs after complete emptying of the reservoir. The lowering of water level must be prepared sufficiently in advance, given the importance of maintaining water supply and the quality of the natural milieu downstream.

For dams with no impact on public safety, special inspections may be carried out at the initiative of the inspection administration, for example when the reservoir is completely emptied, or its level is lowered to an exceptional elevation.

Maintenance

The durability and guarantee of proper operation of structures requires a minimum of maintenance. In fact, dams react to the forces exerted on them: thrust of water, temperature variations, settlement, frost and aggressive water, etc.

In a word, dams age over time.

The owner must set up or have set up a maintenance manual defining the nature of the work required and frequencies.

Those actions will concern the following fields:
- maintenance of access to the various parts of the dam;
- removal of shrub growth from dam faces, channels, drain outlets, and along a 20-metre band downstream from fill dams;
- maintenance of the spillway (repair of masonry joints, removal of trees or branches, materials from landslides or rockslides, etc.);
- maintenance and inspection of proper functioning of monitoring systems (drains, piezometers, pendulum, etc.);
- maintenance and test on the operation of flood discharge and bottom outlet structures.

If any works to modify these structures are envisaged, they must be given prior technical study, which in turn must be submitted to the inspection department (e.g. heightening the dam).

Conclusion

Dam history

The technical brief on the dam should be set up by the owner once and for all and always remain as faithful as possible to reality.

Furthermore, the brief must be filed in a safe place and remain accessible at all times.

It is not only necessary for good analysis of dam behaviour but proves indispensable when unforeseen events occur.

Surveillance

Surveillance is obligatory for any structure with an impact on dam safety.

It is also vital for other structures and proves economically cost-effective as it makes it possible to establish a regular maintenance program:
- to ensure a long service life of the structure;
- to avoid major rehabilitation work that would be inevitable after a long period of slow irreversible deterioration.

Surveillance should be organised by the owner on the basis of clear, simple instructions, and should make it easy to monitor the dam's behaviour.

Training - Awareness

The quality of the visual observations, and therefore dam safety, can only be achieved by sufficient motivation and some technical training for the personnel involved.

For new dams

In addition to the as-built documents, it is important that a report on first reservoir filling be drafted.

Furthermore, any dam should be designed to facilitate maintenance operations and inspections.

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CHAPTER VIII

Conclusion

by Gérard DEGOUTTE

Small dams have many specific features, at every stage in their life: preliminary studies, design, construction, and later surveillance. If we are to be perfectly strict, two types of specific features should be distinguished:

- technical: for example, the lack of any inspection adit for small earthfill or concrete dams, or the technique of drilling a small vertical drain after construction of a small earthfill dam;
- statistical: for example, the fact that small structures may belong to an unspecialised owner or that they generally control small catchment areas.

Without indulging in this distinction, we shall deal with the main points specific to small dams, listed according to the stage at which they are considered.

GEOLOGICAL AND GEOTECHNICAL STUDIES

The cost of preliminary studies is not entirely proportional to the size of the dam, and therefore it is not always possible to undertake all the geological studies that might be desired for a small dam. In this case it may be preferable to abandon sites that appear doubtful, whereas in the case of a large dam the means would be provided to settle any doubt.

In simple cases, on the other hand, it may be interesting and economical to group several types of site investigation in a single stage: for example, it is routine to move a mechanical shovel only once in the case of a small dam that presents no special geological or geotechnical difficulties.

Flood data

It is fairly common to discover that a small stream is completely lacking in gauging stations, thus increasing the uncertainty about flood flows. This must then be taken into account in selecting the design flood. In addition, such uncertainty will lead the designer to prefer certain technical options:

- an ungated spillway, as the increased discharge capacity is greater than that of a sluice spillway when the reservoir water level rises above maximum;
- a very long spillway crest, which decreases the impact of uncertainty on the stability of a small gravity dam.

Design of earthfill dams

The technique of the vertical drain drilled after placement of the fill with a mechanical shovel is advantageous for most small dams, whereas for larger dams, the drain is placed in layers like the rest of the material.

Outlet structures generally take the form of prefabricated piping assembled on site, instead of reinforced concrete structures built with formwork.

Given how easy it is to build, the homogeneous earthfill design is conventionally used for the smallest dams addressed in this volume, even when it results in placing slightly greater quantities. It is rarely selected for dams over 30 m high, as project optimisation often results in a decision to use materials of different characteristics in the core and the shoulders. The simplicity of the profile is therefore favoured for small dams, while the performance of different materials is mobilised for medium-sized to large ones, with clay to provide watertightness and coarse materials to stabilise the shoulders.

Design of concrete dams

Small concrete dams generally have no internal galleries; they are limited to about 15 metres high for conventional concrete and 20 to 25 metres for RCC.

Small dams are rarely provided with drainage in their mass.

It may be acceptable to build a small concrete dam on a foundation of loose material, provided certain precautions are taken.

Spillway design

A very long ungated spillway is preferred to facilitate passage of floating debris.

If possible, all moving parts will be eliminated from the spillway in order to decrease the risk of breakdown and the consequences of insufficient maintenance: small dams are always unattended.

Surveillance

Robust monitoring equipment that is simple to read is preferred. The frequency at which the devices are read, as well as that of in-depth inspections, must be suited to the size of the dam.

All of these decisions combine to fulfil two objectives that can never be dissociated: economics and safety.

In conclusion, a small dam must always be simple in design, simple in construction, and simple in surveillance. But this must not be to the detriment of safety: for example, the filter of coarse material around a drain can never be sacrificed for reasons of economy!

Unfortunately, for the designer of a small dam, simple never means easy. On the contrary, it can be harder to design a small dam than a large one, as the investigations - among other things - are not necessarily adequate.

We hope that this manual will facilitate the dialogue between dam owners, the administration, contractors, and engineers, and will help in reconciling simplicity and safety.