# ICOLD

# COST SAVINGS IN SPECIFIC DAMS

**Bulletin n° 152**

# CONTENTS

# FOREWORD

The Committee has been appointed in 2006 for reviewing past Bulletins on Dams Costs and lessons from Question 84 of Barcelona Congress.

The Committee has prepared a Bulletin n° 144 “Cost Savings in Dams” approved by ICOLD in 2009 and now published.

The present Bulletin is devoted to “Cost Savings in **Specific** Dams” and has been approved by ICOLD in 2011.

Possibly half of future dams investments will be devoted to dams to be built in specific conditions for which design criteria, specifications and construction methods may be very different from other dams and are analysed in the Bulletin.

As examples:

- Low dams in very large rivers
- Offshore dams
- Very low embankment dams
- Dams to be built in difficult climatic conditions
- Dams devoted to floods mitigation.

Mrs BIGRAS, Mr MACHADO, Mr NOMBRE, Mr HAZRATI, Mr KUSUMI, Mr MASON took a great part in drafting this Bulletin which has been reviewed and translated in French by Mr HO TA KHANH.

They all deserve our thanks for their heavy task and their views about challenging subjects.

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## Introduction

The ICOLD bulletin 144 has been devoted to cost savings in Dams, including 2 parts:

- Identifying and mitigating existing factors detrimental to Cost Savings such as procedures, set ideas, unadapted specifications …

- Technical opportunities for innovation and cost savings in the design of high and low dams built in usual conditions.

The present bulletin refers to such technical opportunities for innovation and cost savings when they apply to special dams for which the specific conditions may require or favour unusual criteria or designs and / or construction methods. They may be required by environmental problems, climatic changes or local physical or economical conditions. Such specific dams may in the future require over half of all dams investments. They may include:

### 1. Low head dams in very large rivers

For various reasons, and especially for reducing resettlement, the hydropower equipment of many large rivers may well in the future avoid high dams and relevant very large reservoirs and use mainly run of river power plants with heads under 20 m with average discharges over 1000 $m^3/s$ and exceptional floods over 10 000$m^3/s$. They will include large low structures for the spillway and the power house and possibly for a navigation lock. Long embankments may also be necessary. The lay out may be linked mainly with construction problems and / or sedimentation mitigation.

### 2. Dams used only for flood mitigation

Such reservoirs will be quite always empty and thus more easily acceptable for environment reasons. These dams may accept important leakage and the usual design criteria may be reviewed in depth. Unusual low cost solutions may be possible. Large savings are also possible for relevant spillways.

### 3. Offshore dams

They may be in the future three opportunities of offshore dams:

- Large tidal plants which will be economically justified by the increased cost of power. They will require low head dykes and power plants along tens of km for each site. The design criteria and technical solutions will be very different from the onshore traditional dams. The waves and construction methods using floating equipment will modify traditional designs.
Ten to twenty countries may be involved: The realistic world tidal potential is between 200 and 500 GW (the present hydropower generation is 800 GW).

- The foreseen huge world increase of electricity, including renewable intermittent energies such as wind, sun, tides, waves, … will require enormous capacities of P.S.P. (Pumping Storage Plants); it may be difficult in many countries to find relevant onshore sites operating between two lakes, as over 100 GW operating presently worldwide. There are cost effective opportunities of large schemes using as low basin the sea and as high basin an onshore dam or basin or an artificial lake created offshore. For large schemes the cost per KW could be close to the cost of onshore traditional P.S.P.

- Protecting shores against cyclones, tsunamis and increase of sea level is less expensive onshore than offshore but is not always possible. There may be offshore justified opportunities in the future, possibly associated with large tidal plants.

**4. <u>Onshore basins for Pumping Storage Plant (P.S.P.)</u>**

For environmental or cost reasons, it may be difficult in many countries to build P.S.P. basins along rivers. The design and construction methods of P.S.P. reservoirs built beyond river beds may be very different from traditional dams.

**5. <u>Dams with significant siltation problems</u>**

A chapter of the general bulletin 144 is devoted to this very important problem. Further analysis is presented below.

**6. <u>Very low earthfill dams</u>**

There are worldwide over 100.000 dams lower than 10m but with storage between 0,1 and 10 $hm^3$ and sometimes up to 100 $hm^3$. They are essentially used for irrigation and drinkable water and may be very important for close riparian people, especially in Asia and Africa. They are usually earthfill dams with free flow spillways.
Cost optimization should take in account some specific data:
- The impacts of possible failures are usually very reduced.
- The quality of embankments is now much better than in the past.
- The cost of spillway is a large part of the overall cost.
- The average depth of reservoir is in the range of 2 m and there are often huge losses by evaporation; any increase of storage level is essential.

Design criteria and solutions may thus be much different from those prevailing for higher dams, especially for floods management.

**7. <u>Dams built in very cold weather</u>**

Twenty per cent of hydropower untaped potential is in areas very cold most of the year: the best designs and construction methods may be very different from the usual ones and contractors views may be very useful for relevant choices.

**8. <u>Earthfill dams built in tropical conditions</u>**

Tropical conditions apply to about 20% of the world area and materials available for earthfill dams may have a much too high water content; it may be less expensive to adapt the design to such material than to try to reduce the water content.

## 9. <u>Upgrading existing dams</u>

Upgrading will be necessary for many existing dams for:
- Mitigating ageing impact.
- Improving safety, especially for extreme floods.
- Increasing storage or head.

## 10. <u>Conclusion</u>

# 1. LOW DAMS ON VERY LARGE RIVERS

In many parts of the world, low dams in very large rivers have been developed mainly for the generation of hydroelectric power and head control for inland navigation. These dam projects were and are being built, in most cases, in the downstream stretches of large and long rivers where the useful head is generally small but where the continuous flow resulting from the drainage of large basins compensates the small head. Projects in such rivers include dams and dykes developing heads from 5 metres (or even less) to 15 to 30 metres. In many cases these projects combine hydroelectric generation and navigation facilities, incorporating locks for barges and boats to cross the dam obstacle.

Projects of this kind exist in many parts of the world. Examples of these were or are, in Europe projects on the lower part of Rhine and Rhone rivers, in North America on the Mississippi river, in South America on the large rivers draining the Amazon, Paraná and Caroni basins, and in Asia, among others, the major projects on the lower part of the Yangtze in China and Narmada in India.

<u>Project characteristics</u>

Low head dam projects usually require the construction of dams and appurtenant hydraulic structures which present specific problems in terms of design and construction planning. These undertakings include low or very low dams and relatively very large spillways able to discharge the large flood flows of these sites.

In projects built in the large rivers of South America and Asia, where both the diversion and spillway floods are of the order of many thousands of cubic metres per second, the project layout, as a rule, is formed by a sequence of a power-plant equipped with a large number of generating units, a gated spillway with a large hydraulic jump stilling basin and a earth-rock or concrete dam closing the section. In general the structures are disposed along the project axis in such a way as to comply with a feasible scheme for the river diversion and control during construction, which requires the handling of very large flows. Examples of this kind of projects are in South America, the Yaciretá Project on the Paraná River, in Argentina, Tocoma Project on the Caroni River in Venezuela, and the Santo Antonio and Jirau Projects on the Madeira River in Brazil.

As a result of these factors the arrangement of the structures of the project is, in general, primarily dependent on the river diversion scheme and on the corresponding planning of the construction. It is difficult to define a “conventional” type of scheme for this operation but the usual solution includes at least two phases, starting with the cofferdamming of part of the width of the river – which is generally very large – and the construction inside the dried area, of the spillway (or part of it), through which the river will flow during the second phase of the diversion and the power-plant (or part of it). The need to proceed with the construction of the power-plant from the initial time is an economical requirement to allow hydroelectric generation as early as possible, since otherwise the cost of financing would have a very important impact on the project costs. As a rule, since the aggregated length of spillway and power-plant is significant it is normally very difficult to achieve a dried area compatible with this length in one of the river margins. Therefore a common situation is to locate each structure in either margin. Again this may create a logistic problem because access to both margins may not be available at the initial phase of the construction and crossing of a large river with construction equipment and materials may not be an easy task. One rather conventional solution is to divide the spillway into two parts and using the first built part, contiguous to the power-plant as a second phase diversion conduit. While construction is carried out in one margin, preparation of accesses to the other margin proceeds and a reduced construction time, with corresponding savings, is achieved.

An interesting example of the economic importance of the schedule in a large low head project is the Santo Antonio Project, in the Madeira River, in Brazil. It is a 3,150-MW project, formed by 24 units with nominal capacity of 73,28 MW and 20 units with 69,59 MW. The project utilises the head of 13,9 m created by the dam. It contains 44 generating units of the bulb type each driven by horizontal-shaft turbines. The layout placed the various structures in a linear sequence but considered three separated powerhouses, one near the right bank, one near the left bank and one in the middle of the river. Since the project head is low, and its dams are of course of modest height it will be possible to generate power (and income) before the third powerhouse is completed being it protected by cofferdams.

Other low-head projects in large rivers used different approaches to develop the project layout. Several schemes on the Rhone and Rhine rivers in France were formulated with a low-head dam to divert the flow into a long channel roughly parallel to the river main course (in some cases 10 to 20 km long), reaching sites where a power-plant could be placed using a 15 to 20 metre head. This type of layout allows the construction of the power-plant and, in some cases (Rhone) of the spillway in dry areas using very low dykes to control the river. This type of solution favours the schedule and was very important in the implementation of these projects. However the present-day application of these concepts in other parts of the world must take into account the additional costs required by environmental works to compensate the reduced flow in the river main course. Nevertheless it has been successfully applied, as in the Aimorés Project, in the Doce river, in Brazil [1].

Low head hydroelectric projects rely, of course, in large flows to compensate for the reduced head. This requires large prime-movers (turbines) generally of the vertical Kaplan (for large unit outputs) or horizontal or inclined Bulb, Pit or Tubular type for smaller capacities. These features become an important cost issue in a project and must be examined with care, especially for small projects in the range of 1 to 10 MW per unit. The new Orthogonal turbines, as developed in Russia, could be cost efficient for heads of 2 to 3 m.

One of the major costs in a low-head project in large rivers is the gated spillway necessary t o control the large spillway design floods. Ungated spillways would require very long weirs, often difficult to fit in a project arrangement. A possible alternative is the use of labyrinth weirs with a longer dam and a much lower cost per m. it may be especially interesting if the reservoir level is not higher than the natural level of extreme flood. So far this type of spillway has been successfully built up to design floods of 15,000 $m^3/s$ (Maguga Dam, Swaziland and Ute Dam, USA). This however does not necessarily limit its applications to larger flows. In this question, it may be relevant to consider the criterion to define the spillway design flood: quite often the reservoir volume created by a low dam is not very large when compared with the volume of the natural incoming flood, and therefore, it may be possible to design the spillway without considering the PMF which tends to increase substantially the cost of the structure and prevent the use of alternatives as mentioned above. However, country regulations and environmental requirements may affect the corresponding decisions. Reference could be made to incremental damages.

Another cost-sensitive issue in the formulation and design of low-head projects in large rivers is the management of sediments and floating debris carried by the river. Sediments and silt generally can pass the dam barrier through the spillway passages, which have very low sills. However the issue must be examined in depth and sometimes special features may be necessary to be built, and this may impact the arrangement of the project structures including the general lay out.

Floating debris are much important in low projects in rivers built in tropical forest areas. The management of floating tree trunks, foliage and similar debris has required special structures in the Madeira River projects in Brazil, which are being built in the Amazon region. This of course affects the project layout and cost, and must be considered for each individual project and no general solution is possible.

River diversion and control during construction

The simplified observations above are included here to emphasise the fact that the most important cost issue in these types of projects is the interaction between the designer and the constructor. The impact of reducing concrete or excavation quantities is generally secondary in comparison with the possible savings associated with a project design fully coherent with the construction planning, which of course includes scheduling and adequate construction equipment selection and use. The planning will be fundamentally dependent on the river diversion and control scheme.

The cofferdam for the first river diversion phase is generally an earth-rock dike placed from the margins forming a semi-ring encompassing the area to be dried for the construction of the structures. Since most of large rivers have a marked flood season, the construction of the cofferdams start a the beginning of the dry season to allow reaching final height necessary to protect the area against the required first phase diversion flood (normally between 25 to 100 year recurrence period, depending on the risk analysis of eventual overtopping). If more than one cofferdam is required, for instance, if the construction of structures is needed to start simultaneously in both margins, both cofferdams will generally be of same type. The river space left to accommodate the natural river flow should be sufficiently protected against overtopping and erosion by the increased flow velocity in the constricted section. Economic considerations should compare the costs of cofferdam height, erosion protection means and the risk of providing correction to failures of not achieving adequate protection. It is normally advisable to carry out hydraulic model studies to define the best and more economical solution, especially if the bottom of the river is lined with sediments. This phase of the works carries a substantial risk of cost and schedule overruns and for that reason must be carefully designed and planned.

The second phase diversion will, as a rule, be done through the spillway structure using either part or all of gated passages.  For different projects different solutions have been used. In any case, the crest elevation of the spillway sill must be as low as possible not to affect the elevation of the upstream water level and of course of the upstream cofferdam. With very large flows and low dams even the elevation of permanent works may be surpassed if the upstream water level is affect by excessive constriction of the flow passage combined with relatively high spilling sills. To avoid that it is common to leave part of the surface gated passage sills partially built during second phase diversion, to be completed after the dam is finished. This solution requires very strict planning and of course attachment of construction activities to the planning.

The other alternative also often used is to have the spillway designed to operate as gated bottom outlets (orifice spillways), where the spilling sill is set at a very low elevation. This is the solution used, for example in the Tocoma and Caruachi projects, in Venezuela, for diversion flows of the magnitude of 14,000 $m^3/s$.

REFERENCE

[1] -------, "Aimorés Hydroelectric Powerplant in the Doce River" in Main Brazilian Dams, Vol. III, pgs. 19-32, CBDB, Rio de Janeiro, 2009

## 2. DAMS USED ONLY FOR FLOODS MITIGATION

### General

Many dams used for flood mitigation have multi-purposes such as hydropower or water storage; their design would need to reflect all these requirements and not focus only on flood mitigation.

A dam devoted only to flood mitigation has a number of advantages:

- Environmental impacts are low as the reservoir area is drowned only rarely.
- It prevents the flooding of much larger and more critical areas downstream.
- The design may be simplified because some leakage and/or settlement are acceptable.
- It is almost always available for complete inspection and because of this repairs are also easy.
- Because of these factors and because it is associated with saving lives and property it is likely to be more easily accepted.

However such dam built often close to populated areas should be extremely safe and it is not always possible to test such a dam by a progressive filling. The flood criteria to be applied to such works needs careful thought given that the dam is likely to be immediately upstream of populated areas and valuable infrastructure. Robust solutions and details are also needed given that the works are generally exposed near populated areas and hence potentially prone to vandalism.

Flood prevention dams across rivers will generally incorporate culverts or low level outlet containing gates capable of limiting downstream flows to those that the downstream urban development can accommodate. This may typically be the 1 in 10 or 1 in 20 year flood. Floods larger than this will go into temporary storage in the flood relief reservoir area. Depending on the importance of the downstream infrastructure the dam and temporary flood storage reservoir may be designed to accommodate floods up to the 1 in 100 or even 1 in 200 year events. Larger floods will then have to be discharged. It is usually not economic to provide large low level outlets for this purpose and so the most common practice is to allow the dam to overtop in designated locations.

It will be important to design an adequate spillway capable of accommodating the design flow such that the dam does not fail and increase the flood by releasing the stored water. The choice of design flood will depend on the importance of the urban conurbation downstream which is being protected but in many cases may be as high as the probable maximum flood (PMF).

Many future reservoirs will be rather small but the need of larger ones will also increase for three reasons:

- Many large cities will be built or will increase along rivers, especially in developing countries.
- As populations develop they demand increased security and safety.
- The value of housing and infrastructure will continue to increase.
- Climatic changes may increase the floods volumes and/or peaks.

Flood prevention has often been by conventional dams but comments are given below for possible cost savings for small or large specific dams. They are different for earthfill dams, rockfill dams and concrete dams. Heights will vary from fairly modest ones to significant works between 15 and 50m or even higher.

## 2.1. <u>Earthfill dams</u>

The water-tightness of the dam and the foundation are less critical, provided that the integrity of the works remains and that secure operation occurs when needed. In such cases it may be possible:

- To accept higher permeability materials for the core, for example silty or sandy clays, if they are more readily available on the site and provided they can be shown to remain stable when impounded.
- To accept higher permeability foundation cut-offs, again provided they can be shown to remain stable when impounded. This may enable the use of simple cut-off trenches, diaphragm walls or short clay blanket upstream the dam, coupled with relief wells at the dam toe.
- As it is often impossible to control the first filling of a flood storage reservoir, particular care should be taken concerning any filter and the transition zones, which are the key safety aspects. For lower dams, it may be possible to replace costly filter and transition materials by geotextiles with due care of futures roles.
- In stability analyses, any "rapid drawdown" calculations may be able to take advantage of reduced (partial) core saturation rather than total saturation, given the transitory nature of the design floods.
- A combined probability assessment may mean that the stability of the dam during an earthquake need not be associated with a full reservoir.
- The reservoir being empty most of time, all repairs during the operation, for the dam and the appurtenant structures, can be easily and economically realized.

An example of application of the above suggestions is recently provided by two flood mitigation dams in Morocco (El Gam and Tamedroust) where significant cost savings were obtained.

## 2.2. <u>Very low Earthfill dams</u>

In the UK there are many flood storage reservoirs and these are generally of much smaller in nature, say less than 10m high. In such cases existing river channels through major towns or cities may only be able to accommodate 1 in 10 or 1 in 20 year floods without bank over-topping. Embankments would be built across the rivers immediately upstream and incorporating gated culverts. Water level centres in the city would then trigger partial gate closure during large flood events such that flows were throttled down to those which the river channel could accommodate. Surplus flows would go into temporary storage upstream of the embankment. This may apply also to many other countries.

In such cases the embankments would be sited so as to temporarily flood sparsely populated areas such as recreation fields, parkland or farmland. The works would typically be designed to accommodate floods up to the 1 in 100 or even higher event, depending on the importance of populated areas downstream and the amount of available land.

RCC stepped slabs have been used on the downstream faces of earthfill embankment dams to permit safe overtopping.
It may be possible also to provide a secure overtopping crest using "grasscrete" type blocks. These are open texture concrete blocks filled with topsoil and grass and they are also sometimes cable-tied. Downstream spillway faces can be formed in a similar manner or more simply with geotextiles nets embedded in the topsoil. Safe water velocity tables are available for both forms of construction. In other cases open textured asphalt has been successfully used coupled with grassed topsoil to improve visual appearance. In all such cases the security of the spillway will depend on good grass cover and an established root system. It is important to monitor such works for disease and treat the grass where necessary to prevent this, also to ensure the works are not disrupted by burrowing animals.

### 2.3. Rockfill dams

Waterproofing by an upstream geomembrane may be a cost effective solution. Some settlement will be acceptable, favouring the utilization of medium quality materials for the dam body and geomembranes for imperviousness. However as these types of dams are generally associated with nearby populations, such membranes should be preferably buried under slabs or bedded rockfill so as to be vandal-proof.

### 2.4. Concrete dams

Arch dams are usually not cost effective for the reduced height of most future flood mitigation dams. Gravity dams should be very safe against the risk of very exceptional floods and with some overtopping being acceptable.

Some leakage will be acceptable and provided the works can be shown to remain stable, any foundation treatment should focus more on efficient drainage relief rather than any upstream waterproofing or cut-off.

The Willow Creek dam in the USA was one of the first roller compacted concrete (RCC) dams to be built and served the prime purpose of protecting the community downstream from floods. A lean, low paste RCC mix was used with upstream concrete shutter panels but no specific waterproofing. The dam proved to be highly porous but, given that incoming flood were still put into temporary storage, this did not matter.

A very attractive solution may be a Symmetrical Hardfill Dam using low cost materials. As with Willow Creek dam, for flood relief purposes an upstream facing is not necessary. It may withstand significant overtopping and does not require a very good rock foundation as foundation stresses are low and evenly balanced under all conditions. Such dams also have excellent seismic properties and are discussed and analysed in ICOLD Bulletin 117: "The gravity dam, a dam of the future".

### 2.5. An example of low cost concrete solution

- An attractive solution could be based upon a Symmetrical Hardfill Dam, for instance with upstream and downstream slope of 0,7/1. This dam could be limited to a part of the dam length and associated to an earthfill dam for the other part.

- The floods smaller than the 100 year flood should be managed by a vertical breach along the full cross section of the Symmetrical Hardfill Dam (fig. 1). The width of the breach shall be chosen according to floods data. The sill of the breach may be at the ground level, it may be also somewhat higher creating thus a small recreation lake with a very small reduction of the reservoir capacity.

- Floods larger than the 100 year flood may be managed by a free flow surface spillway. Its cost may be reduced if using a labyrinth shape. It may be quite nil if using very simple concrete fuseplugs which may tilt before or after overtopping and be replaced easily.

- The cost of spilling devices investment and operation is thus quite nil.

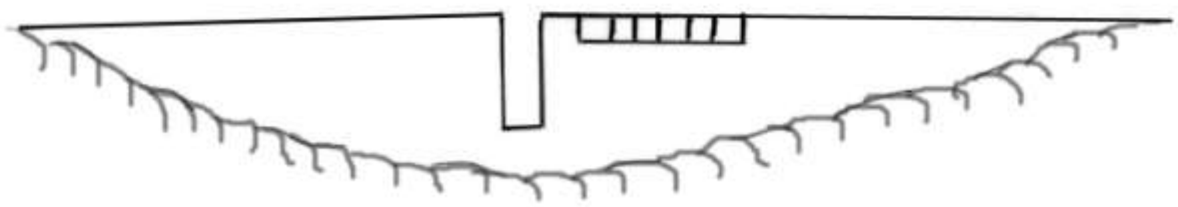


Fig. 1

**<u>Dam longitudinal section</u>**

## 3. OFFSHORE DAMS

There are presently few large offshore dams; they may have two targets:

- Tidal energy (such as La Rance in France since 40 years and Shiwa in Korea from 2010 totalling 1 TWh/year and 100 km²).
- Shore protection totalling some hundreds km such as the huge Netherlands achievements made 30 and 80 years ago or St Petersburg and Venice present works.

Much more important achievements are likely within the next decades for these two targets. Another opportunity may be the energy storage for which future needs may be very important in association with renewable intermittent energies (wind, solar, tidal, waves, …); the onshore energy storage possibilities are limited in many countries and offshore solutions cost effective.
It seems thus useful to analyse the offshore specific constraints and opportunities and the construction conditions and methods in order to help comparing the possible designs: these designs may be very different from the usual designs for onshore dams.
The analysis is made separately for:

- Tidal plants
- Shore protection
- Energy storage at sea (needs of storage and onshore solutions are analysed in chapter 4.

### 3.1. Tidal plants

#### 3.1.1. Potential

Some tidal energy may be supplied by turbines placed in tidal streams, but this solution has a rather low cost effective potential, does not require dams and is not studied here. The most important potential is based upon huge discharges under low head between the sea and large basins closed by offshore dams.
The data below are given for semi diurnal tides which are by far the most usual case for the likely future tidal plants. Other tidal conditions should modify these energy data but would not modify significantly the relevant dams problems and solutions.
The theoretical energy available during a tide of 12 hours 25 minutes is obtained if the basin filled at high tide is completely emptied through the turbines when the sea level is lowest and six hours later is completely filled through the turbines when the sea level is close to maximum. For a tide range H (in m), and a basin area S (km²), the corresponding energy is calculated by multiplying for each <u>half tide</u> the water volume SH by the average head 0,5 H, by g (about 9,8 m/s/s) and by 1,03 (sea water density). It should be multiplied by 2 x 705 (yearly number of half tides) and divided by $3.600 \times 10^6$ for expressing it in GWh.

The theoretical potential per km² is thus about (GWh/year):

$$\frac{2 \times 705 \times 10^6 \times H \times 0{,}5\, H \times g \times 1{,}03}{3.600 \times 10^6} \# 2\, H^2$$

for an average tidal range H (in m).

The total ocean area where tidal range is over 4 m extends along 10 to 20.000 km and 20 to 50 km from shore, i.e. possibly 500.000 km² with an average tidal range close to 5 m.
The theoretical potential is thus in the range of $2 \times 500.000 \times 5^2 = 25.000$ TWh/year. It is close to the theoretical onshore hydropower potential but it is possible technically to use only a minor part (10to 20%) of the theoretical tidal potential of a geographical area and for

environmental or economical reasons a part only of this technical potential will be used; this suggests a likely implementation under 10% of the theoretical world potential. Analysing the potential of main tidal countries suggests also a realistic economic potential between 1.000 and 2.000 TWh/year, i.e. some 20% of the onshore hydropower likely implementation.
The possible energy supplied per km² of basin is between 25 and 50% of the theoretical potential according to the operation method and to the plant capacity and efficiency. It is thus yearly (in GWh/km²) between 0,5 $H^2$ and $H^2$. For a likely value for implemented sites between 4 and 8 m the yearly energy per km² will be between 10 and 40 GWh/year and 25 as average. This figure is close to the yearly energy which may be supplied by offshore wind farms (10 MW x 3.000 hours = 30 GWh/km²). The two energies may be associated in many places with common energy storage facilities.
This energy per km² of tidal basin is higher than the average yearly energy from present onshore hydropower (3.000 TWh/year for 300.000 km² of reservoirs, i.e. 10 GWh/km²).
The total area for future tidal energy may be in the range of 50.000 km² for 1500 TWh/year.
Seven countries: Canada, Russia, Australia, China, India, France and Argentina have each a realistic potential in the range of 100 TWh/year or more. Many other countries have a significant potential: U.K., U.S. (Alaska), Chili, Korea, Myanmar …

The increased cost of world energy and the need of renewable sources will favour tidal energy. The total tidal investment, to be made probably along the next fifty years may be in the range of 500 billions U.S. $ (10 billions $/year). Most will probably be in huge schemes in the range of 1.000 km² and 10 billion $ each such as Mezen in Russia (2.000 km²), Chausey in France (1.000 km²), Severn in U.K. (500 km²), Kutch Gulf or Khambat Gulf in India (2.000 km²), Fundy in Canada (2.000 km²). The total length of offshore dykes will be some thousands km (including 500 km of power plants). Preliminary schemes in the range of 100 km² are likely.

There are various solutions for the basins operation and for turbines. For environmental and cost reasons an attractive solution could well be operating both ways a single basin. This method of operation keeps within the basin tidal conditions similar to the natural ones (tides being shifted by 2 hours) with a tidal range reduced by about 20% for spring tides. Basins may then be along the shore. Orthogonal turbines as developed in Russia seem very well adapted to this solution; Bulb units may be optimised and cost effective where the tidal range is high (fig. 1).

The conditions for design and construction of dykes and plants are very different from the usual ones for traditional onshore dams.

#### 3.1.2. <u>Constraints and solutions</u>

Some conditions are more difficult than onshore:
- Waves impact for permanent structures and during construction
- Foundation to be made under water, at least for a part of the structures (the existing foundation in tidal areas is usually sand or gravel or rock, the water depth in the range of 10 to 30 m).

Some conditions are more favourable for design and construction:
- The water head on structures is low
- Some leakage is acceptable: a leakage of 100 $m^3$/s/km would reduce the energy supply by under 1%.

- There are limited damages and very few human risks from possible failures.
- Huge prefabrication and easy transport and erection of electromechanical equipment are favoured as well as extensive use of cost efficient large equipments.
- No resettlement is necessary.

Small schemes using favourable bays may be built in the dry by cofferdams and by rockfill dykes but most energy will be probably from huge schemes with 10 to 100 km of dykes closing very large basins along shore. This closure will include the power plants and ordinary dykes.

Power plants : The most probable solution is by prefabricated caissons of about 100.000 tons each (100 m x 50 m) with a power capacity in the range of 50 to 100 MW and an yearly energy of about 200 GWh/year (2 GWh/m).
Caissons may be prefabricated far away; electromechanical equipment may be placed during prefabrication or in situ (this last solution being more likely for schedule reasons).
The stability will be favoured by the structure width (in the range of 50 m) required for the hydraulic efficiency of turbines. The two specific problems are the waves protection and the waterproofing under the caissons; however their cost will probably be a small part of a total investment in the range of 1 x U.S. million $/m and there are various solutions (grouting, sheetpiles, … various concrete protections …).

Ordinary dykes will be usually longer than the power plants; their cost may be 50.000 to 100.000 U.S. $/m.
Various solutions are possible:
- Prefabricated smaller caissons.
- Rockfill dykes with concrete blocks protection. This solution may appear less expensive but requires huge quantities of rockfill and the construction and waterproofing in rather deep open area may not be easy.
- It may be cost effective and favourable to the construction schedule to build first a traditional breakwater in caissons (possibly overtopped by high waves during high tide) and a separate dyke built in calm water (for instance 100 m behind the breakwater). This dyke may be built at low unit cost in sand and gravel by large sea dredges. Its waterproofing may be made by traditional slurry trenches or even as a wider dyke operating under low head. This solution favours low cost harbours if increasing locally the width between the breakwater and the impervious dyke.

Basin closure

The water level in the basin may be kept close to the sea level during construction, with the necessary flow through the power plants in by-pass mode. Sluices are probably not necessary for the two ways operation of a single basin, all discharges between sea and basins being through the plant.

#### 3.1.3. Cost efficiency

The main parts of the cost of tidal energy are:

- The plant investment. The direct cost per KW (including civil engineering) seems between 1.000 and 2.000 U.S. $ for a yearly supply corresponding to 2.000 hours of full power as Shiwa and over 3.000 hours for a both way operation (Mezen in Russia). The investment per KW is very roughly proportional to 1/H, H being the average tidal range,; the utilization of tides under 4 m seems thus unlikely. The investment per yearly KWh may vary from 0,3 $ to 1 $.

- The dykes of which the cost per km seems be in the range of 50 to 100 millions $. For keeping this part of investment under 0,2 $/yearly KWh, the length of dykes per km² of basin producing 25 GWh/year should be in the range of 0,1 km. The most attractive future tidal sites will thus be in favourable bays as in China South of Shanghai or Australia or Argentina or in huge schemes (for instance Mezen in Russia with 80 km for 2.000 km², Chausey in France with 50 km for 1.000 km², or Kalpasar in India).
- The structures (locks) for sea traffic which may be important (Severn in U.K.) but most often a small part of the total investment.
- The sluices which are a significant part of the investment for some designs but which are not necessary for a single basin operating both ways.

A total direct investment in the range of 0,5 to 1 $/yearly KWh is likely; the tidal energy which was in the past more expensive than fossil fuel energies may have a great future for a cost per KWh of 5 to 10 cents of $. However the energy is intermittent and 2 cents should be added for corresponding energy storage.
Most very large schemes will probably be implemented after 2030 but some schemes in the range of 100 or 200 m² may be implemented earlier.

## 3.2 <u>Shore protection</u>

Onshore corresponding embankments are much less expensive than offshore dykes but, for various reasons, are not always possible: offshore dykes or specific structures have been implemented for large parts of Netherland or for avoiding huge possible damages to St Petersburg or Venice. Associating long onshore embankments and local offshore structures may be a solution.
Many more achievements should be necessary for avoiding damages from typhoons or tsunamis. And the sea level likely increase along the century will raise enormous problems. Optimization of existing technologies and utilization of increasing resources in developing countries may help to solve these problems.
A part of these works will be essentially based upon breakwaters technology with little need of dams knowledge. However associating the two technologies may be useful, for instance for associating large tidal basins with shore protection.

This may be justified for economical and physical reasons:

- Large tidal basins operation may avoid dangerous high water levels onshore, including from long term sea level increase.
- A limited increase of tidal investments may avoid onshore damages from typhoons and tsunamis; for instance the level of tidal dykes built behind breakwaters may be raised at low cost.
- The often prevailing conditions in tidal areas (limited depth with sand, gravel or rock); favours offshore structures for shore protection.

As examples, it is likely that such solutions may apply before 2050 to China (North of Shanghai), in the Gulf of Bengal where tidal schemes with shore protection may perhaps be associated with long parts of onshore embankments. River floods through the closure could be managed by sluices. Low head huge pumping stations may perhaps be a solution keeping the fresh water level at the best level and avoiding salt water (Mekong in Vietnam).

### 3.3. Energy storage at sea (Pumping Storage Plants: P.S.P.)

Onshore Pumping Stations Plants (P.S.P.) or P.S.P. using the sea as low basin and an onshore high basin may be the best solution. Using the sea for both reservoirs may also be a solution.
The sea may be used as high basin or low basin:

- There are in many areas with a significant tidal range large flat areas with a water depth of 20 to 30 m and sand, gravel or rock foundation. Creating an artificial atoll by a caissons breakwater and an inside dyke for lowering water 20 or 30 m under the sea level does not require new technologies: the inside dyke may be built at low cost by sea dredges and waterproofed by a slurry trench. The energy stored per km² for a basin operated between 15 and 25 m under the sea level will be 0,6 GWh. A 15 km diameter basin could store 100 GWh with for instance 4 GW generating capacity. The cost for 50 km of breakwater and dykes would be in the range of 3 to 5 billion $, i.e. 1.000 $/KW to which would be added the cost of the plant itself (probably with bulb units) for 500 to 1000 $/KW with such head.
  The cost per GWh of this solution is higher for smaller schemes because the cost is proportional to the diameter and the energy proportional to the area. However smaller schemes may be cost effective within tidal basins for storing few hours of tidal energy if the cost of the breakwater is paid by the tidal plant. Using for a storage basin few per cent of the tidal basin area and building relevant dykes in calm water behind the tidal breakwater would increase the tidal investment by about 20% for getting an energy supply better adapted to needs.

- It is possible in many places to use the sea as low basin and to create a high basin within an artificial atoll which may be along the shore with rather limited depth. In a cliff area the cliff may be a part of the basin closure (fig. 3). The stored energy per km² of basin is 2 GWh for a basin reaching 50 m over the sea level and 8 GWh for 100 m.
  The best solution may be to build first a breakwater then the high dyke in calm water. The economical feasibility is linked with the materials availability and the foundation conditions. Tidal areas will usually be favourable with sand, gravel or rock foundation and large resources in sand and/or gravel.
  A possible cross section is represented in fig. 2 with an upstream lining and a sand and gravel dam body. There are presently worldwide 20 similar dams 100 m high in operation or construction. If the dam body is mainly sand, the downstream slope and the drainage will be carefully adapted. Materials for the dyke may be brought by large marine dredges and raised by conveyor belts with possible intermediary screening.
  For storing 100 GWh with a 3 GW plant and with semi circular basin along a cliff, the necessary length of breakwater and dykes will be 17 km for a 100 m high basin and the double for a 50 m high basin. The volume of materials for the dyke will be about 300 millions $m^3$ for the 100 m high basin and 200 for the 50 m high basin. The overall investment per KWh will be about the same, in the range of 1.000 $/KW. The cost of the plant itself should be added; with a total investment per KW between 1.500 and 2.000 $.

It seems thus that, in most countries, it will be possible to have the necessary energy storage at an acceptable cost, either onshore or offshore.
Associating offshore storage with tidal plants and offshore wind farms appear attractive.

## 3.4. Total investment for offshore dams

The total investment for tidal plants may be in the range of 500 billions U.S. $ for 300 GW supplying 1000 TWh/year.
The total investment for offshore reservoirs may be 1000 billions $ for 500 GW.
The total investment for shore protection may be very high.
The future overall investment may thus be 50 billion $/year, well over the past dams investment/year. ICOLD may thus be deeply involved through exchange of views, relevant analysis and suggestions.

Fig. 1

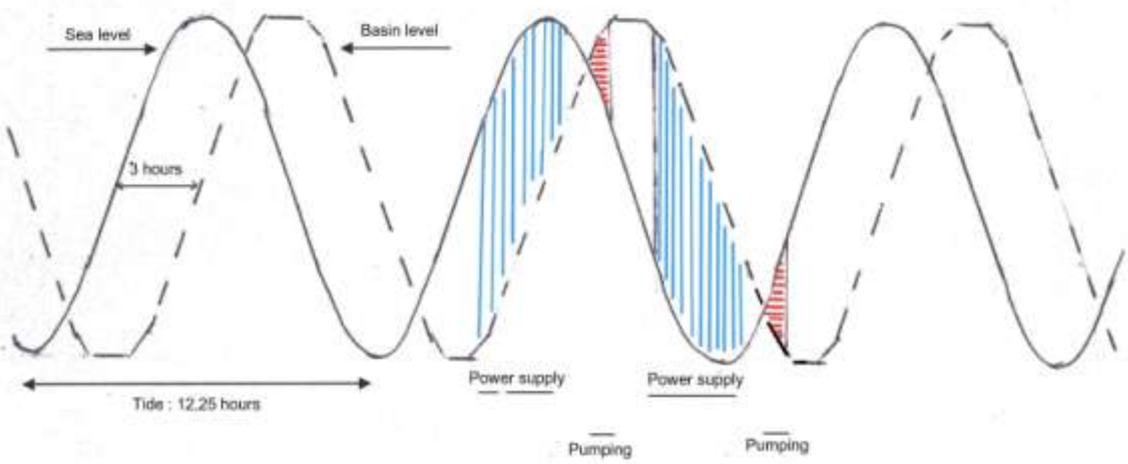



Fig.2

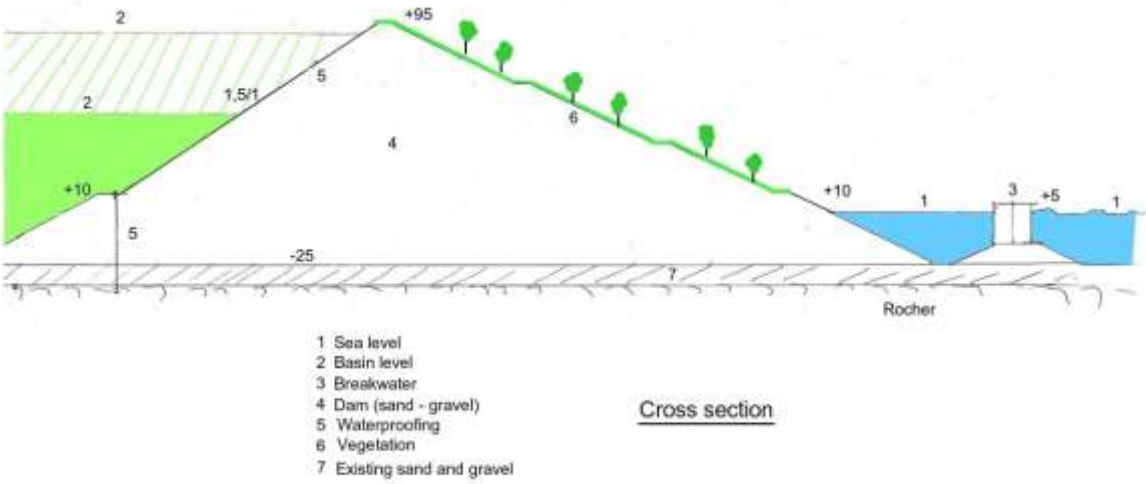
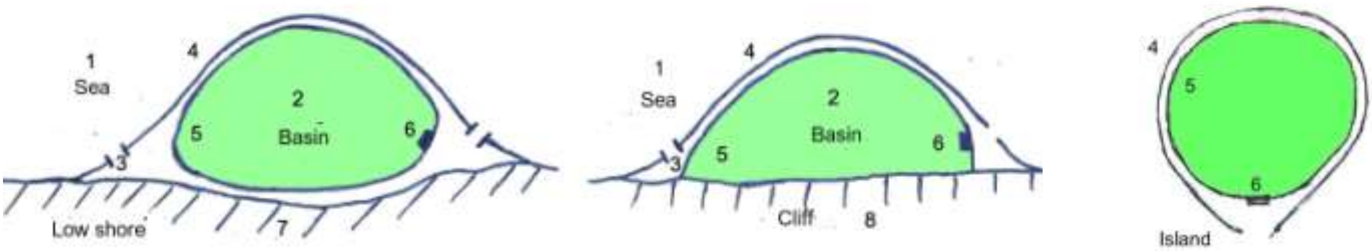



Fig. 3




**Storage lay out**

| | | | |
|---|---|---|---|
| 1- Open sea | 3- Harbour | 5- Dam | 7- Low shore |
| 2- Emerald lake | 4- Breakwater | 6- Power plant | 8- Cliff |

## 4. ONSHORE RESERVOIRS FOR PUMPING STORAGE PLANTS (P.S.P)

### P.S.P. needs along the 21th century

Within 50 years the world production will probably triple and the need of energy utilization will at least double. The acceptable unit cost of energy generation may double and reach 10 cents of $ as average. Hydropower and nuclear will each supply about 5 % of needs as now. Biomass geothermy and miscellaneous will together hardly supply 20 %. For climatic reasons and lack of resources the share of fossil fuels should be under 30 %; Wind or solar power should thus supply 40 or 50 % reaching 40 or 50.000 TWh / year mid century (fig.1). The wind potential at a generation cost under 10 cents of $ per KWh (actualized in 2010) is probably under 20.000 TWh/year but the potential of solar power is far beyond needs. A low cost of solar power will be favoured by technical and industrial progress within 20 years and by the fact that most increase of energy needs is in developing countries with over 3.000 sunny hours per year: and a reduced solar energy during the rainy season is often balanced by much more hydropower at that time.
The key problem is that wind and solar energies which will supply as average over 5.000 GW are intermittent and require storage. As there are some possible adjustments by lakes hydropower and by thermal plants, the storage needed capacity will be under 5.000 GW but probably in the range of 3.000 GW with a storage of 1 or 2 days i.e. 100 TWh (thermal plants should be used for long periods without wind or sun or for part of peaks).
The best storage of intermittent energy will most probably be by Pumping Storage Plants (P.S.P.). For an average cost of PSP of 1 500 $/KW the investments for 3 000 GW shall be 4.500 Billion U.S. $ for 45.000 TWh/year of intermittent electricity supply i.e. an investment of 10 cents per yearly generated KWh and thus an extra cost for storage of about 1 cent per KWh of wind or sun energy. A loss of 20% applying to the stored part of this intermittent energy (possibly 40 or 50 %) with an average generation cost of 10 cent will add 1 cent. The total extra cost for storage of intermittent energies may thus be limited to 2 cents per KWh and such huge storage will favour the grid management and safety as adjusting in minutes the power supply.

Such acceptable cost for storage of electric energy is the key for a huge utilization of wind and sun energies, the only solution for avoiding mid century a much increased utilization of coal, and the exhaustion of all fossil fuels in 2100: 15 000 TWh/y of wind and 30.000 of solar electricity appear reasonable targets mid century.

The problem is thus simple: Is it possible to create within 50 years 3.000 GW of PSP storing 100 TWh for a unit cost under 2 000 $/KW (as in the past for 150 GW)? The main increase of PSP needs will be after 2030 when this investment may be well over 50 GW per year, much more than the present hydropower yearly investment (see fig.2).

Technical Solutions for P.S.P.:

Most of 300 P.S.P. which total worldwide 130 GW have a unit capacity between 0,2 and 1 GW (few over 1 GW), and operate between 2 reservoirs with a differential head of 100 to 1000 m, as average 3 or 400 m. Most reservoirs are made by a dam closing a river: such dams are not much different from other dams, with however a special care of drawdowns. The total volume of relevant reservoirs is under 20 Billion $m^3$, a small part of the hydropower reservoirs totalling 6 000 Billion $m^3$. A significative part of investment is the cost of tunnels linking the 2 reservoirs of each P.S.P.

As the total need of PSP may be multiplied by over 20, the unit capacity will increase and after 2020 or 2030 most investment shall probably be with unit capacities between 0,5 and 5 GW, for example 1 P.S.P. of 1 to 2 Billion $ for 5 Million people and for 10 to 50.000 $Km^2$.
There are many possible P.S.P. sites in mountainous areas: Japan has already 20 GW and India has identified 100 GW but many countries have no high mountains, much wind energy will be close to sea or offshore and much sun energy will be in desert areas without fresh water. Using the sea as low reservoir appears thus an attractive alternative, already tested in few places, for instance in Okinawa (Japan) (fig.6)

Sea water P.S.P.

The lowest technical cost for high P.S.P. reservoirs could be reached by damming rivers close to the sea. This will be usualy unacceptable in populated areas for resettlement or environmental reasons but may be possible in some rather desert sites: In this case the problems of dam design and construction are quite similar to traditional ones and are not analysed here.

There are more opportunities of creating sea water large reservoirs beyond rivers. Cliff or mountain areas close to sea may be the most favourable but it may be possible also to implement cost effective very large high reservoirs for P.S.P. on low shore (except in very poor soil conditions). The design, construction, methods and schedule of works of the dam closing such large reservoir may be very different from those used for traditional dams and most relevant set ideas and traditional solutions should be reviewed in depth:

- The huge impacts of the rivers on dams design construction, schedule and costs are avoided.
- The spillways are avoided or limited to the pumping capacity.
- The dam may be waterproofed by geomembranes which may be checked, repaired and possibly replaced if necessary
- The need of foundation treatment or imperviousness may be usualy reduced. Some settlement is acceptable, reducing the need of excavation under the dam.
- First filling may be progressive and well controlled. Drawdown is easy in short time.
- The schedule of works is much easier than for traditional dams and the utilization of construction equipment much better. Accesses are easier than along rivers; for huge earthmoving quantities, low cost may be obtained by heavy mining equipment used 15 or 20.000 hours during a five years construction schedule: (huge electric excavators, 100 T trucks, buckets wheels, conveyor belts…)

The design of the dams may thus be often based upon the following:

- Imperviousness of dam by geomembranes.
- Limited excavation or treatment of foundation.
- Drainage of dam body.
- Design favouring low unit costs of earthmoving.

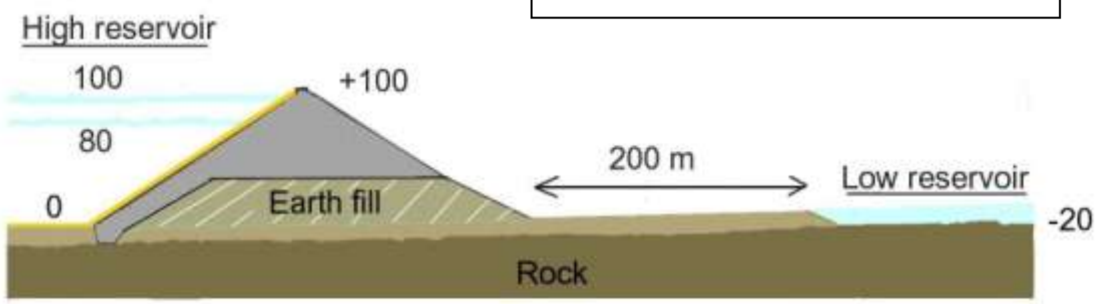
Using the sea as low reservoir is promising. There is some extracost for the materials of turbines and possibly for the waterproofing of an high onshore reservoir but the length of tunnels is reduced and possibly nil. Some countries such as Norway have very cost effective sites with high heads and many countries have cliffs 100 m over the sea; the north of France has dozens of possible sites for onshore basins of few $km^2$ 100 m over the sea level; for high basins upon a cliff the cost of the dykes closing the high basin is rather low and it is possible to have cost effective P.S.P. of moderate capacity for a cost similar to the cost of a traditional P.S.P.

A reservoir of 2 km$^2$ operated under 20 m range and 100 m over the sea level is storing

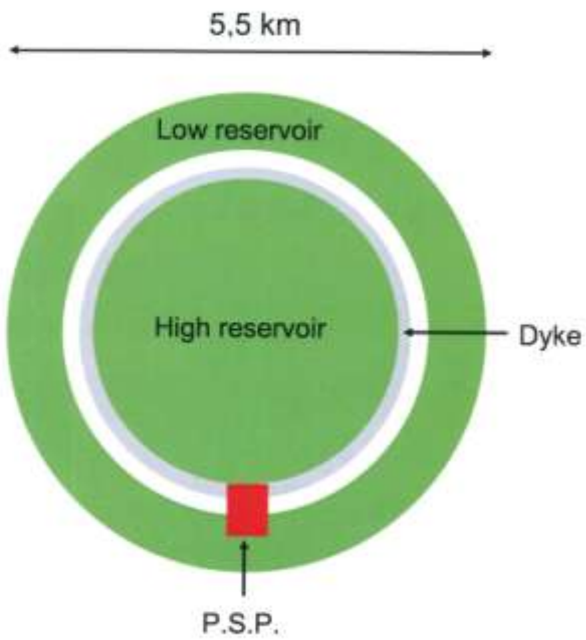
$\frac{0{,}9 \times 2 \times 10^6 \times 20 \times 100 \times g}{3{,}600}$ # 10 million Kwh i.e. 0,5 Gw along 20 hours.

- An other solution is the implementation of P.S.P. in flat large desertic areas. The cost of the dykes of the high reservoir will be high and unacceptable for rather small capacities. Multiplying by 2 the reservoir diameter doubles the length and cost of dykes but multiplies by 4 the area and storage. This solution may thus be cost effective for large capacities as per the example below.

**Cross section of reservoirs**




**Lay out**




A typical P.S.P. on a flat area could be for 3 Gw stored along 15 hours i.e. 45 Gwh. The high reservoir will operate between 80 m and 100 m over the natural ground level. The low basin of same area will be excavated 20 m under the natural ground and corresponding excavated materials used for the 100 m high dykes of the high reservoir. The head on pumps- turbines will thus vary between 100 + 20 = 120 m and 80 m and will be as average 100 m.

For 45 Gwh, the necessary area of each reservoir will be (in km$^2$) $\frac{45 \times 10^6 \times 3600}{0,9 \times g \times 10^6 \times 100 \times 20}$ # 9km$^2$
For a circular high reservoir, the diameter will be 3,4 km and the dyke length 10,5 km.
As a rockfill dam, the dyke will require 15 000 m$^3$ / m i.e. about 160 Million m$^3$ with overall unit cost of 20 U.S $ i.e. 3,2 Billion $. As an earthfill dam it would require 220 Million m$^3$ x 15 $, i.e. about the same cost, close to 1 Billion US$ per Gw.
A dam associating rockfill and soft materials may be the most cost effective solution. The low reservoir may be along the dyke but few hundred m apart i.e. an annular basin of 15 km long, 600 m wide and 20 m deep.

The total area occupied will be 25 km$^2$, under 10 km$^2$ per Gw. The dyke volume will be 50 to 75 Million m$^3$/Gw. The cost for a smaller scheme close to 1 Gw would be about 1,5 Billion per Gw instead of 1. For such heads the cost of the plant itself including its civil engineering may be in the range of 1 Billion $/Gw. The total cost should then be 2 or 2,5 Billion/Gw close to traditional P.S.P. costs.

Such solutions of high dykes for the high basin may apply to P.S.P. using the sea water: the high basin may be on a flat shore, or at sea in shallow water and preferably along a cliff. If using fresh water, the P.S.P. should be close to an important river for getting initially enough water.

The cost of P.S.P. along low shore levels will be acceptable only for very large capacities.
The cost will be much reduced in cliffs areas over 50 m high where smaller P.S.P. capacities may be cost effective.
In mountains areas rather close to sea, it may be justified to accept long tunnels between the sea and a much higher reservoir. Cost may be attractive even for small capacity.
Costs above should evidently be adjusted to local conditions.

The key point is that P.S.P. using sea water may be cost effective in many places.

### Fresh Water Large P.SP:

- The above design of large high reservoirs associated with the sea used as low reservoir may be used for reservoirs associated with existing or new reservoirs built along rivers and used as low reservoirs. The storage capacity of many existing large hydropower reservoirs is used once per year; it could be much more cost effective mid century to use part of it 50 or 100 times per year as low reservoir of a P.S.P. This will not prevent electricity generation by the existing hydropower plant but may reduce it. The design and data of high reservoirs may be similar to these of basins along shore with less care of possible leakage.

Some hundred existing or new dams could be used accordingly with a total P.S.P. capacity up to 500 GW.
The two reservoirs of a P.S.P. may also be totally artificial beyond rivers and this solution cost effective for large capacities.

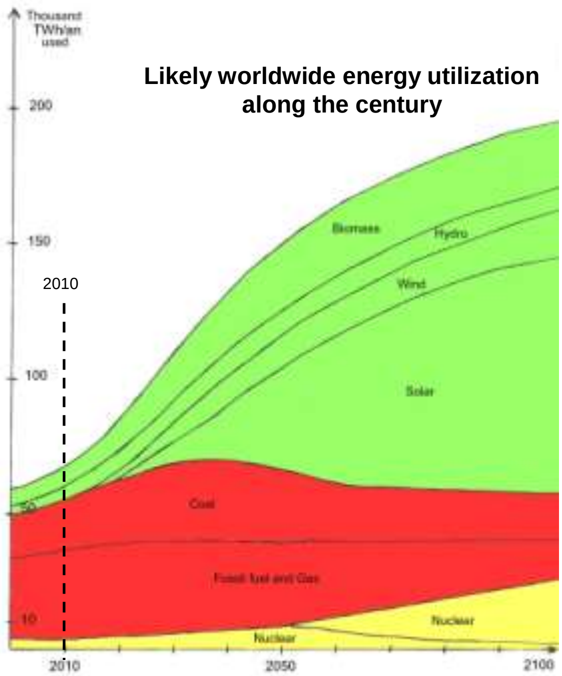
### Overall future P.S.P:

3 to 5 000 GW of P.S.P will require some thousand plants. Possibly half will be similar to present ones with 2 reservoirs on rivers but their number may be limited by environmental, physical or cost reasons. There are many possibilities for high artificial reservoirs close to sea,

perhaps for one third of needs. And associating artificial high reservoirs with large hydropower reservoirs may apply to some hundred P.S.P.
The total volume of earthfill may be 100 Billion $m^3$ for reservoirs of 3 or 4 000 GW of P.S.P. The total area to be used may be in the range of 20 000 $km^2$. The environmental impact will be much lower than for existing or future traditional dams. Resettlement will be very reduced. And associating the sea with an artificial offshore high or low reservoir may be a solution analysed in another chapter.

## Conclusion

Half of future world energy will probably be supplied from wind and sun, thus requiring after mid century a storage capacity of about 3 000 GW and 100 TWh. This storage may cost 2 cents of $ per KWh of intermittent energy and will require some thousands Pumping Station Plants. Half of them may use the sea or a large hydropower lake as low reservoir but their high reservoir may be out of rivers and require much low cost earth or rockfill volumes. The design of relevant dams may be much simpler than for traditional dams and environmental impacts much lower.
The total annual investment for P.S.P. after 2030 may be more important than the overall investment for all other dams.



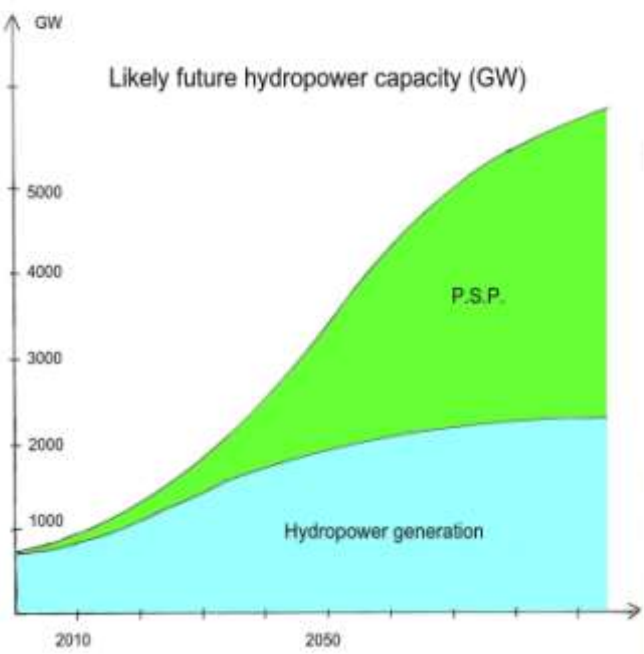





Fig.3


Fig. 4


Fig. 5

Fig. 6

<u>Okinawa (Japan) Pumping Station Plant</u>

# 5. DAMS WITH SIGNIFICANT SILTATION PROBLEMS

The siltation is a rather minor problem for many dams but may reduce by decades the possible long life of 50 % of them and may be a key problem within few years or few decades for over 10% of large or small dams.
The cost efficiency of various solutions for siltation mitigation has varied greatly and it is not easy to optimize their choice because the local data are never the same and the likely siltation rate itself may hardly be known precisely before some years of dam operation.

The key problems are:
- The loss of storage, especially for irrigation or drinkable water
- The damages to turbines of hydropower plants
- The impacts to the river, especialy downstream of the dam.

## 5.1. Present conditions

The total reservoirs storage is 7000 $Km^3$ for 50 000 large dams including very roughly:
- 5000 $km^3$ for 10 000 hydropower dams
- 500 $km^3$ for 30 000 irrigation or water storage dams.
- 1500 $km^3$ for dams devoted to several targets including hydropower.

Siltation impacts and relevant solutions vary greatly with the reservoir target and volume and with the river data.
The annual rivers inflow is 40 000 $km^3$. The annual sediment storage may presently reach 40 Billions $m^3$, 0,6% of the reservoirs storage; the total present sediment storage in reservoirs is close to 1 500 $km^3$ but a large part is in the designed dead storage of hydropower dams.

The overall cost of world reservoirs siltation has been evaluated between 15 and 20 Billion U.S. $ per year, i.e. 30% of the overall annual expenses of 60 Billion for dams or 10% of the 200 Billion $ annual dam benefits. But these ratios are much higher for 10 or 20 % of dams which are most prone to siltation; the siltation problem should impact their overall design including the choice of the dam site, the reservoir volume and all spilling facilities. Care of siltation may be a large part of their investment but it may be possible to postpone by decades a part of this specific investment (by pass tunnels, dredging facilities, dam upgrading) and to adjust its extent and timing to the precise actual rate of reservoir siltation. Huge expenses for siltation mitigation are much more acceptable when the basic dam and plants investments are already paid for.
Some efficient solutions such as sluicing or flushing are well known from a long experience worldwide: some more recent solutions (such as sediment bypass tunnels, specific dredging facilities or managed sediment storage) are promising especially for very large schemes and are analyzed below.

## 5.2. Variability in sediments:

- 90% of the world catchment areas generate less than 500 tons of sediment per year and per $km^2$ but 10% generate 5000 tons/$km^2$ as average and some supply well over 10 000 tons. The annual sediment load reaching many small reservoirs is thousands $m^3$ but many large hydropower reservoirs are reached by million or dozen Million $m^3$/year and the load of 10 rivers is over 150 Million $m^3$/year.

- The concentration varies greatly along the year; 50% of the annual load may be discharged in few days and 90% in one month. Most siltation may be at the beginning of the flood season. The total annual load may also vary greatly.

Measurement of sediment load is thus a difficult problem but it is a key target as well at design time as during operation; many usual methods and procedures are unefficient and were the reason of unadapted designs.

## 5.3. Main impacts of reservoirs sedimentation

### 5.3.1. Storage loss

It is usualy the main impact for dams devoted to water storage as their benefit is quite proportional to the storage. This impact is lower for dams devoted to hydropower: their benefit may possibly be reduced by under 20% when the reservoir is 80% filled (including a large part in the designed dead storage).

### 5.3.2. Turbines abrasion:

Sediment coarser than 0,1 mm may greatly accelerate the erosion of turbines parts; even smaller grain sizes may cause damages if containing quartz. It may be the main siltation problem for high head hydropower. Also sediment concentration and total head are important factors.

### 5.3.3. Downstream impacts

River reaches downstream of dams suffer large environmental impacts due to flow changes, reduction of sediment load, altered nutrient dynamics, temperature changes, and the presence of the migration barrier imposed by the structure and the upstream impoundment.

Clear water released from the reservoir will cause down stream erosion and possibly bank failures.

Sediment trapping by dams can even affect coastal morphology. It sometimes becomes a major factor contributing to the rapid shoreline recession and subsidence (Nile, Mississipi…).

One way of reducing this impact may be to build run-of-river hydroelectric projects which would allow passage of 100% of the fines and an important portion of the bed load.

### 5.3.4. Concepts Of Reservoir Life

With reasonable levels of maintenance, the structural life of dams is virtually unlimited, and most reservoirs are designed and operated on the concept of a finite life which will ultimately be terminated by sediment accumulation rather than structural obsolescence.

Design life is the planning period used for designing the reservoir project. Planning and economic studies are typically based on a period not exceeding 50 years, whereas engineering studies often incorporate a 100-year sediment storage pool in the design.

The target of a very long reservoir life should be a key point of a right design and management of siltation problems.

## 5.4. Reservoir sedimentation management

### 5.4.1. Sediment trapping upstream of the dam

There are only two strategies to reduce the sediment yield entering a reservoir: either prevent erosion or trap eroded sediment before it reaches the reservoir.

The rehabilitation of some watersheds can dramatically reduce the rate at which sediment, nutrients, and other contaminants are delivered to a reservoir.

- A number of reservoirs in Russia are undergoing sedimentation problems. An efficient proposed option was the construction of upstream reservoirs that will act only as sediment retention structures. (Aliev et Al 2009,ICOLD, Q.89)

5.4.2. Sediment Routing

Sediment routing includes any method to manipulate reservoir hydraulics, geometry, or both, to pass sediment through or around reservoir or intake areas while minimizing objectionable deposition.
Routing is the most environmentally benign sediment management strategy.
Sediment routing techniques include sediment passing through and sediment by pass.

*5.4.2.1. Sediment pass through*

It may be by seasonal drawdown, by drawdown adapted to floods or by turbidity currents. This requires initial implementation of the necessary bottom gates to be designed with great care.
- A reservoir operated under seasonal drawdown is either partially or completely emptied during the beginning of flood season. Seasonal drawdown is conducted during a predetermined period each year, as opposed to flood routing, which requires that the reservoir level be drawn down for individual flood events when they occur. At some sites routing can be implemented at very low cost
A major disadvantage of sediment routing is that a significant amount of water must be released during floods to transport sediments. Sediment routing is most applicable at hydrologically small reservoirs where the water discharged by large sediment-transporting floods exceeds the reservoir capacity, making water available for sediment release without infringing on beneficial uses.

From among the case studies, Sanmenxia, Heisonglin, Cachi, Gebidem, and Sefid-Rud all employ reservoir emptying for sediment management. Sanmenxia is an example of sediment routing because the reservoir is empty during the flood seasons, with the primary emphasis placed on passing sediment through the impoundment without deposition. This was successfully achieved before 1960 with the Old Aswan Dam storing 6 billion $m^3$ of Nile water.

- Density of the sediment laden water at the time of the flood may be high and the sediment laden water may sneak and drift along the bottom of the reservoir. Turbidity currents may then be vented through low-level outlets, reducing the sediment accumulation within the impoundment without drawing down the pool level. Jolanda M.I Jenzer Althaus et al. provide the concept of turbidity currents based on case studies of several Alpine reservoirs in Switzerland. Analysis of this intervention showed that the reservoir could last 20 to 50 years longer. (2009, ICOLD, Q.89)

*5.4.2.2. Sediment Bypass For Reservoirs*

When topographic conditions are favourable (for this reason also the site selection is important), a large-capacity channel or tunnel can be constructed to bypass sediment-laden flow around the reservoir or part of it. It may be built initially and possibly used for flood control during construction. Most may also be built according to precise needs after years or decades of operation. Such tunnel may be associated with a low dam in the upstream part of the reservoir. It may bypass some hundred $m^3/s$ or even thousand $m^3/s$ of water. <u>This solution may have much future for large schemes</u>.

- Mitsuzumi, Kato and Omoto provide the results of the monitoring activities that had been conducted on the sediment bypass system at the Asahi Dam in Japan. The 2350 m long bypass system has a maximum discharge capacity of 140m3/s. It can flush out both bed load and/or turbid water. The by pass system managed to control 90% of the sediment. (ICOLD, Q.89)

5.4.3. <u>Sediment flushing</u>

Hydraulic flushing involves reservoir drawdown by opening a low-level outlet to temporarily establish riverine flow along the impounded reach, eroding a channel through the deposits and flushing the eroded sediment through the outlet.
Unlike sediment routing, which attempts to prevent deposition during flood events, flushing uses drawdown and emptying to scour and release sediment after it has been deposited.
Usually flushing is less efficient if the sediment is coarse or is consolidated clay materials. The width of a flushing channel within sediments is often in the range of 100 m and may reach few hundred m; flushing is thus much more efficient in narrow valleys.

- Guo and Cao (China) provide the experience of Sanmenxia Reservoir showing that the operation mode has an important effect on reservoir sedimentation. The reconstruction was done to increase the flood discharge capacity of the dam so that as much as possible of all flood sediment laden water is flushed out, which makes the reservoir in an equilibrium state. Guo and Cao futher provide that having cascade dams upstream greatly reduces the sedimentation in the downstream reservoirs and in the long run prolongs the life span of the downstream reservoirs. (2009,ICOLD, Q.89)

- Sumi et al. (Japan) present their finding on the effect of sediment flushing and environmental mitigation measures in the Kurobe River for two major dams namely Unazuki and Dashidaira. The sediment flushing and sluicing operations for two dams are conducted in a coordinated manner. It was concluded that drawing sediment while operating at low water level significantly improves the flushing efficiency. Evacuation channels have been used successfully in the evacuation of fish and other creatures whenever turbidity levels are high due to flushing.

5.4.4. <u>Managing the silt storage</u>

- It is usually difficult to drawdown completely the large hydropower reservoirs but it is usually possible to drawdown the reservoir by 20 % or 30 % at the flood time in order to store the sediment in the dead storage and to pass through most sediment when the dead storage is full. Corresponding gates may be 20 to 50 m under the dam crest.

- Turbines abrasion may be a key problem for high head hydropower and many desilting underground chambers along head race tunnels designed for avoiding it proofed costly and poorly efficient. It may be possible to design the reservoir itself as a desilting structure upstream of the entrance of the head race to the power house. This solution which is presented in the ICOLD Bulletin 144 appears very promising. The stored sediment may be flushed from time to time or dredged permanently.

5.4.5. Removing stored sediment:

Sediment deposits may be mechanically removed from reservoirs by dry excavation or hydraulic dredging and hydro-suction.

The annual worldwide stored sediment is close to 40 Billion $m^3$; possibly half is in the designed dead storage. Harmfull sediment is thus in the range of 20 Billion $m^3$ for a total relevant damage close to 20 Billion \$, i.e as average 1\$/$m^3$ but much higher or lower according to dams. This means that for many dams, removing materials at a cost of one or few \$/$m^3$ may be cost effective.

*5.4.5.1. Dry Excavation*

All methods of mechanical excavation are costly because of the large volumes of material involved and, frequently, the difficulty of obtaining suitable sites for placement of the excavated material within an economic distance of the impoundment. However, once sediments are deposited in a reservoir, excavation may be the best management option available.

- Sumi et al. discuss a concept of asset management but specifically on its applicability to reservoir sediment management at Kizu River in Japan, using a number of case studies. It was concluded that dry excavation while the reservoir is empty may be the most economically feasible countermeasure even if reduced power production and water production is taken into consideration. (2009, ICOLD, Q.89)

*5.4.5.2. Hydraulic dredging*

The solutions may be very different for small irrigation dams and for large hydropower reservoirs. They have been presented in ICOLD Bulletin 144 (chapter 3.4.8.)

- For rather small irrigation reservoirs siphon dredges may be used and do not need pumps. The slurry is forced through the pipeline by the differential head between the water surface in the reservoir and the discharge point located at the lowest possible place downstream of the dam. It seems adapted to some dozen thousands $m^3$ of silt per year.

- Many large hydropower reservoirs are designed with a huge dead storage witch may be filled in some years or decades: flushing may be inefficient or unacceptable but the cost of dredging millions m3/year may be acceptable if the dredging equipment is designed according to local facilities including fine materials, calm water, electric energy, and paid for along decades of operations. This is studied in ICOLD Bulletin 144. For a scheme of 1 Billion \$, investing 20 to 50 Millions \$ of dredging equipment after 10 or 20 years of operation may be cost effective.

- Dredged sediments may be sent downstream to the river. They may also be sent to a disposal area and the water flown back to the reservoir (Mechra reservoir in Morocco: ICOLD Q.89).

5.4.6. Phased dam construction:

It may be difficult to avoid the reservoir siltation but many earth fill irrigation dams may be designed for an easy level increase after 20 or 100 years of operation. Raising by 5 m a 30 m high dam may quite double the storage when necessary.

## 5.5. Asset management for reservoir sediment management

Sumi et al. (Japan) provide a study on the applicability of the asset management for reservoir sediment management. (2009, ICOLD, Q.89)

5.5.1. Effects and costs of countermeasures

In order to sustain function of dams for more than a century, we must clarify challenges to apply asset management to dams, which are made up of a variety of elements such as dam body, spillways mechanical and electric equipments with different service lifetimes and functions to lower their life cycle costs (Lcc) and to smooth their annual maintenance costs.
In the near future, the sedimentation countermeasure cost may exceed for many dams the cost of electrical and machinery equipment categories. Therefore, it is important to apply asset management not only to the electrical and the machinery equipment categories, but also to sedimentation countermeasures.
The effects and cost of each sedimentation countermeasure are presented on Table 3 as an example.

| | | | |
|---|---|---|---|
| | | 75 millions yen/year | Compensation for reduction of power generation, water supply loss |
| Countermeasure | Cost | | Remarks |
| | Investment to build the facility (initial cost) | Running cost | |
| Excavation | - | 4,000 yean/$m^3$ | Mechanically discharging |
| Dredging | - | 35,000 yen/$m^3$ | Mechanically discharging |
| Sediment check dam + excavation | 5.4 billion yen/dam (sediment check dam) | 4,000 yen/$m^3$ (excavation) | Cost of building a check dam + mechanically discharging |
| Flushing | 10,1 billion yen/channel | 22 million yen/year | Sediment flushing gate installation |
| Sediment bypass | 13.163 million /channel | 121 million yen/year | Construction bypass + prevention of tunnel invert |
| Dry excavation with the reservoir | - | 2,500 yen/m3 | Excavation |

Table 3 Sedimentation Countermeasures Cost
assumed by the sedimentation measure case in Japan

5.5.2. Study of optimization of sediment removal method

- For this example, it has been shown that the dry excavation with the reservoir emptying may be economically feasible even if it needs compensation both for reduced power production and water use loss rather than other countermeasures requiring initial facility investment such as sediment bypass, flushing sediment etc.
- The above result suggests the possibility of “the optimization of sediment removal method” if associating the management of several dams.

## 5.6. Conclusion

**Siltation may be a key problem of many reservoirs**

The design of such dams including the site choice and layout should be linked with it.
It is necessary to investigate following items for sediment treatment in reservoir:
- To measure efficiently the sediment load
- To evaluate the cost of sediment in long term at the time of planning dam structure
- To select most suitable sediment treatment method with consideration of topography and flows of river, effectiveness, economical, environmental and various conditions with overall judgment.
- To apply, not only one, but several measures in rivers with great volume of sediment, such as sediment routing, bypassing, dredging, or measures to origin of occurrence upstream of dam.
- To apply measures for sediment with coordination of several reservoirs of one area or basin.
- In the near future, strategic dam asset management including preventive countermeasures for reservoir sedimentation will be an important challenge.

The sediment in dams is critical. Leaving sediment management as it is may lead to not only reduction but full loss of reservoir functions. With proper treatment of sediment, it is possible to maintain its function economically along centuries.
Since the progress of sedimentation is slow in general, part of solutions may possibly be delayed but the long term possible solutions should be analysed initially and partly implemented from the beginning: they may thus impact the full design.

REFERENCES

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[2] Gregory L.Morris and Jiahua Fan: Reservoir Sedimentation Handbook : Design and Management of Dams , Reservoirs , and Watersheds for Sustainable Use, 1998
[3] Tetsuya SUMI, Kiyoshi KOBAYASHI, Kenichiro YAMAGUCHI, Yasufumi TAKATA: STUDY ON THE APPLICABILITY OF THE ASSET MANAGEMENT FOR RESERVOIR SEDIMENT MANAGEMENT, Japan q89-r.4, 2009
[4] Qingchao GUO, Wenhong CAO: RESERVOIR SEDIMENTATION AND ITS CONTROL、IWHR China Q89-R.4, 2009
[5] ICOLD Bulletin 144 : Costs Savings in Dams (2010)

# 6. VERY LOW EARTHFILL DAMS

## 6.1. Overview of very low earth dams

There are worldwide over 100 000 dams lower than 10 m with storage capacity between 0.1 and 10 $hm^3$ and sometimes up to 100 $hm^3$. Generally they are combined earth and concrete dams. The spillway portion is generally a small concrete gravity free overflow dam located close to the river bed or small lateral weir located on the abutments when they are flat. The catchment's areas are around few $km^2$ up to 100 or 200 $km^2$. The reservoirs are generally seasonal ones.

These dams are essentially used for small scale irrigation, drinkable water, live stock breading and domestic needs. They are very important for riparian villages and people specifically in arid and semi arid zones of Asia and Africa and in some regions like the Nord East of Brazil. In Burkina Faso for example, around one thousand of such dams have been built since the 1920's and more than 5 000 villages out of 8 000 and around 8 millions inhabitants out of 14 millions depend on these small reservoirs for water supply during the long dry season...

## 6.2 Peculiarities and main threats of Very low earth fill dams

Due to their small size, these dams are designed, built and operated in difficult technical environment with the lack of qualified professionals for their operation and maintenance. However the earth works are generally done now with heavy engines and may result in well compacted earth dam. But sometimes, the qualities of the embankments are weak and prone to piping failure after the first impoundment of the reservoir due to lack of water for compaction and use of very fine materials.

Due to the lack of data on small rivers flows, the evaluation of design flood for the design of the spillway is sometimes a difficult challenge.

Thus, the main causes of threat for these types of dams are:

- Submersion and destruction of the embankment by floods due to insufficient spillway capacity; in regions like the sahelian zone of Africa this threat represent 60% of the case of incident and accident of dams and this is increasing these last years with the multiplication of extreme floods possibly due to climate change
- Internal erosion within the embankment or the foundations or between the spillway and the embankment.

One important issue for these small reservoirs is sometimes to be able to retain their storage capacity for the following reasons:

- Huge losses due to evaporation, as many of these dams are located in arid and semi-arid zones with long insulation, high temperature and important dry wind with significant velocity. Generally these reservoirs have a low mean depth of 2 m and rarely up to 5 m.
- Loss linked to sedimentation by fines transported with the early seasonal flows on a denuded watershed due to desertification and human activities such as bad cropping techniques and clearing of wood and forest in the upland.

- Insufficient watertightness of the reservoir due to poor geological studies and sometime only relying on the too thin clay deposit in the future reservoir.

These data are related to Burkina Faso which is a typical example but some conditions may be different in other regions and countries.

According to record on small dam failures, their impact results rarely in fatalities. For around more than one hundred failures recorded for example in Burkina Faso in this last decade for such dams, only few cases have led to loss of lives. Generally, the failures provoke one or more breaches in the embankment up to 50m width and its foundations which can be repaired during the next season when means are available, and some damage on the irrigated perimeter or installations close to the river bed downstream.

The dam break floods are generally in the magnitude or less than the design floods, when the dam is less than 4 to 5 m high. The worst case is when the central free overflow concrete gravity section of the dam failed. This issue need to be considered, as generally these concrete gravity dams are sensitive to uplift which is generally high downstream of the structure during floods due to the low slope of the river downstream.

### 6.3. Area for cost saving in very low eartfill dams

Regarding cost optimization for these small dams, one can consider the following aspects:

#### 6.3.1. Spillway and protection against floods

The cost of the spillway section and protection against floods represents generally an important part of the overall cost. According to data in Burkina Faso and West Africa, the direct cost of the spillway section represents around 20 to 30% of the total cost of the dam when the spillway is located in the central part of the dam or close to the river bed. When taking into consideration the indirect cost of loss of storage capacity due to the necessity to have an important head on the free overflow spillway the total impact of the spillway design is far more important on the overall cost. The cost savings on the spillway design and arrangements can be significant;

#### 6.3.2. Foundations

The second costly part of the dam is the foundations treatment. One should be careful for these aspects as small dam can be funded on complex geological and geotechnical context without detailed and comprehensive investigation means as the dam is very small. In Burkina Faso, data recorded shows that foundations treatment is the most expensive cost in small dam construction with around 30 % to 50% of the total direct cost.

#### 6.3.3. Embankment

The construction of the embankments is now done with heavy mechanical engines and less by hand labour which results in better embankments quality. During the last seasons in Burkina Faso, many small earth dams have withstood floods without failure.

#### 6.3.4. Operation and maintenance

For these very low earths fill dams:

- Local people responsible for operation and maintenance of these small dams lack of skills and means, and even sometimes small companies operating these dams suffer of the same problems. Design solutions have to consider this environment.
- The risks associated with these dams are low due to their size and possible location in remote and non dense occupation zone.
- The need to ensure the availability of the mobilised water resources is of vital importance for local communities close to the dam. It may increase with the future climate.

## 6.4 .<u>Recommendations for the design, construction and operation</u>

Design criteria and solutions may thus be very different from those prevailing for higher dams.

### *6.4.1. Spillway design and protection against floods*

**a) Design and check floods**

The concept of design and check floods is suitable for this kind of dam as the risks are low. The design flood can be defined as the 10-year or 50-year flood with a freeboard of 0.5 to 1 m for the dam crest and the check floods can be the 500-year to 1 000-year floods.

Considering the low risks associated with these kinds of dam, it is possible to undertake a risk optimization analysis to define precisely the level of floods and the design flood for the spillway.

**b) Low cost solutions for spillways**

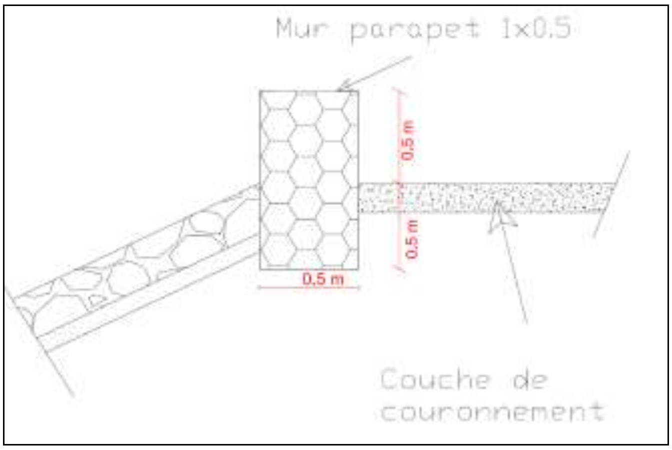
There are today many solutions to reduce the cost of spillways system for these very low earth fill dams. These solutions can be used as single one or combined to achieve the better protection of the dam against floods at a least cost. Some classical and new solutions are presented in the following:

**- Overflow embankment zone protected with concrete or masonry**

For the same reason, it can be envisaged for these dams with low head and flood discharge, low cost solutions such as spillway placed on the dam, RCC downstream face, masonry faced on the downstream slope, lateral spillway with stepped gabions, etc.

**- Use of parapet wall**

Parapet wall constructed with gabions or stone masonry can help to protect the dam against submersion by floods higher than the check flood and wave run up.

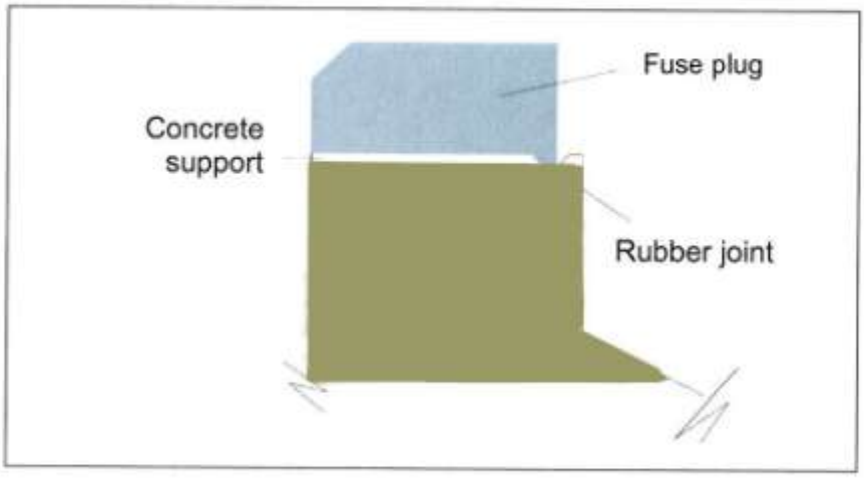



**- Lowering the dam crest at the abutment**

Dam crest can be lowered near the abutment so failure, if it happens, will result in low floods and consequences on the dam and downstream zones. This lowered part of the dam, which is functioning as a fuse dyke, can be protected with concrete or soils cement and other kind of coating to reduce the erodability of the embankment materials under low head (10 to 20 cm). These parts of the dam are generally very low (few metres to less than a metre high).

**- Fuse blocks**

Fuse blocks can also be used to increase the reservoirs capacity and spilling capacity of the spillway associated with a gabions or masonry parapet wall on the embankment dam crest. Fuse blocks are simple concrete blocks installed on the existing spillway crest. They can be precast one or cast in place. Two samples of this new technology are installed today: one in Vietnam on the Saloun dam and one on the Wedbila spillway in Burkina Faso (ICOLD Bulletin n° 144 – appendix 3).



**- PK weirs**

PK weirs are also a new development in the spillway technology (ICOLD Bulletin n° 144 – appendix 2)

6.4.2. Foundations treatment

Foundations treatment is not specifically the area for cost reduction due to the complexity of foundation problems and the fact that the difficulties of foundations are not related to the size of the dam, but to the site conditions. Despite this important aspect, it is still possible for very small dams to reduce cost by choosing adapted and simple methods of foundations treatment as positive cut-off trench, upstream blanket and to scarify and compact the soils in place to improve homogeneity and imperviousness associated with drainage and granular filters system (drainage trenches, pressure relief well, downstream filter blankets …) to avoid internal erosion in the foundations.

6.4.3. Embankment

Generally the embankment section is homogenous with filter blanket for the smallest one and a downstream filter and drain blanket associated to a chimney drain for dam close to ten meters. Due to over consolidation by compacting effort there is not significant deformation or pore pressure within the embankment. Many attentions are focused on the slope stability of very low embankment dams but, in the record of small dam failures, the case of slope sliding is very rare.

The only important problem is to avoid internal erosion within the embankment by choosing non dispersive materials, and when only dispersive materials are available close to the dam site, to provide adapted specification for compaction and water content and to provide adapted granular filter. Experience has shown that many small embankments failed due to lack of suitable compaction and water content of the embankment and development of cracks during construction when the materials are plastic.

For these Embankment dams, the saving in unit cost may be more important than savings in quantities. The construction generally takes one dry or low water regime period which avoid river diversion and protection of works against floods. In arid and semi arid areas water for the embankments construction are sometimes a critical issue;
It is to note that for embankment dam, the intake system includes sometimes a pipe passing through the embankment. This implementation is often the cause of failure by seepage and/or piping along the conduit. If it is recommended, for medium and high dams, to adopt some precautions such as: upstream gate, anti-seepage collars, steel lined pipe, filter along the pipe, etc, it is possible, for very low dams, to avoid these expensive devices since the water head is low and the possible damage very limited.

6.4.4. Sediment siltation management

For these very small reservoir siltation in some area may be very sensitive and increasing the capacity of the bottom outlet works can help reducing the rate of sedimentation by flushing the early flow with high concentration of sediments. To restore the storage capacity of these small reservoirs if this is reduced by siltation it is also easy after few decades to raise these dams.

### 6.4.5. Associated infrastructures for river valley development

These small dams can also be used for river crossing to join villages on both sides of the river. Combining the dam with the road for crossing the river is of utmost importance for cost reduction in river valley development.
It is to note that if a dam is also used as a road, its crest must be:

- very well compacted,
- generally enlarged, compared with a normal crest of a low dam,
- often covered by a bituminous or concrete layer,

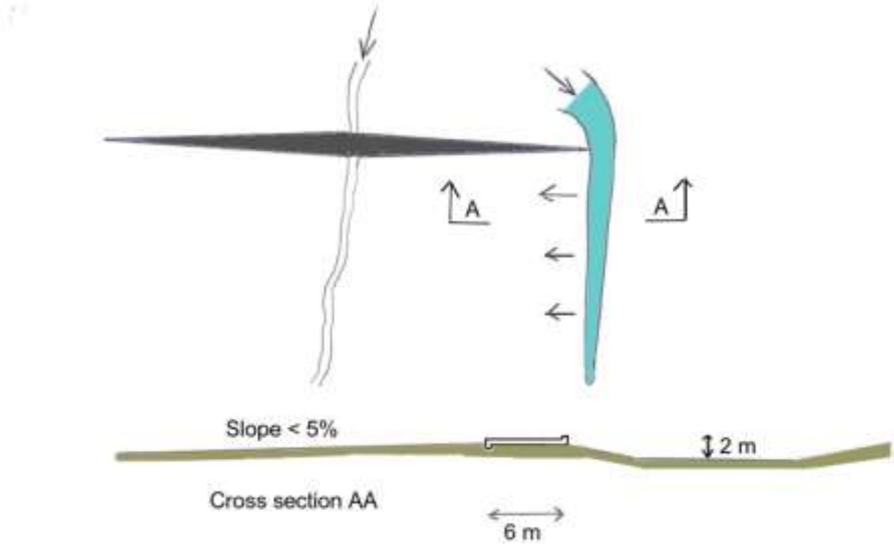
and these factors are advantageous in case of submersion by floods.

### 6.4.6 An innovative solution

This solution may be attractive for dams under about 8 m high, with a rather flat bank (few per cent slope) with soil including some clay for supporting water velocity of about 1 m/s: it does not apply to sandy banks.
It may be used for exceptional discharges up to 100 or 200 $m^3$/s. Such dams include usually a concrete free flow spillway in the river bed with a nappe depth of 1 or 1,5 m. The direct cost of the spillway may be 30% of the investment and the gap of about 2 m between the dyke crest and the spillway sill reduces considerably the storage.
Raising the spillway sill by 0,5 m or 1 m increases significantly the useful storage for a same embankment. This may be obtained by a much longer spillway but a concrete one should be much too expensive. The principle of the proposed alternative is to have a much longer weir parallel to the river, for instance 100 to 300 m long with a nappe depth under 0,5 m. A maximum specific discharge over the sill under 0,5 $m^3$/s/m and a maximum water velocity over the sill and along the bank in the range of 1 m/s, will thus require little or no concrete.

The nappe depth will be nil or few cm during most of flood season and 0,20 m for the ten years flood and 0,4 m for the 1000 years flood.
The access of water to the spillway will be by a channel, about 2 m deep, excavated in the bank. Concrete sill at the entrance of the channel could limit the channel erosion in case of the spillway failure.
As an example for a dam 8 m high and a spillway discharge of 100 $m^3$/s, a traditional solution may be a 25 m long concrete spillway with a sill 2 m under the dam crest and a maximum reservoir depth of 6 m. An alternative solution, as per drawing attached, may be a 200 m long spillway parallel to the river, with a sill 1 m under the dam crest, i.e. with a reservoir up to 7 m deep and a storage volume increased by 50% for a same embankment.
The sill of the spillway may be 6 m wide, lined with 0,20 of masonry or concrete and used as road. A small wall few dozens of cm high between the weir sill and the channel may be precisely levelled.
The bottom gate section used in the river may be slightly increased for discharging usual flows (up to 5 $m^3$/s). The road will thus be dry over 95% of the time and under 5 or 10 cm of water few days per year.
A failure for extraordinary floods would be limited with low downstream risk and an easy repair.
The necessary excavation for the channel along the spillway will be about 5000 $m^3$, a small percentage of the dam volume, using the same earthmoving equipment.

The overall direct cost may thus be reduced by 20% for a storage volume increased by 50%. The quite flat area of 5 or 10 ha between the spillway sill and the river may be kept natural or grassed during the rainy season for reducing the risk of erosion during a very large flood.




Lay out 1/10.000

Cross section 1/500

## 7. DAMS TO BE BUILT IN SEVERELY COLD CLIMATIC CONDITIONS

### 7.1. Introduction

Comments below are essentially based upon Canadian data but most comments appear useful for similar climates elsewhere.

Northern Quebec (Canada) is characterized by a cold harsh climate. With respect to dam building in Quebec additional constraints add further to the other challenges comparable to those encountered in regions with more temperate climates.
Work schedules must be optimized in order that work planning may be based on seasons. Executing dam construction work during the winter months may require extraordinary means. Moreover, some of the work can only be carried out at above-freezing temperatures.
This document first summarizes the conditions that may be encountered in cold areas and act directly upon the selection of a dam type, as well as upon dam construction. Only homogeneous embankment dams or dams with till or clay cores are covered by this document. Afterwards, the parameters influencing dam performance in cold regions are set out. Finally, the most important elements to be taken into account when designing and building dams are presented.

### 7.2. On-site conditions

#### 7.2.1. Thermal conditions

Weather conditions have a significant impact on construction work costs and duration. Indeed, the construction season is most often shorter in regions where winters are cold.

In cold regions special attention must be paid to the thermal regime for the foundation and structure. A proper knowledge of this thermal regime, including the depth of frost/thaw penetration, largely determines the performance of the structure with regard to embankment settlement and stability.
Access to some locations can be arduous during the summer months, particularly during the thaw period. Excavating work and placing of initial fills can only be carried out during winter, when heavy equipment is able to travel on the frozen soil surface. Winter roads and ice bridges were largely used for the construction of La Grande (Quebec, Canada)([1]).

7.2.2. Foundation

Where the foundation material consists of rock, very dense till or any other frost-resistant material the effect of frost and thaw may be neglected in most cases, and a conventional design may then be applied([2]).
Where a foundation is made of frost-susceptible materials different designs are to be considered. One of the approaches consists in removing the frost-susceptible materials and replacing them with more appropriate materials. Another less used approach consists in keeping the foundation materials frozen throughout the life of the structure.
In some particular cases it is possible to adapt the structure design to take into account the probable settlements following the thawing of the foundation.

7.2.3. Drainage

During construction work waters must be controlled. Special attention must be paid to excavation walls to protect these walls against erosion, particularly during thaw periods.
Disruption of natural drainage systems following construction work can also have serious consequences during dam operation where the design includes no effective system for draining the large water volumes generated during thaw periods. In the spring water from the thawing of ice and snow combined with the poor natural drainage typical of frozen (impervious) soils can cause the erosion of an important portion of the soils and fills, sometimes resulting in potential stability problems. To provide proper drainage, ditches, culverts and pumps may be used.

7.2.4. Borrow material

With respect to the construction of impervious cores, working the impervious material borrow sources requires great efforts. Working till deposits in winter involves removing all frozen material prior to using such deposits. Removing the frozen material sometimes requires the use of explosives, which results in significant cost increases. However, it is possible to prepare the required quantities by working the deposit before the frost period. The material is then stockpiled and protected against frost. Transporting the impervious materials from a deposit to a construction site during the winter months requires the use of trucks equipped with heating bodies to protect the materials from freezing.
In the event that fine materials are too dry, increasing their moisture content is extremely difficult during the winter season since the materials would be in great danger of freezing during watering operations.

[1] Société d'énergie de la Baie James, 1987, Le complexe hydroélectrique de la Grande Rivière Réalisation de la première phase, Les éditions de la Chenelière.
[2] Andersland, O.B. and Ladanyi B., ed. 2004. Frozen Ground Engineering – 2nd Edition. Wiley.

During thaw periods fine materials can become very moist. Their placement cannot take place when their water content is too high. Travelling of construction equipment on such materials is difficult due to their low bearing capacity. A special treatment (heating, harrowing…), is then required to reduce their water content and thus enable their placing under acceptable conditions.

## 7.3. Performance of dams in cold areas

### 7.3.1. Frost heave

Frost-susceptible soils that are not protected against frost effects are exposed to the heaving problem due to the formation of ice lenses. The presence of such lenses results from the formation of ice crystals fed by the migration of water from the lower non-frozen layers towards the frost-affected soil portion.
There is ice lens formation where the three following conditions exist([3] and [4]):

- Frost-susceptible or frost-sensitive soil;
- Temperatures below freezing;
- Presence of water near the frost-affected area.

To limit frost action the embankments can be protected with insulation. To design such insulation the frost depth penetration must first be determined. The insulation may consist of synthetic materials, such as expanded cellular polystyrene, or natural materials, such as granular materials.

### 7.3.2. Settlements

The major cause of settlements in cold regions is the consolidation of foundation soils following water drainage during the thaw period([5]). Settlements are more significant where the soil ice content is high.
During the design phase determination of acceptable settlements is important. Settlement estimations are based on the data gathered and experience. The calculated values must be taken into account when setting the final level of the dam crest so that this crest will never be submerged.
Several alternatives are used to limit or compensate for these settlements, including flattening the slopes and raising the crest by a value equal to the expected settlements.
The thawing and compaction of the foundation or excavation bottom and the replacement of frozen soils remain the most effective solutions for limiting probable settlements considerably.

### 7.3.3. Stability

The thawing of ice-rich permafrost foundations generates large water volumes. Where the soil permeability is low there is a significant pore pressure build-up. These pressures lower the soil resistance and may lead to structure stability problems. Indeed, the shear strength is reduced when the thaw face forward speed exceeds the speed at which water is expelled.

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[3] Zaikoff D.W., 1973. Pénétration du gel dans les sols et les barrages en terre et/ou enrochement. Hydro-Québec.
[4] Rooney J.W. and Johnson E.G., 1988. Embankment Stabilization Techniques. Design and Construction in Cold Regions. E.G. Johnson: pp. 13-34.
[5] Icold, bulletin 105, 1996. Dams and related structures in cold climate. Design guidelines and case studies. Icold.

Vertical sand drains connected to a horizontal drainage system have been used to accelerate the dissipation of excess pore pressures and thus increase the shear strength of foundation soils at the thaw face([6]). Using berms is also a way of improving structure stability.
For dams built on foundations which were previously thawed or replaced with control fills conventional stability analyses are used.

7.3.4. Permeability of the impervious zone

The till permeability may be modified by freeze/thaw cycles ([7], [8] and [9]).

**7.4. Design and construction**

7.4.1. Construction methods for cold regions

Construction methods depend on foundation soil type and thickness, and the climate in which a dam will be built.

*7.4.1.1. Active method*

This is the method most commonly used in North America. This method consists in eliminating the frost conditions in order to execute work under conditions similar to those for temperate regions. This method is most appropriate where the foundation material is an incompressible material (very dense granular material) or bedrock. Compressible foundation soils where the frozen layer is not very thick must be excavated and replaced with suitable material.
Where the frozen foundation soils are not removed and likely to thaw following the construction of the dam, major settlements will probably occur and cracking of the impervious core is then unavoidable. The crest of the structure is generally raised by a height equivalent to the anticipated settlements.
The filters and the water drainage and channelling system are essential for controlling infiltrations through and under embankments. Their design is similar to the procedures followed for dams in other regions by taking into account the large water volumes generated during thaw periods. Special measures must then be taken to prevent ice from clogging drainage pipes.

*7.4.1.2. Passive method*

This construction method is not commonly used and essentially applies in northern regions with year-round very cold climates. In addition to the climate, this method requires the use of artificial refrigeration systems during the construction and operating periods, to ensure that structure foundations remain frozen. When the foundation and fill materials are saturated with ice and kept frozen they can act as an impervious barrier to retain water.

[6] Andersland, O.B. and Ladanyi B., ed. 2004. Frozen Ground Engineering – 2nd Edition. Wiley.
[7] Viklander P. and Eigenbrod D., 2000. Stone movements and permeability changes in till caused by freezing and thawing. Cold Regions Science and Technology, 31: pp. 151-162.
[8] Viklander P., 1998. Permeability and volume changes in till due to cyclic freeze/thaw. Canadian Geotechnical Journal, 35: pp. 471-477.
[9] Konrad J.M., Bouchard R., Bigras A., Hammamji Y. 2006. Évaluation des effets de cycles de gel-dégel sur trois remblais d’essai à Péribonka. Annual Conference of The Canadian Dam Association, Quebec City, Canada.

Sayles([10]) reported that frozen embankments operating with permanent reservoirs are located in regions where the average year-round temperature is -8°C or colder. Even at such temperatures embankments ten (10) metres and over in height require additional artificial refrigeration for at least part of the year to ensure that embankments and foundations remain frozen at all times.
The foundation has been kept frozen under the embankment of the A154 (2001)([11]) and A418 (2005-2006) dikes at Diavik within the scope of a mining project executed in Northern Canada. Keeping the soil in a frozen state was achieved through a thermosiphon system. This system consists of a closed cooling system loaded with $CO_2$. It enables transferring heat from the subsurface to the atmosphere.
The HESS CREEK ([12]) dam in Alaska was built on a permafrost foundation approximately 60 metres in depth. A cooling system was provided to keep the foundation frozen under the dam embankment. The dam was operational from 1946 to 1958 and its performance was generally satisfactory.

7.4.2. Excavating

In cold regions frozen soils form a very dense mass similar to that of concrete or rock. Generally speaking, excavating frozen soils requires using the same means as those used to excavate rock. Where the frozen soil depth is minimal mechanical means are used. On the other hand, blasting must be used where the depth and soil volume to be excavated are large.
No special provisions are required for excavating rock in winter. Standard methods are used. For treating bedrock with concrete placement heating shelters must be installed during the winter season.

7.4.3. Injection into bedrock

Injection work is generally not allowed where the bedrock is frozen([13]). Prior to commencement of work and when air temperatures are below 0°C it is recommended that bedrock temperature measuring instruments (thermocouples or methylene blue) be installed to ensure that the bedrock portion to be injected is not affected by frost. Exceptionally, special grouts may be used for injection work executed at temperatures below freezing, depending on the manufacturer’s specifications.

Where the bedrock frozen depth is shallow the use of heated shelters may be considered in order to thaw this foundation and thus enable executing injection work.

7.4.4. Placing of materials

*7.4.4.1. Frost-susceptible materials*

Most of the time, the placement of frost-susceptible materials is suspended during the winter season (November to April) in Northern Quebec given the placing problems and difficulty in

[10] Sayles F.H., 1987. Embankment Dams on Permafrost. Design and Performance Summary, Bibliography and an Annotated Bibliography. Cold Regions Research and Engineering Laboratory, US Army Corps of Engineers.
[11] Rattue D.A. and all, 2006. Design, construction and operation of the A154 dike at Diavik. Twenty-second congress on large dams - Barcelona 2006. Icold. pp. 411-442
[12] Rice E.F. and Simoni O.W., 1963. The Hess Creek Dam, Permafrost international conference Proceedings, Lafayette., Indiana, pp. 436-439
[13] Weaver, K.D. and Bruce, D.A., 2007. Dam Foundation Grouting, American Society of Civil Engineers.

reaching the specified compactions.The materials already in place are often temporarily protected with an insulating blanket to prevent frost penetration([14]). In the spring the frozen materials must be excavated and replaced prior to the continuation of backfilling work.
In particular cases placing these materials was carried out during the winter season.
The rehabilitation of the Sainte-Anne Lake south-east dike([15]) At the Hydro-Québec Toulnoustouc hydro-electric power project 120 km north of Baie-Comeau (Province of Quebec, Canada), which consisted, among others, in placing an upstream till blanket, was carried out during the winter of 2002. This time of year enabled executing work when the reservoir water level was low. The till embankment material was placed successfully due to a number of precautions having been taken. The fill material contained neither ice nor snow and was not agglomerated by frost. The till was transported by heated-body trucks to the structure site. The till water content was very close to the standard Proctor-test optimum water content (optimum –1%). The average compaction achieved for the placed and compacted till was up to 97% of the standard Proctor-test maximum dry density.
The construction of the Frégate TB-1 dike within the scope of the “La Grande-3” project located approximately 970 km north of Montreal, in the north-west portion of the Province of Quebec (Canada), took place during the winter of 1981([16]). Part of the till was dumped up to 0.9 m above the water level. Subsequently the till was placed in lifts and compacted with 10-tonne vibrating rollers. Since the till water content was on average 3.5% below the optimum water content, the lift thickness was reduced from 300 mm to 200 mm and the number of passes made by the heavy vibrating rollers was increased from 4 to 6 passes. Despite these additional measures the average compaction percentage was 95% of the standard Proctor-test maximum dry density instead of 97% as seeked.
To sum up, placing frost-susceptible materials in winter is possible where precautionary measures are taken. The water content must be near the optimum water content and special measures must be taken to prevent materials from freezing and facilitate their placement at low temperatures. The high cost and difficult placing of materials in winter in most cases justify suspending work during this period.

*7.4.4.2. Frost-resistant materials*

Sand and gravel and rock fills must be free of snow and ice prior to and during their placing.
Sand and gravel having a low water content may be adequately compacted when the temperature is lower than 0°C by using heavy rollers([17]). As for rock fills from quarries, these are placed and compacted during the winter months using the same methods as during warmer months, while paying special attention not to include snow and ice during their transportation and spreading.

*7.4.4.3. Temporary protection of frost-susceptible materials*

Generally speaking, till placing work is suspended in winter in Quebec (Canada). Large dams require more than one construction season. A number of insulating blankets have been investigated in Canada in order to limit frost penetration into the till([18]). During work

[14] ICOLD, Bulletin No. 69, 1989. Moraine as embankment and foundation material. ICOLD.
[15] Morency J.Y., Hammamji Y., Beauséjour N., Malenfant S. 2006. “Réhabilitation de la digue sud-est de la Toulnustouc. Barrages : passé, présent et future”. Annual Conference of The Canadian Dam Association, Sept. 30-Oct. 5, 2006.
[16] Société d'énergie de la Baie James, 1987, Le complexe hydroélectrique de la Grande Rivière Réalisation de la première phase, Les éditions de la Chenelière.
[17] Andersland, O.B. and Ladanyi B., ed. 2004. Frozen Ground Engineering – 2nd Edition. Wiley.
[18] ICOLD, Bulletin No. 69, 1989. Moraine as embankment and foundation material. ICOLD.

suspensions in winter till cores are generally protected against frost penetration by the placing of sand and gravel or snow, or a stacking of these materials. Frost penetration is in the order of 3-3.5 m where the till is not protected and exposed to cold with a freezing index of 2000°C/day.
Insulating blankets were used in the Outardes 4 project (located in Northern Quebec) in order to protect the till core between two construction seasons:

Winter 1965-1966: The use of a 1.5-m-thick granular layer enabled limiting frost penetration into the till to approximately 300 mm.

Winter 1966-1967: The addition of a 1.5-m-thick granular layer and 1.5 m of natural and artificial snow enabled limiting the frost penetration into the till to 15 mm.

At James Bay (Northern Quebec) a 2-2.5-m-thick granular layer is often used to limit the frost penetration into the till between two till-core dam construction seasons.

*7.4.4.4. Placing materials in water*

Placing frost-susceptible or frost-resistant material in water in winter is common practice in Quebec (Canada) for the construction of coffer dams, temporary dikes and any other structure where settlements can be tolerated or corrected.
Prior to dumping the materials, the surface ice slabs must first be removed to prevent their being mixed with the embankment material. The voids created by the thawing of the ice slabs can induce appreciable structure settlements.
Materials placed in water are loosely compacted and can be liquefiable in case of an earthquake.

7.4.5. Freeboard

The freeboard is defined as being the difference between the level of the camber-less retaining structure crest and the maximum reservoir operating level. Its role is to prevent, among others, the submergence of an embankment following the heightening of the water body and upflow of waves on the slope. It can also be used to compensate for the settlements, if any, that were not considered in the camber.
In Northern Quebec the frost penetration depth for an operational dam can reach up to 8-9 m (Caniapiscau dam, at the La Grande Hydroelectric Development). Raising the dam crest with an embankment having an equivalent thickness in order to prevent the dam core from being affected by frost was considered too costly a design. A study([19]) has shown that using an additional 3-m-thick embankment load prevents ice lens formation in the till and therefore, core heaving. In northern regions the practice selected up to now for the Hydro-Quebec (Canada) hydro-electric power projects entails using a minimum three-metre freeboard.
The geotechnical freeboard can be reduced where the embankment is protected against frost penetration by a synthetic insulating material.

7.4.6. Instrumentation

Depending on the importance of the dam, instruments are installed to track dam performance during construction and throughout the operating life of the structure. The instrumentation is

[19] Paré J.J. and al., 1978. Frost penetration studies in glacial till on the James Bay hydroelectric complex. Canadian Geotechnical Journal, 15: pp. 473-493.

important for dams built in cold regions. An inspection program must be defined during the design phase in order to provide for instruments that will complement visual inspections.
The instruments are used to:

- Monitor settlements and heaves due to the freezing and thawing of embankment and foundation materials;
- Validate and track frost/thaw penetration;
- Monitor pore pressure variations in cores and foundations;
- Monitor the progress of percolation flows through embankments and foundations.

The key instruments used are: survey monuments and clinometers, to measure displacements; thermocouples and thermistor chains, to measure temperatures; a piezometer, to measure pore pressures; and weirs, to measure water infiltration flows.

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## 8. EMBANKMENT DAMS IN TROPICAL CLIMATE

Hot climatic conditions may significantly affect various aspects of the embankment dams design and construction. The main problem may be to reach an acceptable final moisture content in the clay core.

- The problem is serious in hot dry climates where there is high loss of moisture during excavation carrying, spreading and compaction. But the moisture content may be increased quite easily at low cost for getting the final acceptable content.
  This requires however trial embankment that simulate the whole construction steps for evaluating the right adjustments and serious control during construction.

- The problem is also very serious in tropical countries but the solutions for reducing the moisture content are much more expensive than for increasing it and the present chapter below focus thus on tropical climates.

20% of world land surface is in tropical climates, i.e. warm and wet; yearly rain is usually well over 1 m and up to 5 m.
Fine materials of earthfill dams or of the core of rockfill dams may be very sensitive to these climatic conditions and may require specific designs or construction methods. The main problem is the moisture content of the clay materials because the natural moisture content is usually well over the optimum.
Two solutions may be used:
- To decrease the water content
- To accept consequences of a high water content on design, construction and dam behaviour.

### 8.1. Decreasing the water content

There are some methods to reduce the water content of the soils including air drying (spreading it in relatively thin layers to be dried by air), heating (to heat the material in the high capacity kilns, it has been done since 1940 in Mud Mountain dam at Washington) and mixing with dryer granular material. This mixing method is mainly used in Japan. In this method by mixing coarse granular materials (mainly sand and gravel) with a lower water content than fine materials, a mixture is obtained which has a higher modulus of deformation, more than or equal shear strength and lower moisture content and can be more easily compacted by rollers.

But where the natural moisture of the material is well above the optimum moisture content, the practical difficulties of drying the clay have often caused serious delays to construction projects.

### 8.2. Accepting a water content well above the optimum (Wet Core)

This solution may be cost effective but requires specific designs and construction methods.
It has been used successfully 100 years ago in United Kingdom (puddle clay core) for dams usually lower than 30 m.
It has now been used in high dams and severe conditions such as:

- Monassavu Rockfill Dam in Fiji (1982), 85 m high with an unusually wet low plasticity core where the yearly rain is over 5 m!
- Chivor Dam in Colombia, 237 high
- Wadalintang Dam, 120 m high in Indonesia.

For embankment dams with wet clay core the management of the construction schedule is very important, for instance coarse grain and less plastic materials should be used in parts which can be easily constructed under rainy weather and confine other fills of weaker materials, and finer materials of the core can be used in dryer days.

Rate of construction should be controlled by pore pressure development and decrease if high rates of pressure develop.

Another important aspect of the wet core construction is the compaction process of such materials. Because of the wet and soft condition of the material, it is not possible to use conventional heavy weight rollers and the compaction should be made by means of light compactors such as ground pressure bulldozers and light pneumatic tire rollers.

The acceptable compaction percent of the material would be less than the usual values (95-98%) and the allowable water content of the material is more than the normal range. These parameters should be selected according to the natural conditions of the materials and their effects should be considered carefully in the design.

The basic principle of wet core design is to ensure a low undrained shear strength during construction. This prevents the core transferring its load by arching action to the adjacent shells by shear stresses developed in it. Consequently the eventual total vertical stresses throughout the core are likely to exceed those produced in stiff cores resulting from conventional compaction. Furthermore, the associated total horizontal stresses are likely to attain a higher proportion of the vertical stresses, and thus be higher in absolute terms than for a conventional core.

In a central or slightly inclined wet core dam the vertical and horizontal stresses will approximate to the major and minor principle stresses respectively, which will thus be closer in value to each other and higher than in a stiff core. Hydraulic fracture requires that the seepage pressure of the pore water in the core at any point exceeds the minimum total principal stress, followed by propagation across the core until a crack exists from upstream to downstream. This risk is considerably reduced by a soft wet core design.

The greater deformation resulting from the use of wet soft cores need accommodation so as to avoid zones of high strain which, if excessive, could themselves be at risk of hydraulic fracture. This risk is reduced by appropriate geometric design, particularly of the longitudinal profile of the dam.

The material of wet cores has usually a very low shear strength due to light compaction efforts and because of high water content and low permeability, significant excess pore water pressures develops during construction. These are believed to be the most important aspects that should be considered in the dam design.

## 8.3. <u>Economical aspects</u>

As a general fact, increasing water content of the fine material is much easier and very less expensive compared to moisture content reduction techniques. Thereupon the construction cost of clay cores is generally less in hot dry weather compared to tropical climates. The different methods mentioned for dealing with the high moisture content of the core materials in tropical climates could be generally reviewed from the economical point of view.
The air drying method (if possible according to the weather condition) usually is very time consuming and may affect seriously the construction schedule that could indirectly increase the project cost. For instance in an area with the average humidity of 90%, reducing the clay material moisture content by only 2%, in a sunny day, could take about 6 hours working of loader.

The costs of mixing method mainly consist of the preparation charge of coarse granular materials instead of clay; say in a half volume of the core material. The preparation and filling fee of the coarse granular materials is about 1 USD per cubic meter less than that of clay materials. Depending on the difference in the carrying length of the fine and coarse materials the total preparation fee of the material could increase or decrease with a rate of about 0.25 USD per cubic meter per kilometer. In addition the cost of mixing process of the clay and coarse granular materials is around 0.4 USD per cubic meter.
Heating method (to heat the material in the high capacity kilns) is very difficult and too expensive especially in the case of high material volume.
In the case of using wet core method, the significant reduction of the strength and deformation parameters of the core materials affects the geometry of the dam and increases the dam body volume. Thereupon for economical evaluation of this method the additional cost of increasing the dam body volume or any specific application that may be required to ensure the dam body stability must be compared to the costs associated with the other possible methods mentioned above. In order to obtain the increase in the project cost due to increase of the dam body volume, it should be considered that the preparation and filling fee of the clayey materials, coarse granular materials (sandy gravel) and rockfill materials is about 3, 2 and 6 USD per cubic meter, respectively. The carrying cost of material is about 0.25 USD per cubic meter per kilometer.

# 9. UPGRADING OF DAMS

## 9.1. General consideration

In general, existing dams are upgraded either to increase the benefits they provide or to improve or increase its safety, or of course, to meet both objectives. The improvement of the benefits provided by the dam is almost always represented by the enlargement of the reservoir capacity and/or by the rise of the elevation of the operational reservoir water level for increasing energy supply. The improvement of the safety of the dam, in most cases, is associated with the increase in the spillway capacity to make it able to discharge larger flood flows than those that were used to size the original structure when the dam was designed.

Old dams can suffer deterioration in the materials that constitute their structure or in their foundations. The same can also happen with the structure and equipment of the spillway and other appurtenant organs. The work carried out to restore safety or performance of deteriorated dams, even when these characteristics are not enhanced in relation to the original design, is precisely termed as rehabilitation but for the comprehension of this text, it will also term, as well, as upgrading of the dam.

The increase in the capacity of existing spillways to meet revised safety criteria or revised hydrologic data due to transformations in the contributing catchment area is also termed as dam upgrading. This particular issue is discussed in the bulletin 144 and appendices.

In any case the engineering and design work required to meet the objectives of upgrading the dam, could be equivalent or similar to the work required to build a new dam, with the important difference that modifying an installation which is in operation may raise very important additional difficulties when depleting the reservoir is not possible.

The increase in the maximum reservoir water level, meaning of course increasing the dam height, has been practised primarily to increase the benefits provided by the dam, which may provide extra volume for flood attenuation or for flow regulation for energy production or water supply. In some cases, the original design has considered the construction of the dam in two stages and provisions have been established for future dam height increase. Very well known cases are Guri concrete gravity dam, in Venezuela and Fortuna embankment dam, in Panamá. However there have been numerous examples of dam height increases where the original design did not consider future modifications and decisions for crest raising have been taken after commissioning and mid-way during the project life. Normally these modifications correspond to increases in dam height by a modest absolute figure, of say no more than 5 metres in structures whose original maximum height is of the order of 20 to 30 metres, which however, is an important modification in the original designed conditions. It is clear that these rises create a substantial increase in the loading of the structure and design revisions must carefully evaluate specific stability conditions, especially in the foundations, to ensure the compatibility with the altered situations. A comprehensive and detailed campaign of investigations is mandatory in these cases, to support a design revision programme.

The physical way of increasing the dam height is generally simpler in embankment dams. One relatively simple alternative is to place a “parapet” wall on top of the original crest as in Pitinga Dam [1] in Brazil or in Chisacá Dam [2] in Colombia. In these two ECRD the parapet was created by concrete boxes resting on top of the core. The concrete boxes can be moulded in place or be pre-formed, and be self-stable or filled with soil or crushed rock to increase weight and stability. Of special importance is the preparation of the contact surface of the base of the concrete with the core to preclude the possibility of seepage and infiltration along the contact area.

Other alternatives for raising embankment dams correspond to place additional earth-fill on top of the crest, which requires an original very wide crest or the continuation of the additional layer along the downstream face. The volume of the additional fill can be reduced by increasing the slope of the extra fill, but in general the volume of new fill will be important and the amount of work required, rather significant. However if the increased height is large, this may be the best economical solution.

Raising concrete gravity dams requires modifying the profile of the structure to allow stability under the new loading conditions. If the original design did not consider heightening of the dam a thorough investigation must be carried out to define design and construction solutions.

Besides the construction of the extra height of the dam, an additional complication is the need to change the spillway configuration to meet the new higher reservoir level. The particular solution adopted depends on the type and capacity of the spillway.

Surface spillways require generally the increase in crest elevations which is achieved by adding extra concrete layers to shape the new crest sill and discharge channel. One special and very important consideration is related to the probability of floods occurring during the works and how to handle the need of allowing the discharging flow over the unfinished construction area or through an isolated part of the existing spillway.

With gated spillways this may be done by working in individual passages. Possible solutions correspond to increasing the level of the sill of the gate or the level of the gate sill, closing the passage with the use of the protecting stoplog. However, higher gates produce increased load on the trunnion beans supporting the gate, and this may be a serious limitation. An illustrative case of sill heighten in a surface gated spillway (original capacity 16 000 $m^3/s$) was carried out in Cachoeira Dourada Dam, in Brazil, where a 1.5 m increase in the maximum reservoir water level was successfully achieved without depleting the reservoir original operating level, and therefore maintaining the power-plant operation throughout the modification on the spillway [3]. In this case a floating steel gate was moved through the reservoir to each passage and then submerged and turned up to a vertical position closing the passage. This allowed continued operation of the 620 MW power plant, and protection against floods occurring during the spillway modification.

For ungated surface spillways the work of raising the elevation of the sill requires generally isolating the work area to allow the construction operation (for irrigation dam, the work is possible during a dry season). In low dam projects and long spillways this may be simpler by constructing soil cofferdams upstream, and dividing the length into two or more stretches. Where this cannot be done, because of short length of the spillway or large height of original structure, different solutions have been used, such as building an additional provisional passage of flood in a different part of the project, and thus allowing total cofferdamming of the existing spillway. Each site and project configuration combined with the modification objectives, requires a special and particular solution.

Cost considerations are of course of paramount importance in the formulation and design of upgrading an existing dam. However in this case, if the original structure is in operation and producing benefits, the proposed solutions must, naturally, take into consideration the potential deficit in the benefits during the construction period, so that quite often elaborate and expensive construction procedures can be fully justified. In fact, more than in a new job, each case is a special case, and it is very difficult to establish standards and comparison figures.

## 9.2. <u>Some recent innovations in dam upgrading which allow cost savings</u>

Some recent innovations have lead to interesting cost savings. More details can be found in [4] and in the bulletin 144.

### 9.2.1 <u>Increase of spillway capacity and/or increase of the reservoir capacity and/or a combination of both</u> (ref. Bulletin 144)

For this purpose, flashboards for small dams and fuse dykes for higher dams were already used in the past but they were not generalized due to their disadvantages and lack of reliability. Some recent innovations such as inflatable gates and flap gates supported by air bags (t Obermeyer GATES) can be economic alternatives in some cases. A recent solution, but already used successfully for high discharges in different climates worldwide, is the fuse gates (type Hydroplus) placed on gravity structures which avoid the main inconvenient of the fuse dykes. Other types of concrete fuse plugs, simpler than the concrete fuse gates, were experimented with good result for small irrigation dams with low overflow depths but with a saving lower than for fuse gates.
Labyrinth spillways, such as the more recent P.K. Weirs solutions may at least double the discharge.
Solutions above may be adapted to existing free flow spillways or added as emergency spillways to existing gated spillways.

### 9.2.2 <u>Use of RCC to raise concrete dam or spillway crest or to allow the overtopping of embankment dam</u>

The use of RCC for this purpose is more and more popular in United States and worldwide. This solution presents many advantages (safety, low cost, rapidity of construction) and some examples of recent realizations are provided in [4].

### 9.2.3 <u>Use of geo-membrane in the heightening of dam</u>

The use of geo-membranes for concrete or embankment dams, to provide the watertightness of the heightened part and its link with the older part, is a recent application of this material. Some examples are given in [4] which show how this application has permitted to address this issue with a technical and economical good result.

As indicated in the General Report Q90 [5], it is important to note in this chapter, concerning cost savings in upgrading of dams, that:

- It may be more cost-effective to repair a structure early in the deterioration process, rather than wait until a major upgrade will be needed to save the structure. What is not a dam safety problem today, could be very costly to solve when it will become one.
- When applicable, the raising of a dam is a typically cost-effective solution for adapting an existing dam to an higher design flood.
- There are often several reasons to raise a dam in the future and it may be then wise to consider already at the design stage of a new dam, whether a later raising could be required and, if so, select a cross section which favors subsequent modification, even if its initial construction requires additional work or costs.
- Costly upgrading may be avoided:
    - ✓ by better evaluating the seepage and the risk of piping in embankment dams; the recent methods of seepage measurement, such as with the optic fibre, can limit the cases where rehabilitation works deem necessary,
    - ✓ by analyzing more precisely the seismic risks with modern methods (dynamic and inelastic analyses), it is possible to avoid unnecessary upgrading due to sometimes too conservative analysis assumptions,
    - ✓ more generally, by adequate field and laboratory testing and studies which resulted sometimes in considerable cost savings in the upgrading of many projects.

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## 10. Conclusion

- The differences between dams are extreme: they apply to height and storage and also to local conditions, materials, flows, impacts, risks …
  Regulations may hardly be well adapted and may be very expensive. Questionable common set ideas or rather standardized design criteria and specifications increase also the costs. Specific conditions require a specific design for optimizing costs.

- Choosing for each dams site the most cost effective design is difficult because unit costs vary with each design and with construction methods: advices and/or alternative proposals from contractors may be cost effective.

- As for most other human activities, innovation may reduce costs and improve safety or environmental impacts.

- The worldwide number of fatalities of workers during dam construction is much higher now than the number of fatalities from dams failures: a significant increase of works (and costs) for a small increase of dam safety may thus increase the overall human risk.