Small Dams Safety Guidelines

EASTERN NILE TECHNICAL REGIONAL OFFICE (ENTRO)

Nile Basin Initiative (NBI)
Eastern Nile Subsidiary Action Program (ENSAP)
Eastern Nile Technical Regional Office (ENTRO)

Small Dams Safety Guidelines

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Cover Photo: Mai-Gundi Small dam, Tigray Ethiopia

FOREWORD

I am pleased to launch this Eastern Nile Dam Safety Training Module for planning, design, construction, operation and safety management of large and small dams.

I commend ENTRO for having completed and availed this document at this critical juncture in the water resource development history of Eastern Nile. Dam safety will become a major focus of water resource planners and managers of Eastern Nile for the foreseeable future.

I trust this document will be of practical use and thus make critical contribution to the institutionalization of dam safety in Eastern Nile.

H.E. Mutaz Musa Abdalla Salim
Minister, Water Resources and Electricity, Sudan
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PREFACE

As a result of countries' enhanced efforts to tap the water resources potential of the Nile, the number of large and small dams on the Eastern Nile stretch of the Nile basin has been steadily increasing over the years. These investments are expected to generate economic benefits yielding much needed energy and food demanded by steadily growing populations. While these developments are welcome, it is also necessary to ensure the safe operation of these dams. Dam safety, therefore, will be one area where Eastern Nile countries' interests will converge, other differences notwithstanding. Dam safety will be the glue that holds Eastern Nile countries together now and in the future. Dam safety is a glaring example that demonstrates the fact that Eastern Nile Cooperation is not an option, but an existential necessity!

ENTRO has been cognizant of this fact and has striven to take the first steps in laying the foundation for the institutionalization of dam safety. Over the last two years ENTRO has identified potential technical and institutional gaps in Eastern Nile Dam safety management, large and small, and designed training modules and undertook a capacity building program targeting a range of critical stakeholders including: parliamentarians, policy makers, water resources planners and managers, dam owners and operators, academia and civil society. Thus far about 200 Eastern Nile professionals have been trained in dam operation; dam safety management in transboundary context; environmental and social considerations associated with dam safety; safety assessment of dams; planning, design and construction management of water infrastructure. To avail the lessons of experience from within Eastern Nile and worldwide to those who could not take part in these trainings and to support their adoption, ENTRO has produced three critical documents namely 1) Eastern Nile Reference Dam Safety Guideline 2) Small Dam Safety Guideline 3) Dam Safety Training Module.

Dam safety management needs to be institutionalized. The above are only the starting points. Next steps will include establishment and consolidation of national dam safety units in each Eastern Nile country and development of Eastern Nile Dam Safety Regulatory Framework, which will deal with the legal and institutional dimensions.

It is with a sense of satisfaction I launch these Dam Safety Guidelines and Training Module. I trust the relevant Eastern Nile stakeholders will find these very useful and relevant to their work.

Fekahmed Negash Nuru
Executive Director, ENTRO

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PREAMBLE

The Nile Basin Initiative (NBI) is a transitional cooperative mechanism of nine riparian countries.[1] It was established in February 1999 to realize a jointly articulated Shared Vision whose objective is "To achieve sustainable socio-economic development through the equitable utilization and benefit from the common Nile Basin water resources". Its overriding objectives are poverty reduction, reversal of environmental degradation, promotion of economic growth, increased regional cooperation and integration, and enhanced regional peace and security. The NBI Secretariat (Nile-SEC) is based in Entebbe, Uganda

Eastern Nile Sub Basin countries have made significant strides in strengthening their cooperation since the launch of the Eastern Nile Subsidiary Action Program (ENSAP) in 1999, within the framework of the Nile Basin Initiative. The Eastern Nile basin countries comprise of Egypt, Ethiopia, South Sudan and Sudan. The Eastern Nile Regional Technical Regional office (ENTRO) is based in Addis Ababa, Ethiopia.

This guideline has been prepared as part of the Eastern Nile Technical Regional Office (ENTRO) Dam Safety Program. The dam safety program includes an updated assessment of the baseline conditions as a starting point, the development of a generic set of dam safety guidelines applicable to the region and available for adoption by each of the EN countries, a "Road Map" for the preparation of EN dam safety regulation framework and training in dam safety management. This Dam Safety Guidelines is intended for Eastern Nile small dams safety managment.

This guideline draws concepts and processes from various international guidelines from within the ICOLD family of Technical and National Committees and also various US Federal Agencies. Appreciation is expressed to the developers and authors of those guidelines.

[1] The founding NBI Member countries are Burundi, D.R. Congo, Egypt, Ethiopia, Kenya, Rwanda, Sudan, Tanzania and Uganda. The Republic of South Sudan joined in 2012 while Eritrea is Observer.

ACRONYMS

ACB:	Articulated Concrete Blocks
CIRIA:	Construction Industry Research & Information Association
EAP:	Emergency Action plan
EIA:	Environmental Impact Assessment
ENTRO:	Eastern Nile Technical Regional Office
FEMA:	Federal Emergency Management Agency (USA)
ICOLD:	International Commission on large dams
NRCS:	National Resource Conservation Service
PCC:	Potential Consequences Classification
PMF:	Probable Maximum Flood
RCC:	Roller Compacted Concrete DAMS
SOP:	Standing Operating Procedures
UK:	United Kingdom
USA:	United States of America
USACE:	US army corps of Engineers

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1. INTRODUCTION

1.1 BACKGROUND

Dams provide benefits for a number of reasons. They commonly provide the following services:

- Water supply
- Irrigation water to
- Power generation
- Storage of flood water
- Provision of fishery sources
- Aesthetic benefits
- etc.

While dams provide potential benefits, they also have a risk associated with their design, construction and operation.

As various dam failure and incidents around the world have shown some loss of life, environmental damage, social disruption and economic losses, there is a need to exercise due care at all levels in achieving dam safety. There is a need to embody the level of advice necessary for full implementation of dam safety procedures. The benefits which will accrue from the promotion and achievement of adequate dam safety practice include environmental protection, public confidence and commercial benefits to the dam owners.

Dams with their rated structures and storage reservoirs have a special nature because of their scale, the water force at work and the use of natural ground to form the major part of the reservoir containment. Most other man-made works are built of high strength manufactured materials, involve controlled geometry, do not involve large storage of water, and generally do not use the foundation other than to support the works themselves. In case of dam, however, consideration has to be taken of:

- Dam site topography which usually can not be altered significantly because of cost
- Dam site and regional geology which greatly influences water retention and structural safety
- The most appropriate materials to build the dam from and the dam structure arrangements to assure a safe dam operation
- The earthquake forces which the dam and its stored contents may experience
- The potential floods which may pass through the reservoir and how they can pass without risk of dam overtopping or erosion damage
- Surveillance, maintenance and operational procedures to ensure works as intended
- Management of sediment passage down the river.

All dams constituting a special class of structure, each dam will have unique characteristics, particularly in terms of site geology and geometry. The variation in geology, building material types, geometry, earthquake and flood risk and the like, mean that it is not practicable to develop a standardized code-type design for dams. Each dam must be treated individually, taking all relevant factors into consideration.

1.2 OBJECTIVE

The overall dam safety objective is to protect people, property and the environment from consequences of mis-operation or failure of dam. To ensure dam and reservoir are operated and that activities are conducted so as to achieve the highest standards of safety that can be achieved, measures have to be taken to achieve the following dam safety objectives:

- To control the release of damaging discharges downstream of the dam
- To restrict the likelihood of events that might lead to loss of control over the stored volume and spillway and other discharge
- To mitigate through onsite accident management and emergency planning the consequences of such events if they were to occur

These fundamental safety objectives apply to dam and activities in all stages over the life-time of a dam, including planning, design, construction, commissioning and operation, as well as decommissioning and closure.

Since dam safety is among the nine key environmental and social issues indentified in the Joint Multipurpose Program study and was also addressed in the preliminary assessment report published by ENTRO in 2013, ENTRO has taken the initiative to do the followings:-

- Development of a Reference dam safety guidelines applicable to the region and available for adoption by the EN countries;
- Development of small dam safety guidelines applicable and available to EN countries;
- Road Map for the preparation of EN dam safety regulation framework
- Training in dam safety management

The Reference Dam Safety Guidelines for EN Countries (July, 2014) define the overarching framework for dam safety of EN dams. In particular the Potential Consequences Classification system presented therein applies to all dams. This document presents the Small Dam Safety Guidelines which provides additional specific guidelines for small dams. If there are discrepancies between this guidelines and the Reference Dam safety guidelines, the latter shall take precedence.

Since the consequences of most small dam failures do not go beyond 10 km and accordingly has no impacts on downstream countries, the trans-boundary dimension has not been considered in this guidelines. However, there will be cases where the impacts may be more extensive and trans-boundary issues will need to be considered as described in the Reference Guidelines.

This Guidelines is prepared as a guide for small dam owners, engineering, Government agencies, developers and contractors who are in charge with the design, construction, operation, maintenance and safety of small dams. It is developed mainly to the embankment dams which represent the very large majority of small dam in EN countries

This Guidelines is based mainly on the ICOLD bulletin on Small Dam Design, Surveillance and Rehabilitation issued by ICOLD in September 2011. However, Some other international experiences have been considered in developing this document.

1.3 DAM MANAGEMENT AND OWNERSHIP IN EN COUNTRIES:

The Eastern Nile countries (Egypt, Ethiopia, South Sudan and Sudan) have different dam safety system in terms of institutional arrangement, legislations, ownership, etc. in general, the ministry responsible for water resources is the one has the major responsibility on dam safety as an owner. The operation and maintenance mostly is the responsibility of number of governmental agencies/ departments (water, power, water supply), based on the purpose of the small dam. It is expected in the future with more tendency towards implementing decentralized approach, some of the operation and maintenance will be transferred to water user associations. The Egyptian code for water resources and irrigation works devoted one of its volume to address design criteria for dam and barrages. However, it has not address the operation, maintenance, surveillance, rehabilitation, emergency action plan, etc. In 2011, Ethiopia developed small dam safety guidelines which addressed planning, construction, surveillance, operation and maintenance, emergency action plan and decommissioning of small dam. None of the EN countries has legislations which directly devoted to dam safety, but some of dam safety related points have been reflected in water policy. The National Water Policy Draft of 2000 in Sudan " has a hydropower section which states an integral operation and maintenance of dams shall be adopted to optimize the use of water. Another part of the policy in relation to dam safety issues is addressed in the Disaster Management and Public Safety section of the policy. Also Ethiopian Water Sector Policy has also dealt with dam and reservoir management and operation and disaster, emergency and public safety.

1.4 OUTLINES OF THE SMALL DAN SAFETY GUIDELINES

This Guideline addresses the following areas:

Design criteria of small dam: The basic principle of design is to produce a satisfactory functional structure at a minimum total cost. The following criteria have been considered:

- The embankment must be safe against overtopping during occurrence of the inflow design flood by the provision of sufficient spillway and outlet capacity;
- The slopes of the embankment must be stable during construction and all conditions of operation, including rapid drawdown of the reservoir;
- Seepage flow through the embankment foundation and abutment must be controlled, so that no internal erosion takes place and so there is no sloughing in the area where the seepage emerges. The amount of water lost through seepage must be controlled so that is does not interfere with planned project functions;
- The embankment must be safe against overtopping by wave action;
- The upstream slope must be protected against erosion by wave action, and crest and the downstream slope must be protected against erosion due to wind, rain and cattle.

Key recommendations on design of various elements of small dam have been provided to guide the design process.

Surveillance of small dams: it presents the main recommendations in order to assure that the dams will behave appropriately and with a minimum cost. The construction of a dam can involve a significant investment and dam owners need to ensure that their money is well spent and that their dam becomes an asset.

Rehabilitation practices for small dams: it analyzes in detail ageing of embankment dams, updating of design standards and criteria and the deterioration of conditions affecting the safety of small dams, emphasizing the main remedial measures related to embankment dams.

Emergency Action Plan (EAP): it emphasizes the main points concerned the application of such plan to minimize the consequences of a dam failure or malfunction, regarding the population living downstream, presenting some recommendation about how to develop an EAP, evaluating the possible dam risks and the management of the dam safety as well as the responsibility of various actors.

Legislation & decommissioning: it points out the dam safety and security of people, property and environment downstream of dams and the important responsibility of the Government, who must legislate and enforce the rules through administrative agencies, departments and offices

Environmental and social factors: Safety guidelines for small dams that integrates environment and social issues will contribute to long and useful service life of. Environment and social considerations in small dam safety management are very important both for the safety of the dam and for communities and the environment upstream and downstream of the dam. this section provides recommendation on some of key environmental and social factors which might have a significant consequences on dam safety.

Safety of small dam: It discusses the peculiarities and conditions affecting the safety of small dams. among the causes of dam failure address in this section is overtopping, internal erosion, and slope instability.

Therefore, this document consists of 10 chapters as follows:

- Introduction;
- Dam Classification and Key Safety Principles;
- Types of small Dams
- Safety Of Small Dams
- Legislation & Decommissioning
- Environmental And Social Considerations For Dam Safety
- Features Of The Design Of Small Embankment Dams
- Surveillance Of Small Dams
- Rehabilitation Practices For Small Dams
- Emergency Action Plan (EAP)

2. DAM CLASSIFICATION AND KEY SAFETY PRINCIPLES

2.1. POTENTIAL CONSEQUENCES CLASSIFICATION

A Potential Consequences Classification (PCC) is a classification of dam according to their potential impacts during the event of failure. PCC needs to be applied to all dams in order to ensure that appropriate levels of investigation, design, construction control, maintenance and operation. A dam's PCC determines the frequency and magnitude of ongoing internal and external performance reviews. The PCC system included in the Eastern Nile Countries Reference Dam Safety Guidelines (July, 2014) and shown in Table 2-1-1 should be applied to all dams.

There are many factors which can affect the potential impact of dam failure. These can include:

- the dam height
- the volume stored behind the dam
- the shape and hydraulic characteristics of the downstream valley
- the downstream conditions, particularly habitation or public areas and the valley environment
- the effects to a community of depriving them of the stored water which may be critical for water supply.
- Other factors may affect the likelihood of a dam failure if they are not correctly dealt within the investigation, design, construction or operational phases of the dam's life. These may include:
    - Construction materials;
    - Proximity to active faults;
    - Catchment use (e.g. forestry operations with associated risk of debris)
    - Proximity to volcanic hazards; and
    - Landslides in the reservoir area.

Dams Potential Impact Classification shall be classified using the following criteria:

- Life Safety Risk
- Environmental Risk
- Societal Risk
- Economic Risk.

- **Life Safety Risk:**

The Population at Risk is the estimate of the total number of people likely to be within the inundated area at any time. It is not an estimate of the likely number of casualties due to a dam failure as no reduction is made for those likely to escape or survive the flood. However, this can be used as an approximation to the Loss of Life estimate for the purpose of establishing a PCC for small dams. Refer to the Reference Dam Safety Guidelines for limits to this approximation.

When estimating the potential loss of life the effectiveness of the Emergency Action Plan (EAP) should be considered. For example, if considering a natural flood, then the specific characteristics of the flood and evacuation scenarios should be considered to ensure that the appropriate level of safety is provided. As a starting point, a Population at Risk (PAR) assessment may be used to conservatively estimate the potential loss of life and classify the dam and determine required safety levels and procedures.

Table 2.1 1: **Potential Consequences Classification for Eastern Nile Dams**

Dam Class	Loss of life	Infrastructure, Economic and Social Factors	Environmental & Cultural Factors
VERY HIGH Level 4	Large potential for multiple loss of life involving residents and working, traveling and/or recreating public. Development within the potential inundation area (the area that would be flooded if the dam fails), considering both national and international or trans-boundary areas, typically includes communities, extensive agricultural, commercial and work areas, main highways, railways, ports and locations of concentrated recreational activity. Estimated loss of life could exceed 1 000.	Very high economic losses affecting infrastructure, public and commercial facilities in and beyond the inundation area considering both national and international or trans-boundary areas. Typically includes destruction of or extensive damage to large residential areas, concentrated agricultural and/or commercial land uses, hydroelectric generation facilities, highways, railways, ports and shipping facilities, power lines, pipelines, water supply and other utilities. Estimated direct and indirect (interruption of service) costs could exceed $100 million.	Loss or significant deterioration of nationally or locally important fisheries habitat (including water quality), wildlife habitat, rare and/or endangered species, unique landscapes or sites of cultural significance. Feasibility and/or practicality of restoration and/or compensation are low.
HIGH Level 3	Potential for multiple loss of life involving residents, and working, traveling, and/or recreating public. Development within inundation area typically includes highways and railways, ports, agricultural, commercial and work areas, locations of concentrated recreational activity and scattered residences. Estimated loss of life between 100 and 1 000.	Substantial economic losses affecting infrastructure, public, agricultural and commercial facilities in and beyond inundation area. Typically includes destruction of or extensive damage to concentrated agricultural and/or commercial land uses, hydroelectric generation facilities, highways, railways, ports and shipping facilities, power lines, pipelines, water supply and other utilities. Scattered residences may be destroyed or severely damaged. Estimated direct and indirect (interruption of service) costs could exceed $1 million.	Loss or significant deterioration of nationally or locally important fisheries habitat (including water quality), wildlife habitat, rare and/or endangered species, unique landscapes or sites of cultural significance. Feasibility and practicality of restoration and/or compensation is high.
MODERATE Level 2	Low potential for multiple loss of life. Inundation area is typically underdeveloped except for minor roads, temporarily inhabited or non-residential farms and rural activities. There must be a reliable element of natural warning if larger development exists. Estimated loss of life between 10 and 100.	Low economic losses to limited infrastructure, public and commercial activities. Estimated direct and indirect (interruption of service) costs could exceed $100,000.	Loss or significant deterioration of regionally important fisheries habitat (including water quality), wildlife habitat, rare and/or endangered species, unique landscapes or sites of cultural significance. Likelihood of recovery or feasibility of restoration or and/or compensation is high.
LOW Level 1	Minimal potential for any loss of life. The inundation area is typically undeveloped. Estimated loss of life between 1 and 10.	Minimal economic losses typically limited to owners' property. Virtually no potential for future development of other land uses within the foreseeable future.	No significant loss or deterioration of fisheries habitat, wildlife habitat, rare and/or endangered species, unique landscapes or sites of cultural significance.
REMOTE Level 0	No potential for any loss of life. The inundation area is typically undeveloped.	Minimal economic losses typically limited to owners' property. Virtually no potential for future development of other land uses within the foreseeable future.	No significant loss or deterioration of fisheries habitat, wildlife habitat, rare and/or endangered species, unique landscapes or sites of cultural significance.

A detail on PCC and risk assessment is provided under the EN Reference Dam Safety Guidelines.

2.2. KEY DAM SAFETY MANAGEMENT PRINCIPLES

Basic dam safety management principles that need to be considered during design, operation, maintenance, surveillance, emergency preparedness, and safety review and assessment process are as follows:

- The public and the environment shall be protected from the effects of dam failure, as well as release of any or all of the retained fluids behind a dam, such that the risks are kept as low as reasonably practicable
- The standard of care to be exercised in the management of dam safety shall be commensurate with the consequences of dam failure.
- Due diligence shall be exercised at all stages of a dam's life cycle.
- A dam safety management system, incorporating policies, responsibilities, plans and procedures, documentation, training, and review and correction of deficiencies and nonconformance, shall be in place.
- Requirements for the safe operation, maintenance, and surveillance of the dam shall be developed and documented with sufficient information in accordance with the impacts of operation and the consequences of dam failure.
- Documented operating procedures for the dam and flow control equipment under normal, unusual, and emergency conditions shall be followed.
- Documented maintenance procedures shall be followed to ensure that the dam remains in a safe and operational condition.
- Documented surveillance procedures shall be followed to provide early identification and to allow for timely mitigation of conditions that might affect dam safety.
- Flow control equipment shall be tested and be capable of operating as required.
- An effective emergency management process shall be in place for the dam.
- The emergency management process shall include emergency response procedures to guide the dam operator and site staff through the process of responding to an emergency at a dam.
- The emergency management process shall ensure that effective emergency preparedness procedures are in place for use by external response agencies with responsibilities for public safety within the floodplain.
- The emergency management process shall ensure that adequate staff training, plan testing, and plan updating are carried out.
- A safety review of the dam ("Dam Safety Review") shall be carried out periodically.
- A qualified registered professional engineer shall be responsible for the technical content, findings, and recommendations of the Dam Safety Review and report.
- The dam system and components under analysis shall be defined.
- Hazards external and internal to the dam shall be defined.
- Failure modes, sequences, and combinations shall be identified for the dam.
- The dam shall safely retain the reservoir and any stored solids, and it shall pass flows as required for all applicable loading conditions.

For more information on these principles, you may refer to the ENTRO Reference Dam Safety Guidelines.

3. TYPES OF SMALL DAMS

3.1. INTRODUCTION

Small dams may be grouped into different categories, such as:

- Purpose
- Hydraulic Design
- Type of Materials
- Structural behavior
- Size

3.2. BASED ON PURPOSE

1. Storage Dam Or Impounding Dam
2. Detention Dam
3. Diversion Dam
4. Coffer Dam
5. Debris Dam

1. Storage Dam Or Impounding Dam

It is constructed to create a reservoir to store water during periods when there is huge flow in the river (in excess of demand) for utilization later during periods of low flow (demand exceeds flow in the river). Water stored in the reservoir is used for irrigation, power generation, water supply etc. By suitable operation, it can also serve as a detention dam.

2. Detention Dam

It is primarily constructed to temporarily detain all or part of the flood water in a river and to gradually release the stored water later at controlled rates so that the entire region on the downstream side of the dam is protected from possible damage due to floods. It may also be used as a storage dam.

3. Diversion Dam

It is constructed to divert part of or all the water from a river into a conduit or a channel. For diverting water from a river into an irrigation canal, mostly a diversion weir is constructed across the river.

4. Coffer Dam

It is a temporary dam constructed to exclude water from a specific area. It is constructed on the u/s side of the site where a dam is to be constructed so that the site is dry. In this case, it behaves like a diversion dam.

5. Debris Dam

It is constructed to catch and retain debris flowing in a river.

3.3. BASED ON HYDRAULIC DESIGN

1. Overflow Dam

It is constructed with a crest to permit overflow of surplus water that cannot be retained in the reservoir. Generally dams are not designed as overflow dams for its entire length. Diversion weirs of small height may be designed to permit overflow over its entire length.

2. Non-Overflow Dam

It is constructed such that water is not allowed to overflow over its crest. In most cases, dams are so designed that part of its length is designed as an overflow dam (this part is called the spillway) while the rest of its length is designed as a non-overflow dam. In some cases, these two sections are not combined.

3.4. BASED ON STRUCTURAL BEHAVIOR

- Gravity Dam
- Arch Dam
- Buttress Dam
- Embankment Dam

1. Gravity Dam

It is a masonry or concrete dam which resists the forces acting on it by its own weight. Its c/s is approximately triangular in shape.

- Straight gravity dam – A gravity dam that is straight in plan.
- Curved gravity plan – A gravity dam that is curved in plan.
- Curved gravity dam (Arch gravity dam) – It resists the forces acting on it by combined gravity action (its own weight) and arch action.
- Solid gravity dam – Its body consists of a solid mass of masonry or concrete
- Hollow gravity dam – It has hollow spaces within its body.
- Most gravity dams are straight solid gravity dams.

Concrete Gravity Dams

- Weight holds dam in place
- Lots of concrete (expensive)
- These dams are heavy and massive wall-like structures of concrete in which the whole weight acts vertically downwards
- As the entire load is transmitted on the small area of foundation, such dams are constructed where rocks are competent and stable.

2. Arch Dam

It is a curved masonry or concrete dam, convex upstream, which resists the forces acting on it by arch action.

- Arch shape gives strength
- Less material (cheaper)
- Narrow sites

- Need strong abutments
- These type of dams are concrete or masonry dams which are curved or convex upstream in plan
- This shape helps to transmit the major part of the water load to the abutments
- Arch dams are built across narrow, deep river gorges, but now in recent years they have been considered even for little wider valleys.
- Good for narrow, rocky locations.
- They are curved and the natural shape of the arch holds back the water in the reservoir.
- Arch dams are thin and require less material than any other type of dam.
- It is mostly used for large and medium dam.

3. Buttress Dam

It consists of water retaining sloping membrane or deck on the upstream which is supported by a series of buttresses. These buttresses are in the form of equally spaced triangular masonry or reinforced concrete walls or counterforts. The sloping membrane is usually a reinforced concrete slab. In some cases, the u/s slab is replaced by multiple arches supported on buttresses (multiple arch buttress dam) or by flaring the u/s edge of the buttresses to span the distance between the buttresses (bulkhead buttress dam or massive head buttress dam). In general, the structural behavior of a buttress dam is similar to that of a gravity dam.
Buttress Dams

- Face is held up by a series of supports
- Flat or curved face
- Buttress Dam – Is a gravity dam reinforced by structural supports
- Buttress – a support that transmits a force from a roof or wall to another supporting structure
- This type of structure can be considered even if the foundation rocks are little weaker.

4. Embankment Dam

It is a non-rigid dam which resists the forces acting on it by its shear strength and to some extent also by its own weight (gravity). Its structural behaviour is in many ways different from that of a gravity dam.

- Earth or rock
- Weight resists flow of water

Earth Dams

- They are trapezoidal in shape.
- Earth dams are constructed where the foundation or the underlying material or rocks are weak to support the masonry dam or where the suitable competent rocks are at greater depth.
- Earthen dams are relatively smaller in height and broad at the base.
- They are mainly built with clay, sand and gravel, hence they are also known as Earth fill dam or Rock fill dam.

3.5. BASED ON CONSTRUCTION MATERIAL

Homogeneous Earthfill Dams

Earthfill dams are the most common type of dam, principally because their construction involves the use of materials from the required excavations and the use of locally available natural materials requiring a minimum of processing. Homogeneous dams are mostly adopted where cohesive soils are found abundant. They are also preferable for gravelly cohesive soils (residual soils) as well as loess. However, such dams have a flatter slope, consequently, a greater volume of embankment.

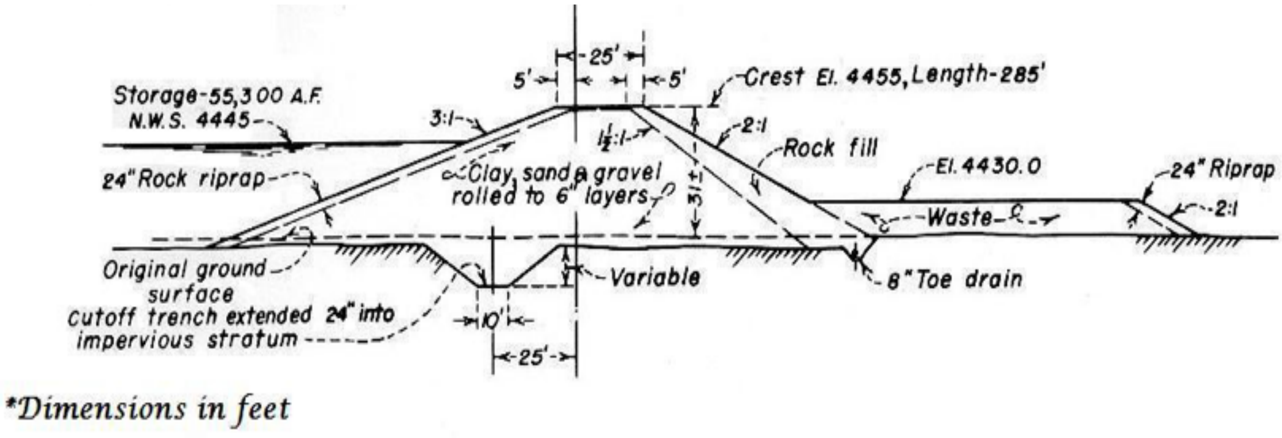


FIGURE 3.5 1: CRANE PRAIRIE DAM (1939 - 40), USA.

Zoned Earthfill Dams

Zonal embankment small dams are usually constructed with central clayey core stabilized by shell material embanked from coarse river gravels. These two basic materials are separated by two sandy-gravel filter layers.

Central Clay Core Dams

Where there are abundant pervious materials for dam shells but little amount of cohesive materials, central core dams are more desirable. The central clay core may be thick or thin, depending on the construction condition and on the availability of impervious soil. The shell zones upstream and downstream of the central core may be filled with highly pervious materials like sand, gravelly soil, weathering soils, etc.

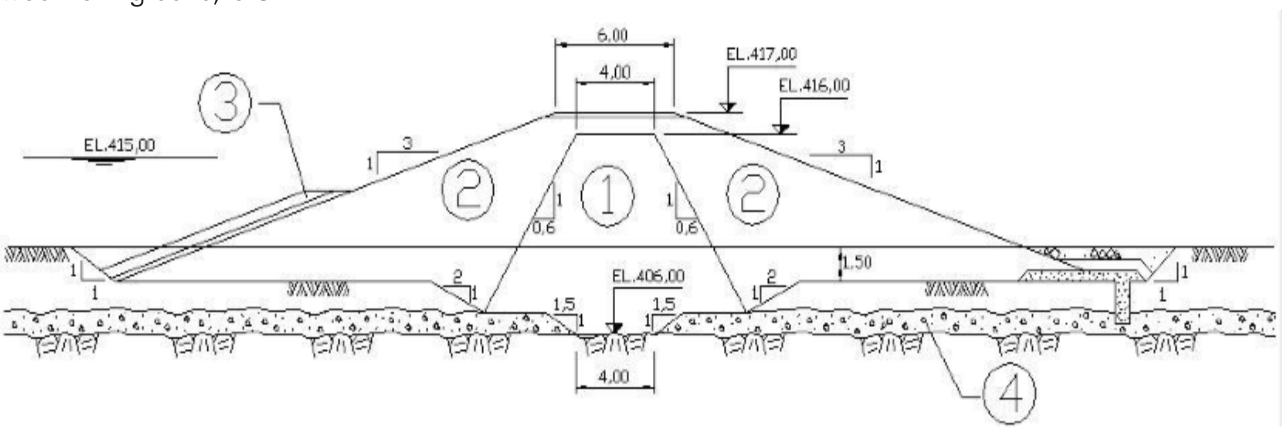
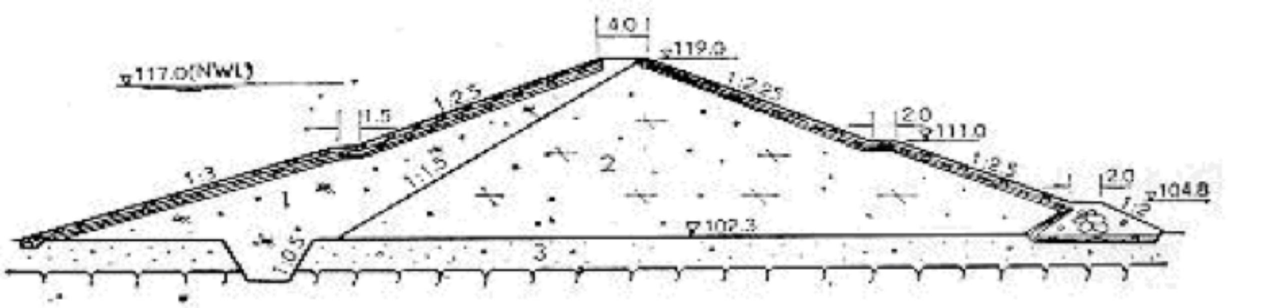


FIGURE 3.5 2: ITIQUIRA EMBANKMENT ZONED DAM, BRAZIL.
1- Clay Core, 2- Sandy clay, 3- Rip-rap, 4- Alluvial gravel and sand

Sloping Core Dams

In order to provide construction flexibility between the core and the downstream part, it is advisable to adopt a sloping core dam. The sloping core may be readily extended toward the reservoir to form the impervious blanket, thereby increasing the seepage path and decreasing the leakage. The soil materials for sloping core are the same as those for central core. However, the embankment materials downstream of the sloping core besides having high shearing strength and high perviousness, should be of small compressibility, and should not cause cracking in the core due to differential settlement. Special care need to be taken with the possibility of desiccation of the upstream core and its consequents cracks.

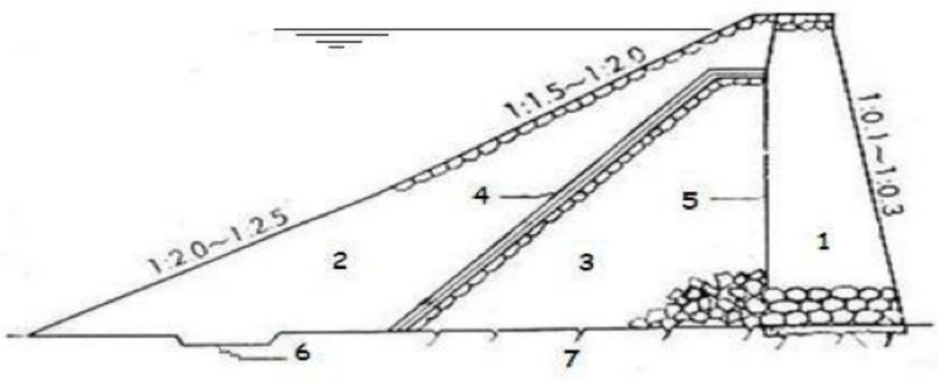


1 – Sloping clay core 2 – Sandy loam

FIGURE 3.5 3: SHANGLIYU EARTH DAM WITH THICK SLOPING CORE, CHINA

Zhaogushe Dams

Zhaogushe type overflow earth-rock dams initiated in China in 1958 and since then they have been widely popularized. This type of dams is suitable in rocky mountainous regions where the river valley is narrow with insufficient soil materials and the deposit on the river bed is shallow and where a chute spillway would involve a large volume of excavation. This kind of overflow dam has developed rapidly and its maximum height has reached 32.5 m. These dams can safely withstand small overflow discharges under a head of about 1.0 m. The Zhaogushe dam is a kind of earth-rock embankment, but mostly built of rock materials. The dry-laid masonry constitutes the key part in ensuring safety, the impervious. sloping core guarantees water impoundment of the dam, the rockfill improves the stability of the masonry, and the filter layers ensure effective and long term functioning of the clay core.



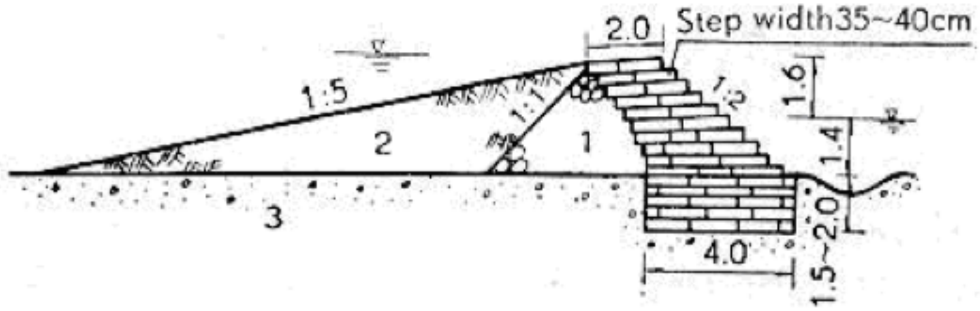
1 – Dry-laid , 2 – Sloping clay core, 3 – Rockfill,
4 – Filters, 5 – Rough interface, 6 – Cutoff, 7 – Rock foundation

FIGURE 3.5 4: CROSS SECTION OF ZHAOGUSHE TYPE OVERFLOWS DAM, CHINA

Stone Masonry Dams

Large pieces of stones have been used in the construction of dams since the construction of Sadd El Khafara dam, in Egypt, about 20km south-east Cairo, 4.600 years ago, one of the oldest dams ever registered. In this dam it is estimated that about 17 000 revetment blocks on both outer faces, each weighing 300 kg, had been carefully placed and part of them still exist until our days.

The reasons for the rapid development of stone masonry dams could be the availability of stones, particularly in the mountain areas. The volume of a masonry dam is smaller than that of an earth dam. Masonry dams can save on timber, steel and cement. Moreover, the crest of a stone masonry dam permits overflowing, thereby solution is rendered easier for river flow diversion and passage of floods during construction.



1 – Dumped , 2 – Earthfill clay, 3 – Foundation of sand and gravel
FIGURE 3.5 5: STRAW-RAINCOAT DAM, CHINA

Concrete Dams

Concrete gravity dams are suitable for sites where there is a reasonable sound rock foundation, although low structures may be founded on saprolite soils or alluvial foundations if adequate cutoffs are provided. They are well suited for use as overflow spillway crests and are often used as spillways for earthfill or rockfill dams or as overflow sections of diversion dams. Gravity dams may be either straight or curved in plan. The curved dam may offer some advantage in both cost and safety.

FIGURE 3.5 6: HLINKY CONCRETE GRAVITY DAM, HEIGHT 11M, CZECH REPUBLIC

Roller Compacted Concrete DAMS (RCC)

The term "roller compacted concrete" describes concrete used in the construction process, which combines economical and rapid placing techniques of embankment material with those excellent mechanical properties of concrete, such as strength and durability. This technique is best suited to multi-layer constructions with a high ratio of surface to thickness, that is to say, pavements and dams.

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Interest in this type of dam has increased for several reasons, the most prominent being economics and construction speed. In many countries the costs of conventional concrete dams have increased significantly faster than corresponding costs for embankment dams. But the fact that concrete is such a good and long-lasting construction material, has stimulated designers to seek new ways of using it in dam construction. They succeeded with the adoption of RCC technology.

t is common knowledge that RCC is a construction technique, and not a design concept. However, when discussing projects that may use the RCC technique, two basic points are usually taken into account:

- Treatment and characteristics of construction joints between lifts; and
- Watertightness and durability, of the upstream face type, seepage and drainage factors and control.

FIGURE 3.5 7: TRAIRAS RCC CONCRETE DAM – BRAZIL

Concrete Buttress Dams

Buttress dams are comprised of a flat deck and multiple arch structures or transversal concrete buttresses with only 7 m in height. They require about 60 percent less concrete than solid gravity dams, but the increased formwork and reinforcement steel require usually offset the savings in concrete. The cost of this type of construction is usually not competitive with that of other types of dams when labor costs are high.

FIGURE 3.5 8: BARRA BUTTRESS CONCRETE DAM, BRAZIL.

Gabion Dams

The use of gabion for the construction of small dams is more recent. The gabion dam as the gabion retaining structures employ generally wire mesh baskets filled with rock at the downstream part of the dam to form a flexible, permeable, monolithic structure similar to a gabion retaining wall. At the upstream part is compacted an earthfill as the impervious element.
using it in dam construction. They succeeded with the adoption of RCC technology.

FIGURE 3.5 9: GABION DAM 12 M HIGH - BRAZIL

Inflatable Dam Type

In recent years, inflatable dam technology has significantly advanced and has become an economical solution for installing new and replacing old spillway crest gates up to 5.4 m. The hydraulic and environmental advantages of inflatable dams have also made them an attractive solution for constructing new and replacing old run of the river low-head gravity dams. Inflatable dam types have been installed in a wide variety of conditions, configurations and environments. The first inflatable dam system known as the "Fabridam" was introduced in the mid 1950s.

Some Rubber dams consist of a sealed rubber tube fabricated from a heavy – duty, nylon – reinforced rubber attached to a reinforced concrete foundation with metal clamping lines. The rubber body is inflated with air or water to impound water. The reinforced rubber material used for the dam body is sometimes similar to that used for conveyor belts, tires for heavy construction equipment, and tires for aircraft.

FIGURE 3.5 10: INFLATABLE SMALL DAM, MLADA BOLESLAV, CZECH REPUBLIC

4. SAFETY OF SMALL DAM

4.1. INTRODUCTION

This chapter discusses the conditions affecting the safety of small dams. In comparison with large dams, the same overall dam safety principles apply to small dams. However, there are differences between large and small dams which in certain cases may allow adjustments to design and construction practices:

- Lower water, sedimentation and gravitational loads are applicable and therefore lower stresses and strains in the structures and on the foundation may be taken into consideration in the design;
- Economics play a very important role because decisions are taken with financial constraints as background. This could lead to design criteria and construction methods with higher risk, more maintenance problems and in some cases to catastrophic failure. The funding of small dams should be sufficient to allow standards for design and construction to be consistent with the potential consequences classification (PCC).

- The time required by engineers for small dam design and construction monitoring is as time consuming as for large dam activities, although due to cost constraints significantly less time is approved for technical services by clients of small dams. This leads to reductions in investigation, design and site monitoring costs, which leads to substandard designs and construction quality achieved during construction. Proper designs must therefore be done and construction monitoring ensuring appropriate quality control consistent with the potential consequences classification.
- Foundation and construction materials investigations are normally limited due to financial constraints, with normally higher cost to the structure due to unknown conditions. Proper investigations tailored to the specific foundation and construction materials consistent with the potential consequences classification for the required dams are therefore necessary.
- Flood hydrological investigations are sometimes limited with catastrophic consequences if the dams are overtopped. Again, these should be consistent with the potential consequences classification.
- It is not acceptable to automatically design for higher risk of failure for floods, e.g. overtopping with flood occurrences associated with lower recurrence intervals. Dambreak floods from smaller dams may often be lower in size due to the lower water head and water volume in the dams so that the potential loss of life and damages may also be lower depending on the actual situation . Due to this, lower design standards may sometimes be applied for many aspects e.g. spillway sizing. Despite this fact, the number of victims is generally significant as a consequence of the great number of small dams and the large number of failures during extreme floods Flood discharge capacity should be consistent with the potential consequences classification.
- One dam failing in a cascade of dams in a river system may cause the other downstream dams to fail
- Consequently, when designing and constructing a number of small dams, whether in a cascade or in separate river basins, the potential consequences classification to be used for design and construction criteria should consider the potential consequences of the entire portfolio of small dams
- Normal seepage rates may also have an impact on the yield of small dams and low water depth makes small reservoirs more sensitive to evaporation. Higher care must therefore be taken to seepage control (reduction) measures.
- Often inexperienced small contractors are used to construct small dams. These contractors have limited resources and sometimes use “farm dam” practices to construct with a low quality structure and poor performance. Appropriate construction quality control processes are required.
- One dam failing in a cascade of dams in a river system may cause the other downstream dams to fail.

4.2. CONDITIONS AFFECTING THE SAFETY

Examples of conditions affecting the safety of existing small dams as well as reasons for dam failure are summarized in Table 4.2.1.

TABLE 4.2 1: Examples of conditions affecting the safety of existing embankment dams

Condition	Effect on safety
Inadequate spillway capacity (dam flood handling capacity) caused by low capacity spillway, heightening of the spillway crest by owners not aware of the hydrologic risks, too little freeboard or obstructions in spillway e.g. tree growth.	Embankment can be overtopped and breached.
Backward erosion of erodible by-wash spillways.	Erosion channel can extend into dam reservoir with consequential dam failure through the spillway.
Uneven crest of embankment (it may be constructed uneven or settle). Cattle footpath provides low point on crest of embankment.	Embankment can be overtopped and breached due to concentration of water in the low crest areas.
Wet areas or seepage through embankment or foundation, as identified on downstream face or in area downstream of embankment.	Saturated conditions in the downstream shell can cause slope failure. Piping failure of embankment can also occur.
Obstruction of the internal drainage system.	Phreatic surface may be raised by blocked drains.
No internal drains and filters provided – especially for embankments constructed with dispersive clays.	Piping failure can occur, most of them during the first reservoir filling
High seepage water along or into outlet pipe – conduit pipes may crack due to settlement or the pipe materials may degrade e.g. wooden pipes exposed to air and water.	Piping failure of embankment along bottom outlet can occur.
Piping (or internal erosion) at the interface of an embankment and retaining wall for the spillway control section and/or "start" of the spillway return channel.	Embankment can breach at that point
Embankment cracks or slope failures occur	Embankment breaching failure can occur.
Tree growth on embankments – roots damaging the embankment especially when dead.	Piping failure of embankment can occur.
Burrowing animals excavate tunnels in embankment.	Embankment slope failure or seepage failure may occur.
Compaction of earthfill not meeting standards	Uncompacted embankments experience settlement, seepage and slope failure problems.
Damage to upstream slope of embankment due to wave action.	This may cause upstream slope instability and erosion of crest with wave water overtopping and breaching of embankment.
Slope protection like grass not effective – erosion of slopes of embankment by storm water	Slope failures and breaching may occur.
Monitoring Instruments not working	Behavior of embankment or spillway cannot be monitored.

Best practices to overcome shortcomings causing high risk of failure or claims are described in the following chapters on design, construction, operation and maintenance, surveillance and monitoring,

4.3. CAUSES OF DAM FAILURES

4.3.1 General

There are always design measures that a dam owner can take to prevent dam failure in response to earthquake, extreme storm activity and failure of upstream dams. However, normal margins of safety should be capable of accommodating earthquakes of a magnitude that is appropriate for the region, based on geological information.

Although statistics are patchy concerning small dams, the overall failure for dams less than thirty meters high can be estimated at nearly 2%; many failures caused no casualties but several dozen have been disastrous and the total number of victims has been ten times higher than for failures of very high dams. The risk has varied with time and construction methods and it is possible to evaluate that it is higher for small dams, as a consequence of the poor care usually taken during the design, construction and maintenance of such dams.

A recent research about the failure of dams in Brazil showed the following results about the failure of small dams: overtopping 65%, piping 12%, slope failure 12%, all others 12%. The great number of dam failure during severe flood periods, clearly shows that overtopping is the main cause of failure, with piping and internal erosion appearing in a second place. Overtopping occurs when the actual flow over a spillway exceeds its design capacity. It may therefore be regarded as a "natural hazard", resulting from extreme low probability weather conditions, but overtopping may also be regarded as a human error in case of underestimation of the design flood. The other main types of failure listed may be regarded as related to human error. Of these human error related failures, piping and slope stability are more related to improper construction and operation control. Foundation failures are more related to errors of judgments in design and geological assessment.

4.3.2 Overtopping Caused by Flood

Undersizing of spillways usually causes many failures with small dams. In industrialized countries, the corresponding rate of failures has been very low for the dams built after 1930 (less than 0.1%) and since 30 years the yearly rate is in the range of 10-5. This is true for large reservoirs but also for smaller ones of which design flood was often in the range of 10-3.

In some cases, the overtopping causes a general sliding of the downstream part of the dam embankment then a rather large initial breach. But in most cases, the initial breach resulting from the toe erosion is relatively narrow, as in case of piping, and may or not widen according to cross section and materials characteristics. For a same dam and same initial plan of breach, breach flow is higher for overtopping than for piping, due to higher water level and arriving flow.

Over 3% of dam failures occurred due to the failure of a dam further upstream. This risk should not be overlooked in the coming years for both existing and new dams. Nor should partial obstruction of the spillways by vegetation or floating debris be forgotten.

4.3.3 Internal Erosion

Internal erosion and piping in embankments and their foundations is the second cause of failures and accidents at embankment dams. For new dams, the potential for internal erosion and piping can be controlled by good design and construction of the dam and provision of filters to intercept seepage through the embankment and the foundations. However many existing dams were not provided with filters and are susceptible to internal erosion failure, with a likelihood increasing with ageing. The four types of internal erosion are:

- erosion at concentrated leaks,
- erosion at contacts between coarse and fine materials,
- backward erosion in which erosion at the toe of structures from erosion pipes under them,
- internal instability, which may occur in gap-graded materials where the soil fabric is such that small particles can be eroded out between larger particles.

In any case, internal erosion is due to non controlled seepage through the embankment, through the foundation or at a transition between the embankment (or foundation) and a rigid structure (spillway, bottom outlet, etc).

4.3.4 Slope Instability

Another reason for dam failure is slope instability, but it happens more seldom than the two previous causes.

Instability of downstream slope often occurs mainly when the pore water pressure inside the embankment is not controlled with an adequate drainage system. High pore pressures in the downstream part of the embankment may lead to circular rupture of this face, especially in case of a steep slope.

Instability of upstream slope often occurs after a rapid drawdown of the reservoir, when permeability of the material is too low to allow dissipation of pore pressure.

5. LEGISLATION & DECOMMISSIONING

5.1 INTRODUCTION

As a general rule, protection of persons and property is a responsibility of the Government, who must legislate and enforce the rules through administrative bodies to provide for the safety and security of the people, property and environment. That is why dam design, construction, rehabilitation, enlargement, alteration, operation, monitoring, maintenance, repair, breach, abandonment and removal must rely on a legal frame that establishes rights, responsibilities and duties of the parties involved. Therefore, it is paramount that Governments clearly define the responsibilities of the Owner and establish a regulatory body with dam safety as its primary concern, completely independent from entities currently owning, operating or using dams.

5.2 THE OWNER

The responsibility for constructing, operating and maintaining a safe dam rests with the Owner. Dam owners should be aware of their legal responsibilities for the continued safe operation of their dams and the structure's maintenance and inspection requirements.

Negligence by owners in fulfilling these responsibilities can lead to the creation of extremely hazardous conditions which may lead to a potential dam failure and thereby threaten downstream residents and properties. Dams cannot be considered as a part of the natural landscape, but rather as artificial structures which require ongoing inspection, and maintenance (and in the case of high and significant hazard dams carefully developed emergency action plans). Maintenance is an ongoing process that not only involves such routine items as mowing grass and clearing spillways, conduits, channels, trashracks, etc., but also requires regular inspections of the structure and its various components.

Legally the Owner is any person who own, control, operate, maintain, manage or propose to construct, rehabilitate, enlarge, repair, alter, remove or abandon a dam or reservoir. Permits shall be obtained by Owners to construct, repair or alter dams, dikes or similar structures and that Owners demonstrate that existing dams are being properly maintained and meet modern safety standards, and include specific responsibilities a dam Owner must implement to ensure ongoing safety. Measures include:

- Requirements for record keeping (design plant, permanent reference points, embedded instruments location and readings etc);
- Inspection and maintenance plans;
- Scheduled inspections by a professional engineer working for the owner;
- Scheduled safety reassessments to confirm that a dam meets up to date safety criteria;
- Emergency action plans to all dams with a high hazard rating;
- Disclosing the presence of a dam when property is transferred;
- Periodical review of a dam's hazard classification;
- Demonstration of financial assurance, collectible if the state is forced to conduct necessary remedial work at a dam; and
- Annual certification that the dam's inspection and maintenance plan, emergency action plan, and other requirements are being met, for all small dams with a high hazard rating.

In order to protect life and property, Owners of high potential hazard small dams should develop and periodically test and update an emergency action plan EAP that shall be implemented in the event of an emergency involving their dams.

Owners of dams and reservoirs have the primary responsibility for determining when an emergency involving a dam / reservoir exists. When facing an emergency, the Owner shall implement the EAP, notify any persons who may be endangered if the dam should fail, notify emergency management organizations and take additional actions necessary to safeguard life, health and property.

5.3 DECOMMISSIONING

Decommissioning is defined as the full or partial removal of an existing dam and its associated facilities or significant changes to the operations thereof. There are many reasons for removing a dam - obsolescence, environmental concerns, economics, safety criteria, risk reduction, and operation and maintenance costs.

A small dam could be decommissioned when:

a) It no longer meets required safety standard and it is economically not viable to incur in the expenses to make it safe;
b) It can no longer fulfill its functions and it is economically not justifiable to repair it.

Other reasons could be the water is no longer required or better manages via alternative sources, increased environmental flow requirements, ecosystem or catchment restoration or the potential legal or financial liabilities incurred.

It is very important to consider the costs of decommissioning (such as sediment removal, landscaping, approval process, safe demolition etc) that may, in some cases, be even higher than the costs of repairs and upgrades. Also the impact on upstream and downstream infrastructure and replacement of benefits, such as water supply and power generation. The economic aspect here must be thoroughly examined. Each situation is different and must be considered on a case by case basis. The decommissioning of small dams is usually done by full or partial removal in such manner that the remaining structure does not store water and lets it through without retention. In the case of small dams with floodgates, one can choose between raising the floodgates or removing them completely. The main objective of decommissioning a dam is to restore the natural flow of water, avoiding the concentration of sediments, and preventing or eliminating all actual and potential unsafe situations regarding people, property and the environment that could arise out an abandoned or unsafe dam. In some cases, the dam may be removed to improve upstream fish passage.

The suspended solids contained in runoff tend to settle in the quiescent reservoir waters. In industrialized areas these sediments may contain contamination such as metals, oil and grease and many other chemicals. In rural areas these sediments may contain contaminants such as pesticides and herbicides from agricultural operations. The quantity and concentration of sediments, and the rate at which they will be returned to the fluvial system are also a major concern. Sediment disposal is a significant issue to be resolved when considering dam decommissioning. The sediments regularly concentrated in the bottom of the reservoir could require adequate treatment or removal to avoid environmental damage downstream. Removal of the saturated sediments from the reservoir and disposing of it on land is expensive. Therefore, allowing the sediments to be flushed downstream as a dam is removed has been the most common practice to-date in the USA. This method has a detrimental effect on the water quality; however, the streams restore themselves after a period of time after the flushing has been allowed. This method has been used on small dams and has also been proposed for large dam decommissioning. The decommissioning can be proposed by the owner or by the regulator and should be approved by the responsible Authorities for dam safety. After the approval, a project plan must be prepared in accordance to the regulation. The owner of the small dam should draft a decommissioning project plan in order to avoid any situation of risk when the dam is abandoned upon completion of the concession/authorization period or due to lack of maintenance and conservation. During the works Decommissioning completed

a) Official documentary proceedings shall be filed;
b) Draft of a decommissioning plan regarding the dam and its reservoir, defining all the required refurbishment works to be executed on site and its surroundings, as well as its area of impact;
c) The decommissioning plan should include the necessary measures to be taken to guarantee the safety of the area, especially the capacity of outflow of water as well as to guarantee the stability of permanent structures without causing any adverse impact upstream and downstream and without affecting the fixed structures which might be in service;

d) The project shall require the processing of an Environmental Impact Evaluation clearance which varies from case to case;
e) This project should be preapproved by the competent Authority for safety of dams and reservoirs. Once the decommissioning project is executed, the competent authority for safety of dams and reservoirs will carry out an inspection of the site and its area of impact before issuing the concerned report which will be required for the final approval of the decommissioning procedure.

5.4 CONCLUSION

Formal regulations by Government play an essential role in promoting increased physical and environmental safety, responding to a social need and requirement. The regulation shall consider the four main phases regarding a dam: planning-approval, construction, operation and eventual closure. Governments must focus on supervision and monitoring of dams, small and large, since potential losses in lives and property damage caused by flooding are increasing as development progresses bottom land alongside rivers. Considering the large number of small dams in each country it is essential to establish a dam risk prioritization criteria, and concentrate attention and efforts on the high hazard small dams, in a first stage. It is paramount that the supervision and monitoring activities are conducted with the necessary skill, according to the size and potential risk of the dam, in a context of clear assignment of roles and duties of the Owners, Concession Holders and Supervising Authority.

6. ENVIRONMENTAL AND SOCIAL ASPECTS FOR DAM SAFETY

ICOLD findings about failure of dams indicate that "the proportion of dams failing varies little with the height of the dam and so most failures involve small dams."[2]

Safety guidelines for small dams that integrates environment and social issues will contribute to long and useful service life of dams in the Eastern Nile region. Environment and social considerations in small dam safety management are very important both for the safety of the dam and for communities and the environment upstream and downstream of the dam.

The Eastern Nile Basin covers 1.7 million km2 and has a population of 149.5 million people in three countries and includes four Sub-basins: the Baro-Sobat-White Nile, Abbay-Blue Nile, Tekeze-Atbara and the Main Nile from Khartoum to the Aswan High Dam. The Eastern Nile region has a complex and varied environment and social setting. Altitudes vary from 0 to over 4500 meters above sea level. Similarly rainfall and temperature vary greatly across the region. It is thus important to consider the environment and social factors related to small dams in the context of these varied local settings.

6.1 ENVIRONMENTAL CONSIDERATIONS[3]

The major environmental issues related to small dams include floating debris and vegetation as a result of the barrier created by the dam itself, blockage of the passage of sediments, retention of floods and flooding of the reservoir area, water quality (effects on water temperature, effects on oxygen content, eutrophication), effects on aquatic flora and fauna, landslides and degradation of river channel below the dam, erosion(backward erosion), and animal burrows.

Floating debris and vegetation

Floating debris and vegetation or large trunks of tress may obstruct gates and outlets that may endanger the safety of dams due to blockage of gated spillways.

"Structural instability can occur due to falling/decaying tree/woody vegetation and root system growth. Large, seemingly stable and innocuous trees can easily be blown over or uprooted in a storm/flood and cause a large hole left by the root system. This in turn can shorten the seepage path and initiate piping, or a breach in the dam."

This may be countered through a proper environmental assessment studies which usually address issues of vegetation clearing, soil and land stability studies.

Landslides, sedimentation and degradation of river channel below the dam, erosion (backward erosion), and animal burrows

A reservoir may cause a landslide on the banks, which may partially fill in the reservoir or cause overtopping of the dam. Geologic and soil mechanics investigation should precede reservoir filling and zones of probable sliding should be stabilized, drained or flattened.
The creation of a reservoir changes the hydraulic and sediment transport characteristics of the river, causing increased potential sedimentation within the storage and depriving the river downstream of material. Sedimentation is an important sustainability issue for some reservoirs and may reduce the long-term viability of developments. Reduction in the sediment load to the river downstream can change geomorphic processes (eg. erosion and river form modification).

Development proposals need to be considered within the context of existing catchment activities, especially those contributing to sediment inflow to the storage. Reducing reservoir sedimentation through cooperation with local communities and regulatory authorities in improving catchment management practices is an option. Specific actions, such as terracing or reforestation, may need to be considered. In some cases sediment by-passes, flushing systems or dredging should be investigated. Operational or physical mitigation measures to reduce erosion of downstream should be considered for both proposed and existing developments and appropriate objectives set.

Retention of floods and flooding of the reservoir area,

Small and medium floods provide access to spawning grounds and renew the water in them while preventing the river banks becoming overgrown with trees.

In order to maintain small and retention floods water can be released through the dam. Water release should take in to consideration ecological requirements downstream.

The reservoir created by the dam may affect animals that use to reside in the riverine ecosystem. This may be addressed by salvage operations by moving out animals to other locations. The creation of a new reservoir can establish conditions conducive to a large increase in bird and other species.

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[2] The World Commission on Dams, 2000. Dams and Development: A New Framework for Decision-Making(.p63)

[3] Adopted and summarized from "The International Hydropower Association (IHA) has issued "Sustainability Guidelines" in February 2004 entitled "Hydropower- Environmental Aspects of Sustainability" andICOLD Bulletin 86 (1992) Dams and Environment - Socio-Economic Impacts,Bulletin 35 (1980) Dams and the Environment

Water Quality (water temperature, effects on oxygen content, eutrophication)

Changes in water quality are likely to occur within and downstream of the development as a result of impoundment. The residence time of water within a reservoir is a major influence on the scale of these changes, along with bathymetry, climate and catchment activities. Major issues include reduced oxygenation, temperature, stratification potential, pollutant inflow, propensity for disease proliferation, nutrient capture, algal bloom potential and the release of toxicants from inundated sediments. The maximum dissolved oxygen capacity is 13mg/l for cold water and 7.5mg/l for warm water. When the river s heavily loaded with organic material, the oxygen content will fall and deplete. One of the factors for oxygen depletion is the amount of decomposition products in the reservoir. Remedies consist of purifying the inflow, injecting oxygen and building intercepting ditches.

Adequate data collection and an EIA process that identifies potential problems prior to dam design are critical. Design and operational systems that minimise as much as possible the negative impacts within the storage and downstream; examples include multilevel off-takes, air injection facilities, aerating turbines, and de-stratification capability. While removal of vegetation from proposed impoundments is expensive, the potential benefits for water quality means that at least some removal should be considered. Working with local communities and regulatory authorities in improving catchment management practices can have significant water quality benefits for hydro reservoirs.

Effects on aquatic flora and fauna,

In some regions a significant long-term issue with reservoirs, irrespective of their use, is the introduction of exotic or native pest species. The change in environment caused by storage creation often results in advantageous colonization by species that are suited to the new conditions, and these are likely to result in additional biological impacts. In some instances, proliferation may interfere with power generation (eg. clogging of intake structures) or downstream water use through changes in the quality of discharge water (eg algal bloom toxins, deoxygenated water).
Identifying the risk of infestation prior to development should also help identify potential options for future management or mitigation. Shorter residence time of water is one viable mechanism for reducing risk. Downstream water uses must also be considered when examining potential options for control.

6.2 SOCIAL CONSIDERATIONS

The major social issues include health effects on communities, population displacement, loss of land and underground resources, loss of heritage resources, social acceptance.
Health effects on communities,

The main diseases spread due to storage reservoirs include malaria, bilharziasis and onchocerciasis. Recommended actions for preventing the spread of the above mentioned diseases include:

- Before the first filling of the reservoir, clear the area of vegetation; backfill, deepen, impound or drain any areas liable to become swampy through fluctuations in the lake level;
- During operation of the scheme, increase and lower the lake level a few centimeters weekly in the egg-laying season;
- Pesticides are the most important means of defense.

Public health and emergency response plans should be developed in conjunction with local authorities. The health benefits due to improved water supply, economic improvements and flood control should be recognized. Proper reservoir management can be highly effective in eliminating mosquito-borne illnesses such as malaria.

Population displacement and loss of resources,

These impacts include: the physical loss of homes and lands; the transition to alternative means of earning a livelihood, particularly for populations that rely heavily on local land and resources for their way of life or that have a traditional existence; disruption of established community networks and loss of cultural heritage and identity, loss of heritage resources, loss of land and over and underground resources,

Appropriate environment and social impact assessment and management plan that includes mitigation measures of negative impacts should address the above impacts. A resettlement action plan needs to be prepared and implemented with full participation of affected communities. This will improve the social acceptance of the project.

7. FEATURES OF THE DESIGN OF SMALL EMBANKMENT DAMS

7.1 INTRODUCTION

The prime purpose of small dams is storage, mainly for irrigation or water supply. A common multipurpose project involving small dams combines storage, flood control and recreational uses. Design criteria and typical features for small dams are generally different from those of high dams, because the construction methods focus upon economy. So the risks may increase and corresponding accidents may cause significant consequences. Little attention is given to downstream Consequences from floods caused by the failure of small dams; yet this is frequently the major the case during severe rainy season.

It is important to emphasize that for new dams, no high risk can be accepted for economical reasons. The knowledge is available to build safe dams.

Criteria for Design: The basic principle of design is to produce a satisfactory functional structure at a minimum total cost. Consideration must be given to maintenance requirements so that economies achieved in the initial cost of construction will not result in excessive maintenance costs. For minimum cost the dam must be designed for maximum utilization of the most economical materials available, including materials which must be excavated for its foundation and for appurtenant structures.

An earthfill dam must be safe and stable during all phases of construction and operation of the reservoir. To accomplish this, the following criteria must be met:

- The embankment must be safe against overtopping during occurrence of the inflow design flood by the provision of sufficient spillway and outlet capacity;
- The slopes of the embankment must be stable during construction and all conditions of operation, including rapid drawdown of the reservoir;

- Seepage flow through the embankment foundation and abutment must be controlled, so that no internal erosion takes place and so there is no sloughing in the area where the seepage emerges. The amount of water lost through seepage must be controlled so that is does not interfere with planned project functions;
- The embankment must be safe against overtopping by wave action;

The upstream slope must be protected against erosion by wave action, and crest and the downstream slope must be protected against erosion due to wind, rain and cattle.

Summary of Wrong Practices Followed for Small Dams

Summary of wrong practices, too often used, are summarized below:

- Foundations not investigated or investigated to a limited standard;
- Hydrology study not done. Furthermore for smaller catchment areas less data is available and empirical methods are usually used;
- Freeboard not determined;
- No camber provided;
- Soil not appropriately compacted;
- Seepage control not designed to standard;
- Wrong material sometime used (e.g. dispersive clays);
- Wrong equipment used during construction;
- Erodible by-wash spillways, cutback erosion of spillway;
- No slope protection;
- No adequate features around outlet pipes;
- Part time (and not full time) construction monitoring to confirm quality;
- No routine inspections of the dam during its life;
- No monitoring instruments installed;
- Adoption of lower cost with higher risk.

Summary of Good Practices to be Followed for Small Dams

Summary of the best practices normally used are summarized below:

- Investigation of foundations (to a minimum depth of 4-5m or the height of the dam) and excavation of the organic and others inappropriate soils at the dam foundation;
- Hydrology study, using at minimum empirical methods;
- Choose cohesive materials for embankment, which are more stable during overtopping;
- Use appropriate and durable material for slope protection;
- Adopt a standard design for filters and drains;
- Use appropriate equipment for construction (especially for compaction) and monitor construction quality;
- Embed outlet pipes into concrete;
- Access to outlet works necessary during floods and for maintenance;
- Cattle not allowed on dam slopes and abutments;
- Pipes are not allowed through or over embankments as they may cause piping through removal of fines or pipe bursts;
- Install some monitoring instruments and perform routine inspections during dam operation.

7.2 DESIGN FLOODS

Inadequate flood handling capacity not meeting design standards is normally caused by underestimating the design flood, not acknowledging the effect of the upstream approach channel on the hydraulic gradient or not ensuring that the dam was constructed with its total freeboard over the entire crest length of the dam. The hydraulic control must be correctly defined for the design flow by determining the hydraulic flow and energy lines based on the correct sectional information.

Design Floods shall be consistent with PCC. The EN Reference Dam Safety Guidelines should be used to determine Design Floods for different PCCs.

Table 7.2 1: Extreme flood and earthquake hazards for the standards based assessments

Potential Consequences Category	Annual Exceedance Probability (AEP) Hydrologic: Inflow Design Flood (IDF
Remote	1 x 10-2
Low	1 x 10-3
Moderate	PMF
High	PMF
Very High	PMF

Acronyms: AEP - Annual Exceedance Probability; PMF – Probable Maximum Flood;

It is possible to see that the safety considerations could differ from one country to another, depending on the attitude of the society as regards to acceptable risk and on the level of economic development. It is however always important to have adequate requirements related to the dam safety, in order to reduce the risk and avoid any possible dam overtopping during severe storms. Most of the small dams are embankment dams, and this type of structure does not accept overtopping without the risk of failure.

During the life of the dam, the potential consequences due to dam break flooding area can change due to developments in the area. Upgrading to a higher Potential Consequences Classification (PCC) class is then necessary.

7.3 HOMOGENOUS AND ZONED EMBANKMENTS DESIGN

Essentially, designing an earthfill dam embankment primarily involves determining the cross section that, when constructed with the available material, will fulfill its required function with adequate safety at minimum cost.

Where considerations is given to the possible loss of life, to the possibility of costly property damage, and to the waste of money incidental to the failure of a dam, ample justification is provided for conservative procedures. For small dams, where the cost of explorations and laboratory testing of the embankment material for analytical studies together with the cost of the engineering constitutes an inordinate proportion of the total cost of the structure, the practice of designing on the basis of successful structures and past experience becomes even more appropriate.

In China, for instance, as can be seen in publication , the dam slopes are to be determined mainly based on the nature of the foundation, the strength of soil material, the dry density, the dam height and the downstream drainage condition. However, for small dams (H<15 m) stability analyses are not required, and reference may then be made to the available data from existing dams. , as can be seen in Appendix I.

Considerable progress has been made in investigations and studies directed toward the development of methods that will offer a comprehensive analysis of embankment stability. Present practice in determining the required cross section of an earth dam consists largely of designing to the slopes and characteristics of existing successful dams, making analytical and experimental studies for unusual conditions, and controlling closely the selection and placement of embankment materials. Although the above practice may be criticized as being overly cautious and extravagant, no better method has been conclusively demonstrated. Where considerations is given to the possible loss of life, to the possibility of costly properly damage, and to the waste of money incidental to the failure of a constructed dam, ample justification is provided for conservative procedures, as pointed out by the Bureau of Reclamation, 1987(3rd Edition).

Most small dams (85% to 95%) involves the construction of embankment dams as a consequence of a method that is simple, straightforward and suitable for those sites where there is sufficient and proper material. Protection from seepage and slipping is provided by flattening the slopes, that is from a 2:1 to 3:1 gradient (H:V), and providing a thick covering of topsoil to carry any seepage to the toe of the dam.

7.3.1 Homogeneous Type

Embankment materials vary widely from place to place, particularly in respect to gradation and permeability. If the difference in permeability between the impervious core and the downstream batter is great, no internal drainage is required. If the variation of the permeability between the inner core and the outer zones is not sufficient, then the embankment will become saturated after prolonged storage at full supply. Consequently, the downstream slope will show seepage to a height of approximately one-third the depth of the high water level. Such saturation reduces the stability of the dam and creates maintenance problems.

Although formerly very common in the design of small dams, the completely homogenous section has been replaced by a modified homogeneous section in which small amounts of carefully placed pervious materials control the action of seepage so as to permit much steeper slopes. The effect of drainage at the downstream toe of the embankment is shown on Fig. 7.3.1.

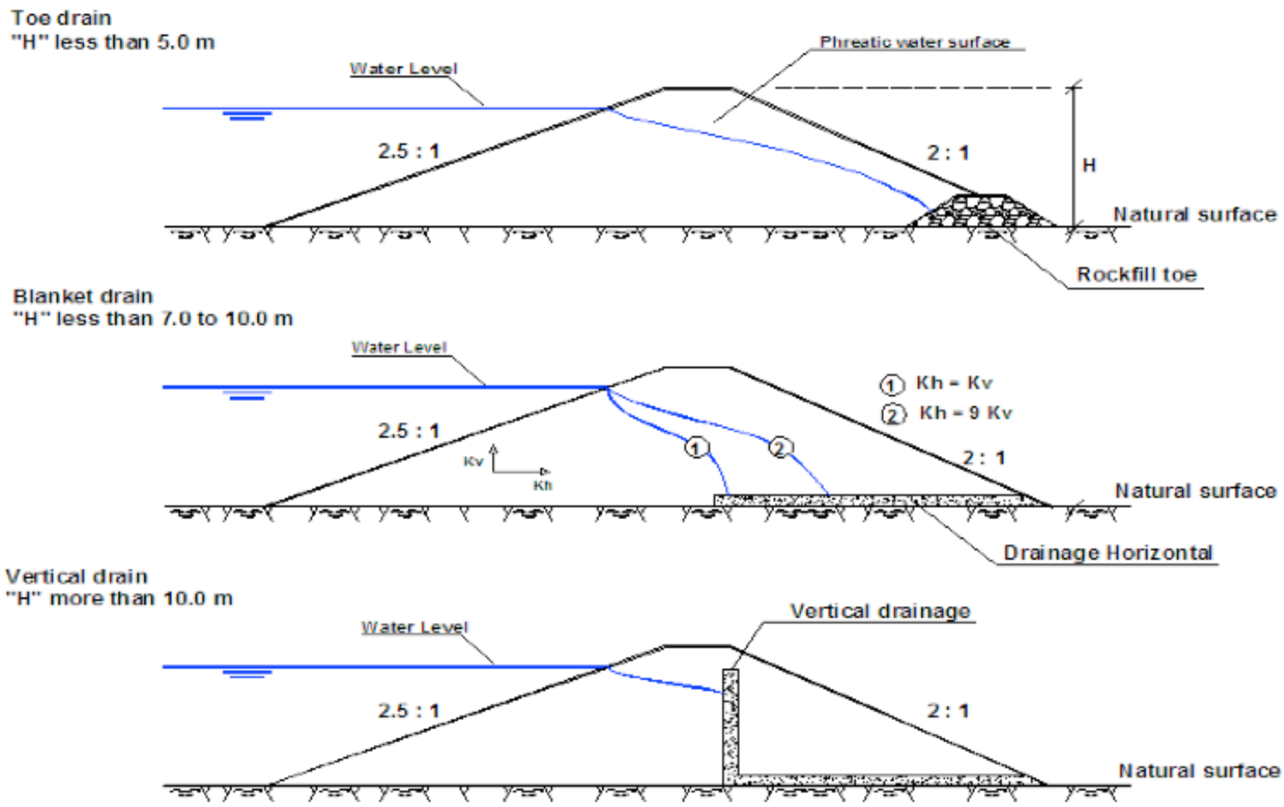


Figure 7.3 1: Homogeneous dams with different types of drainage (lewis, 2002).

If there are soils of satisfactory quality and sufficient quantity (1.5 times to twice the volume required) available on site, the homogeneous earthfill or pseudo-zoned alternative imposes itself as the most economical alternative. The pseudo-zoned dam is a variant of the homogeneous type, which consists in distributing materials in the dam body according to their grading or their humidity, without requiring filters to separate them, so there are no true zones. For example, a homogenous dam may be built with the fine materials placed upstream and the coarsest ones downstream, or with the wettest materials placed in the centre, as pointed out by the French Guidelines on Small Dams.

7.3.2 Zoned Embankment Type

The most common type of a zoned dam section is that in which a central impervious core is flanked by zones of materials considered more pervious, called shells. These pervious zones or shells enclose, support and protect the impervious core, the upstream pervious zone affords stability against rapid drawdown; and the downstream pervious zone acts as a drain to control seepage and lower the phreatic surface. In many cases, a filter between the impervious zone and downstream shell and a drainage layer beneath the downstream are necessary. In Table 5.4 are presented some examples of zoned and homogeneous small dams in Japan, illustrating their typical sections and also some rehabilitation measures under way in order to improve their safety conditions.

7.4 FREEBOARD

7.4.1 Definitions of Freeboard

Freeboard is the vertical distance between the crest of the dam and a specified still-water reservoir water surface elevation. Common definitions of freeboard are as follows:

- Normal freeboard – vertical distance from dam crest elevation to normal operating reservoir still-water surface elevation.
- Minimum freeboard – vertical distance from dam crest elevation to maximum reservoir still-water surface elevation during inflow design flood with spillway operating

7.4.2 Freeboard Design Considerations

Freeboard must be sufficient to prevent overtopping of the dam that could result from reasonable combinations of a number of factors, as illustrated on Figure 7.4.1.

- Wind-generated wave action, wind setup, and wave runup;
- Earthquake and/or landslide-generated waves and runup;
- Post-construction settlement of embankment dams and foundations;
- Provision for malfunction of spillways (especially gated structures) and outlet works, and
- Site-specific uncertainties including flood hydrology.

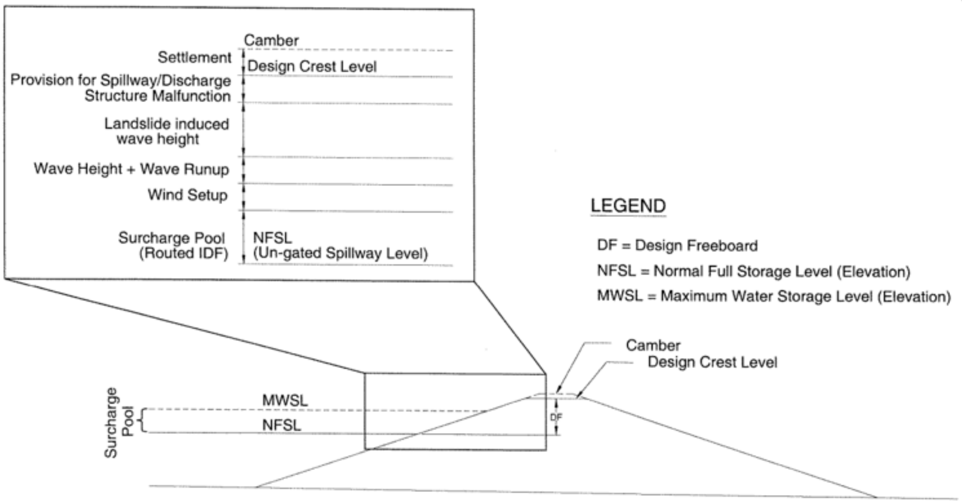


Figure 7.4 1: Design freeboard

7.4.3 Freeboard Design for Wave Run-up and Wind Setup

In general, freeboard design should consider reasonable combinations of appropriate components defined on Figure 5.2. However for most small dams, wind setup and wave run-up are the principal design criteria that are most often used to determine freeboard requirements. Wave run-up and wind setup are highly sensitive to site-specific conditions. In the U.S., for example, hurricane wind conditions may control freeboard requirements for reservoirs located along the Atlantic and Gulf coasts, whereas controlling wind conditions in mountainous regions are more strongly influenced by "orographic" effects of topography and seasonal meteorological conditions that can produce sustained high winds (USACE, 1997).

Widely used design procedures in the U.S. for calculating freeboard based on wave run-up and wind setup are outlined in USBR (1992) and EM 1110-2-1100, Part II (USACE, 2003). These procedures generally involve the following steps:

Evaluate Effective Fetch (Fe)

Fetch is the straight-line distance across a body of water subject to wind forces, and is limited for inland reservoirs by the surrounding topography. Effective fetch (Fe) is defined as the average horizontal distance in the general direction of the wind over water, corrected for reservoir plan geometry, over which a wind acts to generate waves.

Evaluate Design Wind

Use of the actual wind records from a site is the preferred method for establishing wind speed-duration curves (USACE, 2003). Alternatively, generalized maps have been developed for the continental U.S. that show contours of overland wind velocities for summer, fall, winter, and spring (USBR, 1992). Maps for the fastest mile (approximately 1-minute duration) and sustained 1-hour winds are published. USBR (1992) suggests developing 2-hour wind velocity by multiplying the 1 hour chart values by factor of 0.96. Adjustments are then made for overwater wind speeds by multiplying the map values by the velocity ratios listed on Table 7.4.1.

Table 7.4 1: Wind velocity relationship – water to land (usbr, 1992)

Effective Fetch (Fe) (km)	Wind Velocity Ratio (over water/over land)
0,8	1.08
1,6	1.13
3,2	1.21
4,8	1.26
6,4	1.28
≥ 8	1.30

The relationship between the wind velocity (Uf) over water and wind duration is found using charts based on the effective fetch (Fe).

Compute the Design Wave

Design wave terms are defined on Figure 6.4.2. Significant wave height (Hs) and wave period (T) are first computed from the design wind and effective fetch (steps 1 and 2), using design charts or equations. The deep water wave length (L) is then computed as follows:

$$L = 5.12\ T^2$$

Wave run-up (R) is defined as the vertical height above still-water level (SWL) to which water from an incident wave will run up the face of the dam. Wave runup is calculated as follows:

$$R = Hs/(0.4 + (Hs/L)0.5\ \text{Cot}\quad)$$

Correction factors are applied for run-up when the wave propagation direction is not normal to the upstream face of the dam. For embankment dams with smooth upstream faces, the computed run-up is increased by a factor of 1.5.

Different design charts are used to evaluate run-up on rockfill dams or on earthen dams with a rock U/S protection.

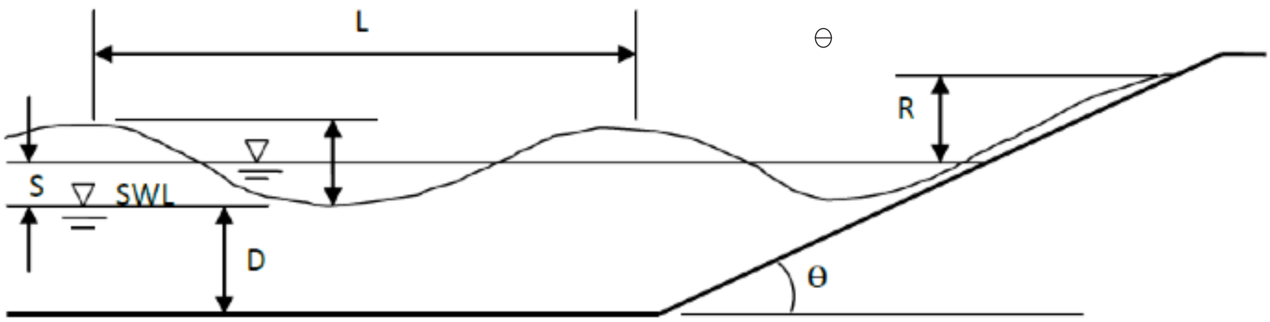


Figure 7.4 2: Definition of terms for design wave parameters

Wind setup (S) is computed as follows:

$$S = \frac{U_f^{\,2} F_e}{1400 D}$$

Where Uf is the design wind velocity over water, and D = average water depth along the fetch. The freeboard requirement for wind-generated waves is the sum of the wave run-up (R) and wind setup (S). Freeboard is generally based on maximum probable wind conditions when the reference elevation is the normal operating level. When estimating the freeboard to be used with the probable maximum reservoir level, a lesser wind condition is used because it is improbable that maximum wind conditions will occur simultaneously with the maximum flood level (USACE, 1997).

7.4.4 First Approximations for Freeboard Requirements

As a first estimate, the U.S. Bureau of Reclamation (USBR, 1992) uses the empirical guidelines summarized on Table 7.4.2 for preliminary studies of small dams. The values shown on Table 7.4.2 were based on wind velocities of 160 km/hour (100 miles/hour) for estimating normal freeboard and 80 km/hour (50 miles/hour) for minimum freeboard.

Table 7.4 2: Free board requirements for preliminary studies of small dams for rock faced slopes (usbr, 1987, 1992)

Longest fetch (km)	**Normal Freeboard (m)**	**Freeboard – MFL(*) (m)**
< 1.6	1.2	0.9
1.6	1.5	1.2
4.0	1.8	1.5
8.0	2.4	1.8
16.0	3.0	2.1

(*) MFL = Maximum flood level.

A large number of dams have failed due to overtopping and consequently greater attention must be paid to this feature. Freeboard should not be less than 1,0 m, even for small dams, as pointed out by Lewis (2002).[4]

In the French Guidelines for Small Dams, 2006, it is recommended that the minimum freeboard for fill (providing a safety margin from maximum water level, settlement and upstream cracking of the crest) depending on the parameter H2 √V , in which the minimum values are presented in Table 7.4.3. Of course, if calculations using more detailed formula give a higher value for freeboard, that higher value should be used. In such a case a less rigid wave wall (e.g. gabions) can provide protection between minimum freeboard and calculated freeboard.

Table 7.4 3: Minimum freeboard for fill dams according to parameter h2 √v

H2 √V	5	30	100	200
Rmin (m)	0,40	0,60	0,80	0,70

According to practice in Czech Republic, and considering that the breakwater at some small dams with large reservoir and long run-up of water waves can be considered.

Table 7.4 4: Height of wind waves

Kind of reinforcement of the dam slope	**Effective length of wave run-up in meters**	**Height of the wave run-up in m For a design speed 72 km.h-1**	
		1 : 3	1 : 2
Coarse surface (stone packing, armouring, vegetation cover)	100	0.33	0.42
	200	0.43	0.54
	300	0.50	0.64
Smooth surface (asphalt-concrete, concrete, pavement)	100	0.42	0.53
	200	0.54	0.67
	300	0.62	0.80

7.4.5 Camber

Camber (overbuild) of the crests of embankment dams is generally provided, in addition to conventional freeboard allowances, to accommodate anticipated post-construction settlements. Federal guidelines in the U.S. also recommend increasing freeboard in areas that have high seismic activity to accommodate the possibility of permanent embankment displacements and/or reservoir seiches during large earthquakes (ICODS, 1998). If the reservoir rim is unstable, additional freeboard may also be provided for the possibility of landslide-generated water waves and/or displacement of reservoir volume.

7.5 SOIL COMPACTION

Compaction is the most important factor in achieving a stable, durable and solid earth embankment, which is resistant to the constant seepage of water through the soil as well as having stable slopes. It is important to point out that many dams failed because of poor compaction.

Compaction occurs when pressure is applied to the soil so that the individual soil grains are pushed together as air is expelled. Compaction in the field is directed at reducing the percentage voids to less than 5%. Compaction of soil to a certain standard (i.e. density and water content) not only prevents excessive leakage and failure but also provides the basis for the determination of other properties e.g. strength, permeability, settlement and elasticity. By applying compaction to a specific standard a norm is set against which the properties are known and the behaviour and safety of an embankment can be forecasted.

Stress/strain deformation characteristics of embankments are also important. Embankments should under no condition develop low compressive stresses and excessive shear forces as this can lead to failure.

7.5.1 Compaction Standards

Regardless of the type of compacting equipment or the degree of cohesion of the soil, the effectiveness of the compaction procedure depends to a large extent on the moisture content of the soil. This statement applies especially to almost nonplastic uniform fine-grained soils.

Maximum compaction is obtained at certain water content for a specific energy application. In the Fig. 7.5.1 is presented the relation between dry density and placement moisture content for a particular soil under a specific compaction procedure for a given compaction equipment.

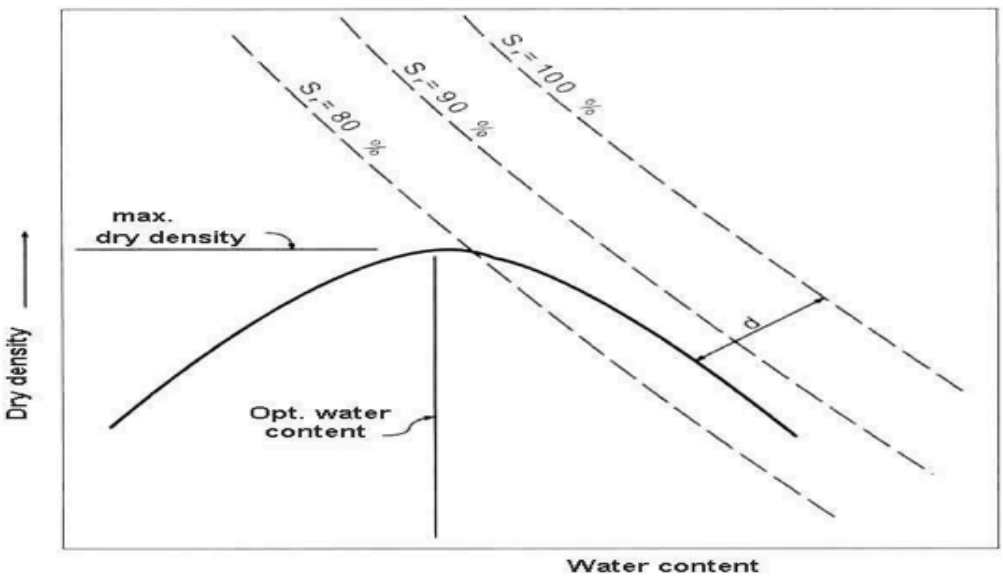


Figure 7.5 1: Usual relation between dry density and optimum water content for a specific compaction energy

When the soil is compacted by the same method but using different compaction equipment (different energy), it is obtained a family of similar curves, as illustrated in Fig. 7.5.2.

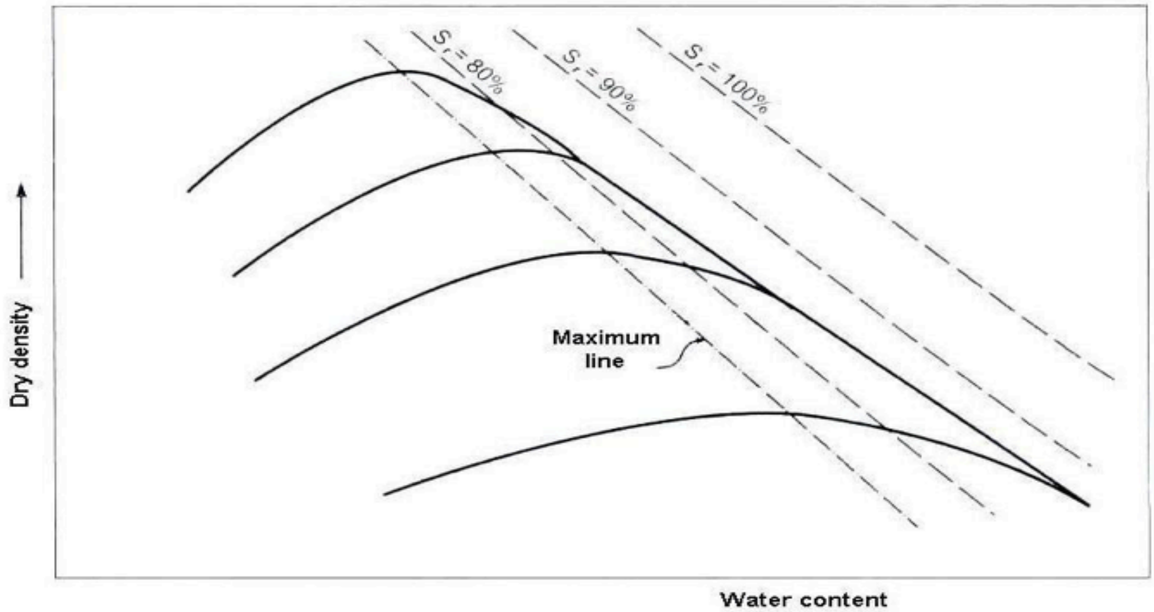


Figure 7.5 2: Typical moisture-density curves for the same soils but with different compaction energy.

With reasonable care it is usually possible to maintain the water content at +/- 2 % of optimum, which is the usual standard.

If the moisture content of the soil in the field is greater than the optimum, the soil should be given an opportunity to dry out in storage after spreading.

If it is less, water should be added in the borrow pit or by sprinkling before compaction. The addition of water can be undertaken by:

- Mixing water into the soil by cultivation with a disc plough or rotary hoe;
- Irrigating as the soil is spread out on the embankment;
- Deep ripping and irrigating the soil before excavation, also known as "borrow pit irrigation".

Borrow pit irrigation is usually more economical than adding water directly to the construction surface. It results in more even distribution of water, and saves time by avoiding the necessity to water the construction surface between each layer.

The water content at which a soil is compacted has an effect on all the physical properties of the compacted soil, including the permeability. Experience indicates that an increase in initial water content from a value somewhat below the optimum to a value somewhat above is likely to cause a large decrease in the coefficient of permeability.

7.5.2 Water Content Variation and Effect on Geo-mechanical Properties

For the core of an embankment, sealing and therefore elasticity and cohesion are dominant and strength is second in line. For the outer zones of an embankment, stability and therefore strength is most important and elasticity not a priority.

The above provides the motivation for specification of water variation 0% to 2% of optimum for core impervious materials and –1% to +2% or +3% for semi-pervious materials.

The strength and elasticity values obtained in the laboratory at the specified density and optimum moisture content of a material can be taken as design parameters if water content and density control were done during construction in accordance with the mentioned criteria.

7.5.3 Layer Thicknesses

Layer thicknesses are to be tested with the placement and construction machines to be used during construction. Special care for proper mixing with water and effective compaction through the complete layer must be investigated. Test sections must also be constructed, opened and checked visually.

If layer thickness cannot be determined experimentally then layer thicknesses must be limited to 20 cm after compaction, depending on the material. Maximum thickness after compaction must always be limited to 30 cm.

7.5.4 Quality Control During Compaction

This is very important. The frequency of testing per layer and per volume placed must be specified. A test section is normally specified with more frequent testing by Troxler and sand replacement methods. Troxler machines must be calibrated. When soil materials in borrow areas change a new Standard Proctor maximum dry density and optimum water content characteristic must be determined. This must be used as standard to compare the characteristics of the placed and compacted materials.

When during compaction a layer dries out before the next layer is placed the layer must be reworked and recompacted at the correct water content after watering and mixing. Badly compacted layers can cause high seepage rates and possible piping failures.

7.5.5 Compaction in Confined Areas

In areas inaccessible to large compaction machines compaction by smaller machines in thinner layers have to be performed. If necessary compaction by hand can be used but then at 2% of optimum water content to ensure proper wetting of all soil particles. Quality assurance here is of most importance as many failures of earthfill dams occurred due to piping along say the bottom outlet pipe caused by below standard compaction (Oosthuizen 1985).

Figure 7.5.3 shows a pipe with intact seepage collars, with the photograph taken after an internal erosion failure of a small dam. The fact that anti-seep collars have not prevented many failures implies that most of the crack features causing the failure occurred in the earthfill outside the collars, probably as a consequence of the poor compaction of the soil between the collars.
In some cases internal erosion occurred through hydraulic fractures in surrounding embankment soils that were dispersive clays, but not along the conduit as is often presumed.

Figure 7.5 3: Pipe with intact anti-seepage collars shortly after failure of a small dam (mccook/2004).

Seepage and piping along outlet pipes are a major cause of failure of small dams. As pipes become larger in diameter it becomes more difficult to compact the soil properly, down the sides of the pipe. A good precaution consists in placing draining or filtering granular material on either side of the pipe over its downstream third, in order not to block or drainage strips in the fill.

7.5.6 Testing of The Pipe

After the pipe is in place, and before any backfilling, it should be tested for leakage applying pressure through the use of compressed air. The weak point will be at the joints. Testing should be carried out at a pressure of 150 % of the maximum working pressure, as used in Australia (Lewis, 2002). No leakage is permitted over a two-hour test period. Where pipes that are prone to damage are used, such as rubber ring-jointed PVC pipes, a further test is recommended after 1.0 meter of soil cover has been

emplaced. Failure at this time can be corrected at a much lower cost than after completion of the embankment.

The French Guidelines for small dams recommend to do a water tightness test before embedding in concrete by plugging the two ends of the pipe and raising pressure up to twice the reservoir depth plus 0.2 MPa, and then maintaining this pressure for 8 hours.

7.5.7 Compaction of Filters

Recent practice regarding compaction of filter materials is in the line of not to compact filters in excess, for the following reasons:

- filter material must collapse during differential movement of the adjacent earthfill materials
- grading of filter materials change during compaction and after compaction filter criteria or permeability requirements may not be met (Sherard, 1984).

If filters are compacted it is recommended that its strength, crushing strength and grading are tested during construction with the specific machine.

7.6 DAM FOUNDATION TREATMENT

The dam foundation must always be stripped to a depth of at least 0.50 meter to remove topsoil, organic material, trees, roots, grass, etc.

The mechanical characteristics of loose materials in the foundation (alluvium, colluvium, eluvium) are often sufficient to support a fill dam about 10 to 15 meters high. In regions where predominate warm weather most part of the year, the presence of organic soils at the river bed is very frequent, and in such cases it is recommended to remove such layers completely, as a consequence of their low shear strength and high compressibility.

Organic material in the form of partly decomposed vegetation is the primary constituent of peat soils. Varying amounts of finely divided vegetation are found in plastic and nonplastic sediments and often affect their properties. Organic soils are dark gray or black and usually have a characteristic odor of decay. The tendency for soils high in organic content to create voids as a result of decay or to change the physical characteristics of a soil mass through chemical alteration makes them undesirable for engineering use.

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

Water tightening and drainage systems must be installed to achieve an acceptable leakage flow and avoid any risk of piping (internal erosion) and uplift on the downstream side. A filter may have to be placed at the fill dam/ foundation, at the downstream axis position in order to assure better internal drainage conditions to the earthfill.

7.6.1 Foundation Watertightness

The three following cases can be considered for homogeneous and zoned dams, as pointed out by the French Guidelines on Small Dams.

Foundation consisting of relatively impermeable materials

It is recommended that a cut-off trench be built of compacted clay materials in order to deal with any surface cracking or heterogeneous zones. The dimensions of such a trench should be:

- Minimum width at the base: 3 meters;
- Side slopes of the order of 1/1;
- Some meters depth with a minimum of 2 meters below natural ground level.

Foundation with permeable layers to a depth of a few meters

The trench must cut through those layers and be anchored in a watertight layer. If the latter is unaltered rock, after it has been cleaned and possibly its surface has been smoothed, sweeping or washing of the surface and infilling the opening joints with cement mortar, and placing a first layer of wet (optimum moisture content (+ 2 or 3%) clay a few decimeters thick, is placed to guarantee good contact, it may be necessary to set a filter between the downstream face of the trench and the permeable foundation materials.

Permeable foundation to a significant depth

Grouting can be used both for a loose foundation and for a more or less cracked rock mass, with the grout adapted to the material being treated (bentonite-cement grout, specially designed grouts); the cut-off will usually involve three lines of staggered drill holes; as grouting cannot be effective at the surface, either the first few meters of grouting are relayed by the cut-off trench, or treatment is started at a certain height in the fill.

As concerning dams with an artificial water tightening element, the connection between this structure in the fill and the water tightening in the foundation is a difficult point.

When the reservoir cannot be water tightened by a cut-off at the dam, the solution consists in sealing the reservoir basin totally or partially with a geomembrane or with a blanket of compacted clay materials (at least two layers about 0.20 meters thick each), with the latter protected from any risk of drying out. Such techniques always result in a high price per cubic meter of water stored, and it is always very hard to avoid any superficial cracks along the impervious layer.

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

- Meet filter conditions for an upstream blanket;
- Eliminate any rough areas that might puncture the geomembrane;
- Avoid any risk of uplift, in particular due to gases under the geomembrane.

Flattening of steep slopes especially at the river banks to slopes meeting differential settlement criteria of above soils. Maximum slopes of 1V:1H is recommended where specialised techniques are not applied. All corners, holes and unevenness have to be filled with concrete to provide a surface on which positive compaction can be supplied.

Rough surfaces on which positive compaction of earthfill is not possible, are to be smoothed by blooming a cement mortar.

7.6.2 Foundation Drainage

For drainage of flows from the foundation, the most satisfactory solution consists in placing a drainage blanket at the base of the downstream shoulder, at the fill-foundation contact, leading to the vertical or slopped drain in the central part of the fill.

This drainage blanket, which may be compartmented in order to determine the behavior of each different zone, should be placed for any large structure ($H^2 \sqrt{V} > 700$). For smaller dams ($H^2 \sqrt{V} < 700$), where geological conditions permit, the drainage blanket may be reduced by placement of draining strips (in particular in the areas judged to be the most vulnerable in the river banks). The thickness of the layers must be sufficient to discharge the foreseen flow with a minimum thickness for each horizontal granular layer of 0.20 meters (drain and filter). When using only sand the minimum layer thickness need to be 0.50 meters for practical purposes.

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

Drainage of the foundation for very small dams where H2 √V <20 provided that the foundation is sufficiently watertight.

Special attention must be paid to the placement of the first earthfill layer. In case of compaction of earthfill against earthfill the top layer of in-situ soil must also be ploughed and mixed with the first imported layer. In case of compaction of the first earthfill layer on a rock foundation use of a sheepfoot roller is to be prevented so that the foundation surface is not damaged. In confined areas hand compaction in thin layers 0 – 3% above optimum water content must be specified (slush grouting).

The contact below the core cut-off must be treated with special care. An area 3m upstream of the core as well as the sides of the core must be treated, rock surfaces especially with concrete to enhance good compaction. The contact below the outer zones can be treated at a lesser degree. Rough areas and holes for example can be treated with earthfill instead of concrete.

Slopes in the foundation are not to be provided too steep because e.g. where the elasticity modules of clay is less arch action can develop with associated low stresses. With high hydraulic gradients water can burst and a leak can develop. Finite element techniques can be used to determine the correct slope.

7.7 DRAINAGE SYSTEM FOR EARTHFILL DAMS

A drainage system for a homogeneous fill dam consists of two parts (see Figure 7.7.1):

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

It is proposed that the thickness of the chimney drain (0.50 m minimum) could be decreased as the fill is placed according to the value of H^2 v corresponding to the lower level of the part in question (for a chimney drain with two or three thicknesses in all). Table 7.7.1 gives the recommended minimum thicknesses.

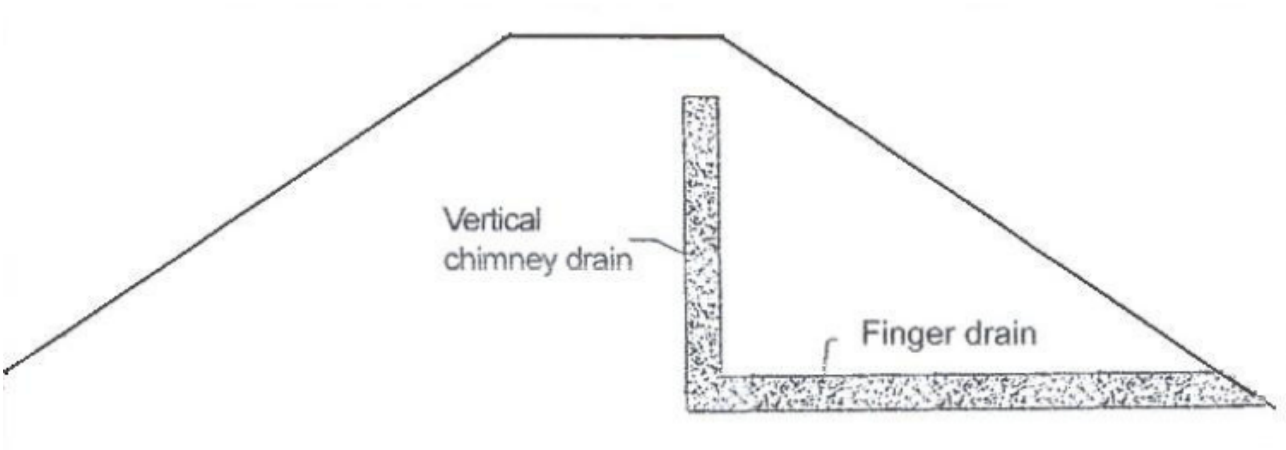


Figure 7.7 1: Drainage system for a homogeneous clayey fill.

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

Table 7.7 1: Minimum thickness of a sand chimney drain (french criteria)

H2 √V	<30	30 to 100	100 to 200
Thickness	0.50 m	0.80 m	1.00 m

7.7.1 Seepage Control Measures

Seepage from the reservoir takes place through the embankment and foundation materials. The top surface of the water seepage mass is called the phreatic surface. The mass moves under pressure higher than atmospheric pressure and is carried through the pores around soil particles, through fractured zones or cracks etc. Hydraulic gradient refers to non-dimensional unit change in hydraulic potential between two points in the flow medium (water).

The basic practice in seepage control design is to provide a number of seepage control measures to ensure maximum water storage capability of a dam. Defensive design is necessary to accommodate a series of unknown factors which can be as follows:

- Unknown characteristics of geological and foundation materials due to limited available data.
- Degradation or ageing of seepage control measures or materials.
- Change in the operations method of a dam.
- Non conforming quality control during construction e.g. below standard compaction.

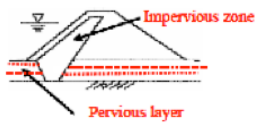
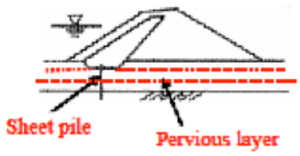
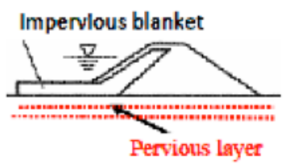
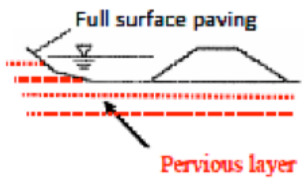
Dispersive clay materials susceptible to piping can be used in the central part of the embankment as core to provide an impervious medium and be protected with semi-pervious materials not susceptible to piping in the shells.

7.7.1.1 Measures for Pervious Ground

In a case where it is impossible for engineering reasons or uneconomical to insert an impervious zone to seepage resistant ground, the quantity of seepage must be maintained in the allowed range and seepage water must be safely discharged outside the dam with reference to Table 7.7.2 considering on-site conditions.

The blanket method is an example of this Seepage Control Measures.

Table 7.7 2: Treatment of pervious ground (japanese criteria)

Thickness of pervious layer	Design method	Sketch	Applications
Thin	Impervious zone		Provides complete impervious effects. But it is used when the pervious layer thickness is within 1/3 of the present dam height above the ground
Medium	Sheet pile		Provides incomplete impervious effects. It is not suitable for a layer containing pebbles. It is effective in fine sand and silt layers.
	Grout		It is effective in a pervious bedrock layer.
Thickness	Blanket		It effectively prevents piping. It is inexpensive.
	Full surface paving		It effectively prevents piping. It is inexpensive. It is extremely expensive. It is only used to restrict leakage to an extremely small quantity.

During the execution of the upstream blanket especial care must be taken with superficial cracks as a consequence of the drying clayed soils.

7.7.1.2 Blanket Grouting Method

This is a method that restricts seepage of reservoir water by controlling the vertical seepage flow inside the reservoir, and by making the seepage route longer it lowers the hydraulic gradient accordingly and it reduces the seepage quantity.

- Natural blanket

  $Xr = \sqrt{(t.d.k/k1)}$

In the case where impervious soil is deposited as the surface layer of the pervious ground to form a natural blanket, the effective seepage length xr created by the blanket is obtained by equation (1)

Where:

t :blanket thickness(m)
d :pervious ground thickness(m)
k1 :coefficient of permeability in the vertical direction of the blanket(m/s)
k :coefficient of permeability of the foundation ground(m/s)

And xr is, as shown in Figure 5.8, the horizontal distance of the blanket necessary to create the head loss (Δhb). This means that the head loss caused by this blanket is equivalent to laying a completely impervious layer horizontally for only xr in the upstream reservoir. And the quantity of seepage in the foundation ground qf is obtained by equation (2)

qf =k h d/(Xr + Xd)…………………………………………………………….……(2)

Where:

q_f :seepage in the foundation ground (m3/s)
h :difference between the reservoir water level and downstream water level(m)
x_r :effective seepage route length(m)
x_d :width of the dam body base (m)

(Head loss caused by the blanket)

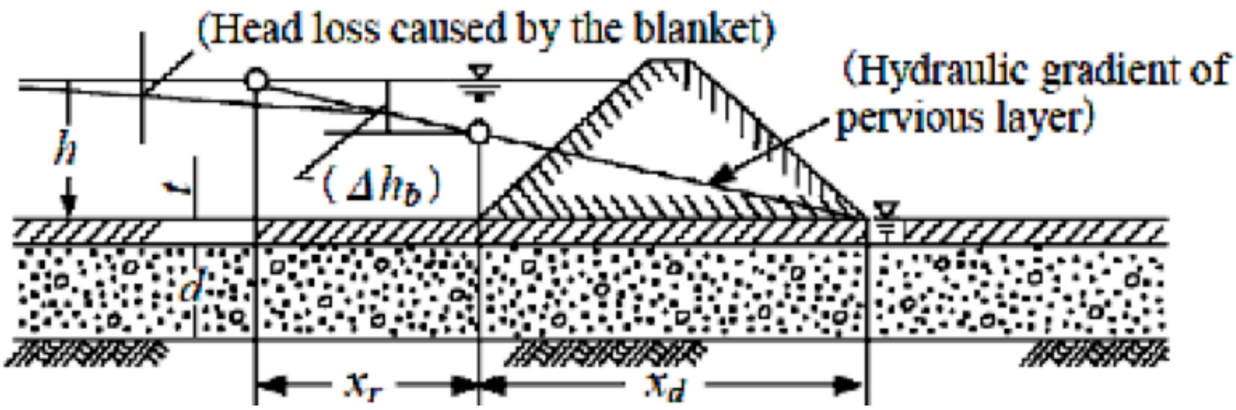


Figure 7.7 2: Natural blanket (impervious)

7.7.1.3 Artificial Blanket

The necessary length of an artificial blanket x is calculated by equation (3)

The value of q f is determined based on the allowed leakage of the reservoir, xr for this is obtained by equation (1), and the required length of the blanket x is obtained by substituting xr in equation (3). The standard for the thickness is 1/10 of the water pressure. It is often from 1.0 to 3.0 m and the closer to the dam body, it is thicker, and the further upstream, it is thinner.

But care is necessary, because in ground with a large coefficient of permeability in the horizontal direction, only executing a blanket may not necessarily provide adequate resistance to piping.

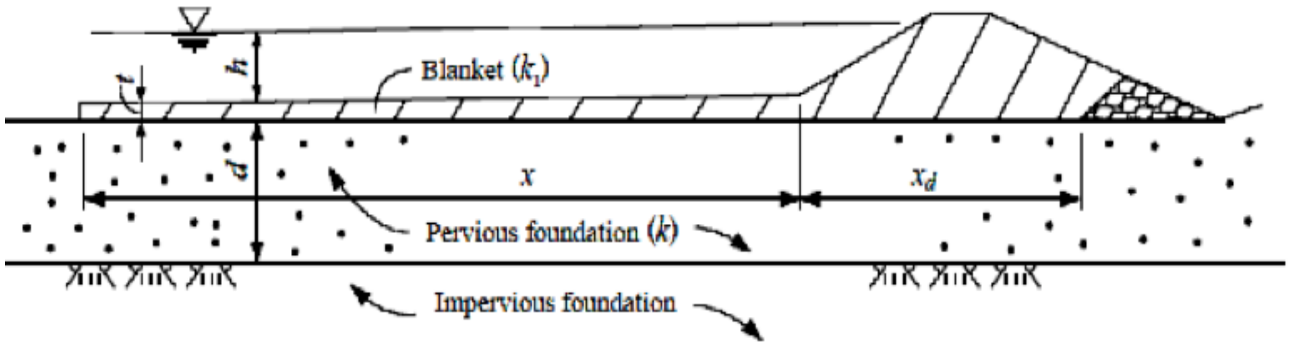


Figure 7.7 3: Artificial blanket design method

7.7.1.4 Practical Considerations

Water can seep through horizontal layers especially where compaction standards are not met. The provision of a chimney drain can intercept seepage. In case of small dams expensive drains are sometimes omitted. In case of dispersive earthfill materials drains, especially the chimney drain, must be inserted and extended into the core trench, because the correctly designed chimney filter not preventing migration of small soil particles is then a requirement. In case of differential settlement between embankment material zones or cracks a chimney drain is a requirement. Sherard (1984) showed that based on laboratory tests filter systems can heel concentrated leaks. Collars of sand must be provided around bottom outlets on the downstream side to prevent piping formation when seepage occurs through below standard compaction areas.

The thickness of a drain can be determined using Darcy's law ($Q = kiA$) by determining the area required. The hydraulic gradient can be taken as the relation of the height difference between the beginning and end of the system and the length of the system. The following aspects are also important:

- Construction constraints, e.g. the limitation of machines to place filter materials in thin layers, cause the minimum thickness to be 25 cm.

- Safety factors regarding theoretical capacity and design capacity of drains of 10 to about 100 are warranted as provision must be made for unknowns, decreases in permeabilities due to over compaction, ageing of materials, cementation of sand in filters.

7.7.1.5 Filter Material Criteria

Based on the bi-lateral functions of filter and drain systems namely maximum retention of soil material of the base and maximum through flow of water, limits of grain sizes for base and filter material were established. In 1922 Terzaghi developed piping criteria which are still applicable. Elges, HFWK (1986) describes natural filters. Sherard et al, 1985.

7.7.1.6 Piping Prevention Criteria

The criteria for the prevention of removal of particles and failure of the base material including dispersive materials by a filter are shown in Table 7.7.3.

Table 7.7 3: Criteria regarding piping

Criteria N°	Description
1	D15 filter / D85 basis = 5 or less
2	D15 filter / D50 basis < 25
3	D15 / D85 < 5 for sandy silt and clay (D85 of 0,1 to 0,5)
4	D15<0,5 for fine clay (D85 from 0,3 to 0,1)
5	D15<0,3 for fine silts with low cohesion and plasticity (LL<30 (D85 from 0,3 to 0,1)
6	D15 < 0,2 for fine soil (D85 of 0,02)

Criteria Regarding Permeability
Criteria ensuring sufficient flow in the filter/drain are shown in Table 7.7.4.

Table 7.7 4: Criteria for drains

Criteria number	Details
1	5 < D15 filter / D15 base < 40
2	The grain size associated with the 0,075 sieve size based on the washed grading of sand must be less than 5%

Note: Laboratory studies showed that the permeability of filter sand significantly (in portions of 100) decreases with the fines more than 5% through the 0,075 sieve size. The washed grading test must be used to remove the fine fraction (Cedergren, 1977).

7.7.1.8 Uniform Criteria

Depending on the uniform characteristics of the materials the limits for criteria number 1 in Table 7.7.4 must be used as those shown in Table 7.7.5.

Table 7.7 5: Filter uniformity criteria

Characteristic	Lower limit	Higher limit
Uniform: D60/D10 = 3 to 4	5	40
Non-uniform: rounded grains	12	40
Non-uniform: sharp edge grains	6	18

7.7.1.9 Criteria for the inherent stability of a filter

The Stability Relation defined as D15 (coarse) / D85 (fine) applied to 5% intervals on the particle grain size distribution must always be smaller than 5. Sherard showed in 1984 that the coefficient of uniformity Cu = D60 / D10 should satisfy the following conditions:

Table 7.7 6: Criteria for inherent stability of a filter layer

Cu-value	Guidelines
10	Inherent instability possible
10 < Cu < 20	Inherent stability only possible in soils with grading curves with sharp changes in bends
20 < Cu < 75	The soil will be stable when the grading curve is smooth without sharp changes in directions or significant flat zones.

7.7.1.10 Cohesion Clay Criteria

It is generally accepted that cohesive non-dispersive clays are stable in some cases where above-mentioned criteria are not satisfied. The standard is not well defined. The explanation by the Corps of Engineers in 1955 is mostly accepted.

"The above criteria will be used when protecting all soils except for medium to highly plastic clays without sand or silt partings, which by the above (basic) criteria may be required multi-stage filters. For these clay soils, the D15 size of the filters may be as great as 0,4mm and the above D50 (supplementary) criteria will be disregarded (This refers to the D50 (filter)/D50(base) < 25 rule). This relaxation in criteria for protecting medium to highly plastic clays will allow the use of one-stage filter material; however the filter must be well graded, and to ensure non-segregation of the filter, a coefficient of uniformity (ration of D60 to D10) of not greater than 20 will be required".

7.7.1.11 Criteria Regarding Dispersive Clay

First criterion is that the filter material must not be dispersive. Criteria stipulated in Table 7.7.6. are to be followed. Second practice to obtain less permeability is to compact to at least 98% of the Standard Proctor density at 0% to 2% above optimum. The paper by Melvill (1986) describes further aspects regarding filtration of dispersive soils.

7.7.1.12 Organic Material Criteria

Less than 2% organic materials in filters are acceptable.

7.7.1.13 Criteria and Practice Regarding Synthetic Materials

Woven, needle punched polyester synthetic materials are available in South Africa, since 1972. Synthetic materials are to be compared to natural materials regarding filter criteria. The exact size and void ratios, however, can only be determined accurately using experimental data.

It is generally assumed that synthetic materials have an application. The following is important regarding the application to dams:

- Stresses in embankment caused by differential settlement can cause the material to be torn apart which influences the permeability and stability against piping.
- Synthetic materials can therefore be used where permeability is not a priority e.g. downstream side of sand of a chimney filter.
- Access to synthetic materials must be possible. Therefore use as chimney filter material is

not acceptable as access is not possible. It must be remembered that the material can clog.

- Construction problems are to be acknowledged. Synthetic materials can be damaged under vehicle loads. When one hole develop, water and soil can move through with no control regarding filter material. Ultra-violet rays of the sun can cause deterioration and exposure must be prevented.

7.8 SLOPE STABILITY

7.8.1 Introduction

To prevent failures that can develop through the slopes of embankment dams the slopes must be flattened to acceptable grades which provide acceptable safety factors, based on the characteristics of the soil from which it is built.

Stability analysis of a fill dam concerns the determination of the forces exerted on the dam and analysis of combinations of those forces; of those combinations, the worst case scenarios are considered in terms of the envisaged failure mechanism.

7.8.2 Critical Cases for Analysis

During construction and life of an earthfill embankment structure the following cases may occur:

- During construction temporary slopes must always be stable. Furthermore, the outer slopes must also be stable during development of hydrostatic pore pressures.
- During full reservoir conditions and developed seepage conditions the stability of the downstream shell is critical. Sufficient drainage systems will improve the stability.
- In case of sudden draw down of water in the reservoir the upstream slope can fail under saturated earthfill conditions.

Dynamic forces can also develop and must be considered in seismic zones.

7.8.3 Limiting Equilibrium Methods

The two dimensional equilibrium methods are based on the following:

- An embankment cross section is evaluated.
- The earthfill above the slip failure is divided into blocks.
- Each block is analysed for weight and shear resistance and the final safety factor determined.

For all methods except for the wedge method vertical blocks are selected. For the wedge method a series of levels are selected and is applicable to the following:

- where a horizontal earth layer with lower shear resistance (e.g. clay with high plasticity) is situated in the foundation.
- where the foundation consists of rock, the core consists of material with small grains and the outer zones consist of material with rough grains.
- Methods

Various methods as follows were developed in years indicated:

- Simple Bishop (1955)
- Spencer (1967)
- Janbu (1957)
- Morgestern and Price (1965)
- Determination of most critical slope surface

The following approach must be followed to select the most critical surface:

- Determine a series of safety factors with various radii but same centre point and plot on scale.
- Redo above with different centre points encroaching to the lowest safety factor. Plot of safety factors contours to be made.
- More than one critical surface must be identified for e.g. rockfill dams where surface slopes are occurring.
- Selection of shear parameter

The selection of shear parameter to be tested meeting site conditions is normally based on specific test methods. A summary and explanation are given in Table 7.8.1.

Table 7.8 1: Summary of triaxial tests

Test method	Application in stability analysis	Remark
UU	Test represents end of construction condition for core zones. Also applicable to impervious foundations where consolidation is slow in comparison with rate of earthfill placement.	At low test pressure cavitation occur during shear. Stress curves may have curves if materials are saturated.
CU (Consolidated, Undrained)	Test represents behaviour of impervious or semi-pervious materials of embankments or foundations which consolidated during construction and which were exposed to stress change during sudden draw down conditions. Test also used to analyse stationary conditions in downstream slope.	Undrained stress increases with decreasing moisture content and increases in consolidation stresses.
CD (Consolidated, Drained)	Test applicable to impervious and semi-pervious soils with hydrostatic pore pressures, before or after shear, under slow increase in load is revealed. Also used to evaluate shear values under setting out conditions where excess hydrostatic pressure during the life of the dam is determined.	CU strength is higher in value than CD strength for expansion soils.

7.8.4 Minimum Safety Factors Against Slip Circle Failures

A summary of recommended minimum safety factors, shear test and application is shown in Table 7.8.2.

Table 7.8 2: Summary of design case, minimum safety factors, and shear test

Design case	Minimum safety factor	Shear Test **	Applicable to slope of embankment
End of construction	1,3	UU or CD *	Upstream & downstream slopes
Sudden draw from full supply level	1,2	CU or CD	Upstream slope of full section
Normal operation	1,5	CD	Upstream slope
Seismic forces, (cases above)	1,1		Both slopes

* In zones where no significant hydrostatic pore pressures are excepted, use strengths as determined in CD test.
** Refer Table 7.8.1. for definitions.
Note: Effective stresses are to be used.

◊ Other important factors

The following aspects are important:

- The effects of differential settlement in steep valleys on stability.
- The increase of hydrostatic pore pressures placed above optimum moisture content. High hydrostatic pore pressures decrease shear resistance and therefore the safety factor.
- Some materials originated from e.g. weathered mudstone and shales are problematic soils of which the shear properties decrease with time.

Non linear finite element methods can be used to determine stresses and strains for specific problems.

7.9 SURFACE DRAINAGE

The main function of the surface drainage of small dam bodies is a secure diversion of rainfall water so, that no undesirable effect of water erosion or other damaging impacts on the surface of the dam occurs. Possible constructions and methods for surface dewatering are not as varied as in the case of inner draining elements. Yet, it is necessary to mention some principles and possible solutions.

7.9.1 Principles

- The elements of surface drainage of dam bodies slopes are proposed further to properties of the material, the dam is built up of as well as to the shape of the dam in a cross section, i.e. depending on the rate of resistance to the influence of the flowing rainfall waters. The surface drainage finds its use mainly at the embankments dams built from the earth materials.

- The elements for surface drainage are usually applied at dams higher than 6 m. At lower dams, the fundamental condition of a good drainage is a good and maintained grassing over of the downstream slope or an effective and stabile fortification of the upstream slope.
- For the design of the surface drainage, knowledge of the filtering environment (mutual relations regarding the permeability of the dam and the background) is important. It is unnecessary to use the surface drainage in a case, when a permeable sandy dam is founded upon a permeable (sandy or gravelly) and more than 3 m thick background.
- At dams higher than 10 m the slopes are usually divided by berms, which in a case of downstream slope use to be 5 ÷ 10 m vertically from each other, are min. 1,5 m wide (better 3 m) and have a longitudinal inclination 1 ÷ 2 %. The berms restrict the flow velocity of rainfall waters on the slopes and contribute to the protection of the dam surface against the water erosion.
- From the dam crest, upon which a road surface is solidificated by asphalt, blocks etc., the rain-fall waters are drained to both abutments of the dam by a surface-water sewerage system.

7.9.2 Possible Mechanical Solution of the Surface Dewatering

- Upon the lower part of the downstream slope and to the area of downstream toe, into a partly digged out adjoining downstream area (ditch-shaped), it is possible to put layers of permeable materials in a configuration of a inverted filter (Figure 7.9.1).

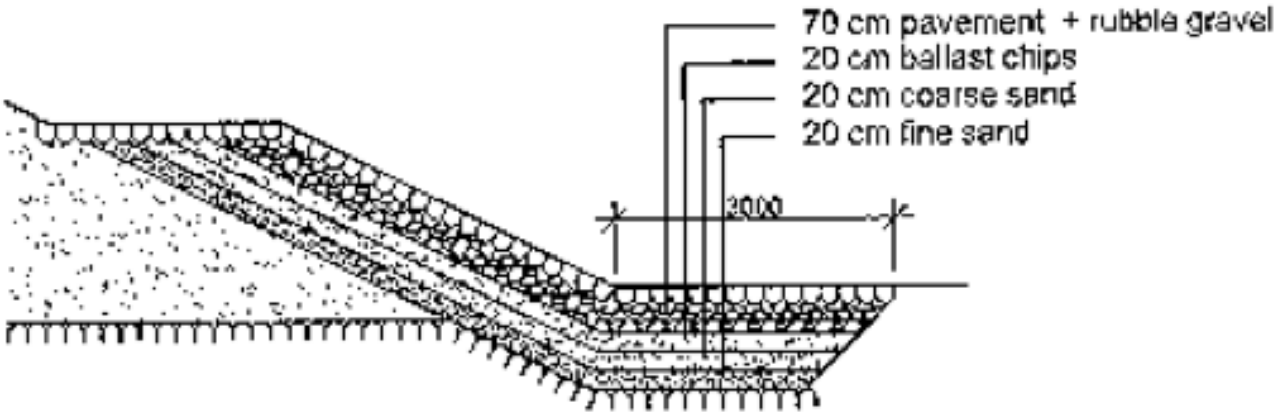


Figure 7.9 1: Configuration of a inverted filter.

By this modification the surface water will be drained into a downsloped permeable element. The construction is suitable for earthy or earthy-sandy homogenous dams for territories with mild winters (without a risk of freezing), with a dam height approx. to 10 m.

- The formation of longitudinal strengthened open ditches, draining gravitationally the rainfall water running upon the grassy downstream slope along the downstream toe to the channels from outlets or from spillways. For to limit the expansion of energy of the flowing water it is convenient to place the dewatering ditches or rills also upon the berms, as long as the dam has them. The waters are then safely deflected into the downstream area upon a slope with a strengthened slope in the dam abutment.

- In case of concerns for a local damage of the construction by a concentrated rain-fall water at other dam types (especially made of concrete and masonry) it is possible to utilise common elements of a surface-water sewerage system (e.g. plumber's downcomers etc.).
- The mentioned dewatering elements must be regularly maintained if a reliable function should be secured steadily. It is especially necessary to clean the foot ditches of fallen leaves, accumulated dry grass and of other undesirable stuff, which could restrict their function. However, due to their good accessibility in comparison with the inner dewatering systems, the main advantage of the elements for surface drainage is their easy checking, trouble-free maintenance and relatively easy repairs.

Other technical solutions of surface dewatering of dams are used in various countries locally, according to the local conditions. It is rather a question of unique constructions which should not be generalized.

7.10 SLOPE PROTECTION

7.10.1 Upstream Slope Protection

The upstream slopes of earthfill dams must be protected against destructive wave action. In some instances, provision must be made against burrowing animals. The usual types of surface protection for upstream slopes are rock riprap, either dry dumped or hand placed, and concrete pavement. ICOLD Bulletin 91, published in June 1993 specifically deals with protection of upstream slopes for fill dams and it is important to be consulted.

Other types of protection for small dams that have been used are soil cement pavement, and (on small and relatively unimportant structures) wood and sacked concrete. The upstream slope protection should extend from the crest of the dam to a safe distance below minimum water level (usually several feet). In some cases, it is advantageous to terminate the slope protection on a supporting berm, but it is generally not required.

Where the water level in the dam can be expected to fluctuate widely, or where a high degree of protection is required, the use of a rock layer, usually called "rip-rap" is the most effective method of control, as illustrated in Fig. 7.10.1.

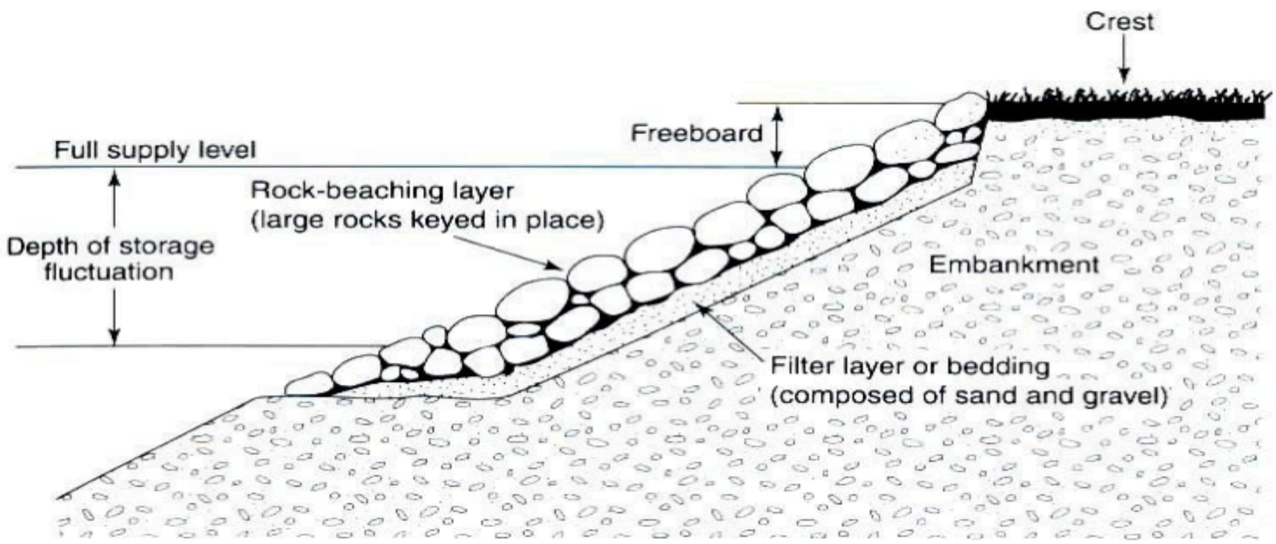


Figure 7.10 1: Typical rip-rap layer for the protection of the upstream slope (lewis, 2002).

The supporting layer for the rip-rap is intended to protect the fill from the hydrodynamic effects of waves and from erosion. The U.S. Corps of Engineers recommends the following rockfill diameter (D50) and thickness of rip-par layer presented at the following table.

Table 7.10 1: Rip-rap layer recommended by u.S. Corps of engineers

Wave height (m)	Average rockfill diameter – D50 (m)	Layer thickness
0 – 0.60	0.25	0.30
0.60 – 1.20	0.30	0.46
1.20 – 1.80	0.38	0.61
1.80 – 2.40	0.46	0.76
2.40 – 3.00	0.53	0.91

The same corporation recommends the following filter layer or bedding thickness between the rip-rap and the earthfill.

Table 7.10 2: Transition thickness layer recommend by u.S. Corps of engineers

Maximum wave height (m)	Bedding thickness layer (m)
0 – 1.20	0.15
1.20 – 2.40	0.25
2.40 – 3.00	0.30

For wave height from 0.30 to 1.55 m it is also possible to use the following table, present by the French Guidelines.

Table 7.10 3: Dimensions of upstream rip-rap (French Guidelines on Small Dams)

Wave height (m)	Thickness - e (m)	Block diameter – D50 (m)
0.30	0.30	0.20
0.55	0.40	0.25
0.80	0.50	0.30
1.05	0.60	0.40
1.30	0.70	0.45
1.55	0.80	0.50

The supporting layer may be replaced by a puncture-resistant geotextile in cases in which the fill material is fairly resistant to erosion.

7.10.2 Selecting the Type of Upstream Protection

Experience has shown that in most cases, dumped riprap furnishes the best upstream slope protection at the lowest ultimate cost. Approximately 100 dams, located in various sections of the United States with a wide variety of climate conditions and wave severity, were examined by the Corps of Engineers. The results of this survey were used as a basis for establishing the most practical and economical means for slope protection. The dams were from 5 to 50 years old and were constructed by various agencies. This survey found that:

- Dumped riprap failed in only 5 percent of the cases it was used; and failures were due to improper size of stones.
- Hand-placed riprap failed in 30 percent of the cases it was used; failures were due to the usual method of single-course construction.
- Concrete pavement failed in 36 percent of the cases it was used; failures were generally due to inherent deficiencies with this type of construction.

This survey substantiated the premise that dumped riprap is by far the most preferable type of upstream slope protection. Figure 7.10.2. presents the riprap at the upstream slope of Paraitinga dam, in Brazil, which is a very well dam built for water supply using a good gneiss as rock material.

Figure 7.10 2: Upstream gneiss rip-rap at the paraitinga dam, in brazil.

The superiority of dumped rock riprap for upstream slope protection and its low cost of maintenance compared with other types of slope protection have been demonstrated so convincingly that it has been considered economical to transport rock considerable distances for major dams.

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

Of course, partial protection of the upstream slope can be envisaged using the same technique for dams with little variation in water level, such as lakes for recreational purposes, diversion dams, etc. In this case, protection by plant cover may be considered if wave height is less than 0.50 meter.

For very small reservoirs (fetch of only a few hundred meters and good face orientation), it may be tempting to provide no upstream protection, or only a grass cover, as can be seen in Figure 7.10.13. It is always possible to take action after the face has deteriorated.

Figure 7.10 3: Upstream slope protection with grass at the chibarro dam, 12 m high, in brazil (fetch = 300 m).

During the rip-rap construction it is important to avoid the use of rock materials that can disintegrate along the time, as a consequence of the several wetting and drying cycles along the time, and the fast weathering of the rock.

7.10.3 Downstream Slope Protection With Grass

The downstream slope of an earthfill dam must be protected from the effects of rainfall runoff. Grass is practically always planted to this end on the downstream slope of small dams. For fill dams over 12 meters high, it is recommended to install an intermediate berm halfway up the downstream slope. For others dams over 15 meters high, this recommendation practically becomes a requirement. This berm offers two advantages:

- It limits the effects of runoff along the slope;
- It gives access to piezometers halfway up the slope, as well as to spread topsoil, to plant grass, and later to maintain the slope.

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

The root system of the grass cover holds the surface soil in place and protects very successfully the slope from any kind of erosion. Most of the dams which resisted and don't failed were that with a grass cover. But it is very important the grass cover be maintained by irrigation and cutting. Grass cover is useful also during the periodic visual inspections, when fresh green spots appear, they are an indication of seepage or high phreatic surface. Of course an inappropriate vegetative growth, will screen deficiencies as cracks, sinkholes and animal burrows.

The grass cover must be maintained by irrigation and cutting. Irrigation by sprinklers is necessary to prevent withering of the grass. For this reason grass cover is not fitted in arid regions. Grass cover is useful also during the periodic visual inspections; when fresh green spots appear, they are an indication of seepage or high phreatic surface.

The grass must be cut from time to time to prevent high and inappropriate vegetative growth, which will screen deficiencies as cracks, sinkholes and animal burrows. Excessive vegetation can obscure large area of the slope and prevent good visual inspection. Problems that threaten the dam integrity can develop and remain undetected if they are obscured by high vegetation. Excessive vegetation can provide habitat for rodent and burrowing animals, which burrows are a threat to the embankment dam by causing piping.

Although grass cover is desirable as slope protection, growth of deep rooted large shrubs and trees is undesirable. Their deep root system could shorten the seepage path, providing seepage pats and initiate piping, especially on the decaying root system when vegetation dies.

7.10.4 Downstream Slope Protection With Gravel

The riprap slope protection is made up of at least two layers:

- The outer layer consists of broken rock or boulders which prevent erosion.
- The inner layer is the filter or bedding and could consist of one or two layers. If it is one layer it consists of a sand-gravel mixture. If it is of two layers, the inner layer is of coarse sand and the outer is of gravel. The relation between the coarseness of the grains of two adjacent layers must be the filter rules in order to prevent the removing of material from the underlying inner layer through the pores of the outer material. If these rules are not regarded, the surface runoff can erode the under layer, undermine the rockfill layer, damage all the surface protection and form gullies in the protected embankment.

The slope upon which the riprap is placed must be flat to prevent rockfill from moving down the slope. Hand-placed rockfill, providing usually good protection, is a thin blanket. Most modern rockfill is dumped in place, resulting in a thicker blanket of protection. Vegetation on the riprap must be weeded out, because it could move and replace the stones and damage the protection. A berm on the mid height of the slope will stop the flowing down rain water and decrease its velocity. It will also increase the slope stability. A ditch on the interface of the berm with the slope to collect the flowing down water is very useful. The berm will also support the upper slope protection.

7.10.5 Protection From Seepage Piping

The downstream slope protection of embankment dams serves to protect the slope not only from surface erosion, but also from erosion due to inner seepage from the dam. For this reason it is advisable the filter under the rockfill and the concrete protection to have the possibility to avoid erosion from any uncontrolled seepage.

The junction of the downstream slope with the ground surface is protected from seepage and piping with a downstream toe. It is a small rockfill zone, divided from the main dam by one or two filter layers and acting as drain. Except of horizontal blanket drain and chimney drain with blanket, the toe zone of rock material is a good internal drain to control seepage. Dams without internal drains have seepage problems, because the seepage (phreatic) surface emerges on the slope surface. The toe dam protects the dam also from tail water if any.

The part of the valley side against which the dam is built is the abutment. Surface runoff from the abutment could cause rapid erosion on the slope-abutment interface. To avoid this there must be provided a concrete ditch to collect all rain surface water from the abutment and the dam slope. Seepage is also often coming out on the lower part of the slope-abutment interface. This contact is prone to seepage because the embankment fill near the abutment is less dense and less watertight. The embankment fill near the abutments is less dense because compaction is difficult along the interface. A toe drain is to be put on the slope-abutment interface to carry the internal seepage water away from the dam. It must be met by filter under the slope protection to avoid any seepage erosion.

7.11 CREST DESIGN

Placing a layer of gravel on the crest will in particular avoid the formation of ruts due to traffic and desiccation of the last layers of compacted clay materials. If the crest is not protected, it can experience severe erosion. Crest erosion protection is usually the road surfacing such as gravel, concrete pavement or asphalt and depends on the amount of the anticipated traffic. If no traffic is expected, a grass cover could be enough. In designing the dam crest of a small earthfill dam the following items should be considered:

- Width
- Drainage
- Camber
- Surfacing
- Safety requirements

7.11.1 Width

The crest width of an earthfill dam depends on considerations such as nature of embankment materials and minimum allowable percolation distance through the embankment at normal reservoir level, height and importance of structure, possible roadway requirements and allowable materials at the site.

Because of practical difficulties in determining these factors, the crest width is, as a rule, determined empirically and largely by precedent experience. According to the Bureau of Reclamation, in the United States, the following formula is suggested for crest width for small earthfill dams:

$$w = z/5 + 10$$

where: w = width of crest in feet

z = height of dam, in feet, above the streambed.

For dams with 5 to 15 m height, the following crest width are then recommended.

Table 7.10 4: Crest width recommended for small dams – bureau of reclamation/87 [3]

Dam Height (m)	Crest Width (m)
5	4
10	5
15	6

Many successful small dams have been constructed with crest less than 6 m wide. No dam should have a crest width of less than 3 m, because this is the minimum needed for an access road to permit maintenance works.
The French Guidelines for Small Dams observed that for zoned dams their width also depend on the number of zones at the crest. The following minimum widths are proposed, depending on their safety parameter H2 √V :

Table 7.10 5: Minimum crest widths proposed for small dams (french guidelines) [5]

Parameter H2 √V	< 100	100 to 300	>300
L minimum	3 m	4 m	5 m

Lewis, 2002 [4] observed that the crest width increases as the height of the dam increases, and presents the following empirical formula for crest width:

Crest width (m) = H0,5 + 1

Where H is the height of the crest above the bed stream. Table 7.10.6 presents the Australian recommended top width for dams of various heights based on the above formula.

Table 7.10 6: Crest width recommended for small dams (lewis, 2002)

Dam Height (m)	Crest Width (m)
4	3.00
5	3.25
6	3.50
7	3.65
8	3.85
9	4.00

In China, at the Hongshiyan homogenous dam, 12 m maximum height, in Hubei Province, a crest width 5 m had been used, as can be seen in Fig. 7.10.4.

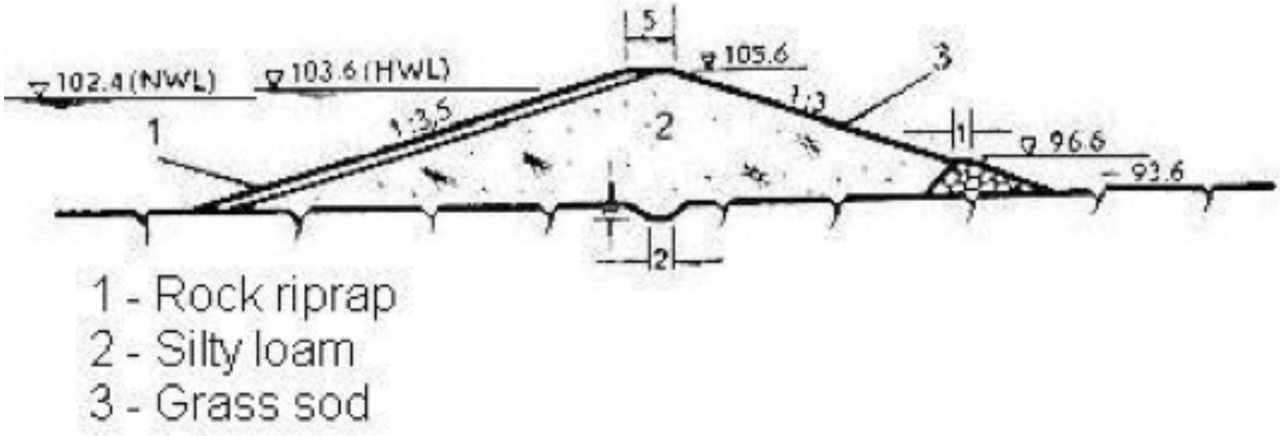


Figure 7.10 4: Hongshiyan homogenous dam in china

For ease of construction with power equipment, the minimum width should be 4.0 m.

7.11.2 Drainage

Surface drainage of the crest should be provided by a crown of at least 8 cm, or by sloping the crest to drain toward the upstream slope. The latter is preferred, unless the downstream slope is protected against erosion. In such a way the crest should be drained with a slope toward the reservoir (3% to 4% inclination).

7.11.3 Allowance for Settlement

Settlement in small dams is due to consolidation or saturation. Consolidation is the process of squeezing out the pore water by the weight of the embankment itself. Consolidation settlement can be settlement of the embankment or the foundation, and need special care when there are collapsible soils at the abutments, which usually occur in regions with tropical weather. The collapsible soils suffer sudden settlement when the soil layer, usually located at the abutments, are saturated during the reservoir filling period.

In general, most consolidation settlement takes place during construction, particularly with coarse grained (silty and sandy) materials. Consolidation settlement will continue for an appreciable time after construction when materials, such as highly plastic clays, are used. This is particularly evident when the materials are wet during construction. Under these conditions, consolidation settlement of up to 5% can be expected.

Poorly compacted soils may exhibit a sudden settlement when saturation occurs in response to seepage flow. This may lead to settlement of the soil below the seepage line. The joint between the settlement soil and the overlying unsettled soil is a common location of tunnel failure in dispersive soils. There is no simple method of predicting the amount of saturation settlement, for this type of soils. Settlement includes consolidation of both the fill materials and the foundation materials due to the weight of the dam and the increase moisture caused by the storage of water.

Settlement by consolidation depends on the properties of the materials in the embankment and foundation and on the method and speed of construction. The design height of earth dams should be increased by an amount equal to the estimates settlement.

7.11.4 Surfacing

Some type of surfacing should be placed on top of the crest for protecting against damage by wave splash and spray, rainfall, wind, frost and traffic when the crest is used as a roadway. The usual treatment for small dams consists of placing a layer of gravelly material or selected fine rock at least 10 cm thick.

In the south part of Brazil it is usual to use a granular material with some cohesion, to protect the small dam crests against erosion at a low cost. This material is composed by a blend of gneiss and limestone granular material with some percentage of bituminous shale, grading in such a way to form an adequate and appropriate material, used in the construction of rural roads, as can be seen in Fig. 7.10.5. This material has been used successfully for the crest protection of small dam crest and berms, in which the bituminous shale cause the gluing of the particles together, implicating in a good cohesion and avoiding erosion during rainy season, and also the developing of cracks during the dry season.

Figure 7.10 5: Constructing a rural road using a granular material composed by gneiss, limestone and bituminous shale.

7.11.5 Traffic Safety Requirements

When the crest of a dam is used as a highway, cable or beam-type guardrails are usually constructed along both shoulders of the crest. In other cases the crest can be lined with guard posts at 8 m interval or, on very minor structures, by boulders placed at intervals along the crest. If little or no traffic will use the crest, special treatment may not be necessary.

7.12 CONSTRUCTION TECHNIQUES FOR THE FOUNDATION AND THE FILL

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

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

- Avoid fine, cohesion less and uniform soil in saturated zones;
- Provide a chimney drain or enlarge it in the case of homogeneous fills;
- Design transition zones that are as wide as possible;
- Be specially prudent with the thickness of filters;
- Place a layer of sand upstream from the core to plug any cracks that may be caused by an earthquake;
- Build the dam or its core with materials offering good resistance to internal erosion (plastic clays, sandy gravel with fines of a very continuous grading).

In addition, as recommended by the French Guidelines, if the design earthquake intensity is very strong, it may be wise to increase freeboard and crest width.

8. SURVEILLANCE OF SMALL DAMS

8.1 INTRODUCTION

Surveillance is the continual examination of the physical condition and operation of a dam. Surveillance programs should be capable of detecting problems or unsafe conditions at an early stage so that corrective measures can be taken and dam safety is not compromised. Any unusual behavior, regardless of how seemingly insignificant, should be identified and recorded because this may be the forewarning of a newly developed unsafe condition. Each dam should have its own surveillance program. The scope of a surveillance program should be appropriate to the size of the dam and storage, the population at risk and other consequences of dam failure, the level of risk at the dam, and the value of the dam to the Owner.

A surveillance program should include:

- monitoring of instrumentation
- collection of information or data relating to dam performance (e.g. investigation, design and construction reports)
- evaluation and interpretation of the data
- a range of inspections, from routine inspections by operational staff through to comprehensive inspections by engineers.

Each of these is considered in more detail in the following sections.

8.2 MONITORING

Monitoring is the collection, presentation and evaluation of information from measuring devices installed at or near dams. Monitoring is needed:

- to detect deterioration in performance of the dam
- to detect trends or behavior to establish compliance with design expectations
- to rectify dam design issues which could not be resolved to high reliability during the design and construction stages

The designer, review engineer, or inspection engineer should identify the issues that need to be monitored and incorporate appropriate instrumentation into the dam.

Forms of monitoring include:

- deformation surveys
- water level measurements (including rainfall)
- seepage and pore pressure measurements
- measurements to confirm design parameters
- foundation pressure management
- stresses in embankments or structural components
- spillway performance and condition
- monitoring of deficiencies (eg cracking or erosion)
- seismic monitoring
- level of surveillance data.

The preferred frequency of monitoring varies over time. Factors influencing the frequency of monitoring include:

- the consequences of a dam failure
- the nature of the behavior being monitored
- the stage of maturity of the dam
- the existence of any problems or events.

8.3 DATA COLLECTION AND MANAGEMENT

There are two types of data:

- Static data does not change with time
- Dynamic data changes with time

Dam owners should ensure that the system used to collect and process the data has facilities to detect the occurrence of “obviously different” data, which can be caused by:

- data recording and transfer errors
- instrumentation malfunction
- Abnormal behavior of the dam.

These situations should be investigated immediately.

8.4 SURVEILLANCE EVALUATION

Not all dam deficiencies can be detected by visual inspections. For ease of understanding the data is evaluated on a regular basis to monitor the continued safety of each dam. Data evaluation should be assigned to an experienced dams engineer who should make recommendations based upon their interpretations. Following evaluation, a Surveillance Report should be prepared. Experienced dam engineers familiar with the entire history of the dam should prepare this report.

The Surveillance Report should:

- Review all dam safety inspections and surveillance data for a dam
- Identify any anomalous trends
- make recommendations on any actions required to ensure the continued safety of the dam
- Summarize and extend previous reports to provide a clear picture of long-term trends.

Anomalies and concerning trends identified in the Surveillance Report should be considered as deficiencies. It is the responsibility of the dam owner to ensure that appropriate remedial actions are taken and documented.

8.5 DAM SAFETY INSPECTIONS

One of the most important activities in a dam surveillance program is the frequent and regular dam safety inspection for abnormalities in conditions and deterioration of the dam. Dam safety inspections are conducted to determine the status of the dam and its features relative to its structural and operational safety. The frequency of dam safety inspections is tied up with the PCC of the dam. A frequency of inspection for various PCCs is given under Table 8.5-1.

Table 8.5 1: Frequency of Inspection

PCC	**Inspection type**			
	Comprehensive	**Intermediate**	**Routine visual**	**Special/ emergency**
High & very high	On first filling then 5 yearly	Annual	Daily	As required
Moderate	On first filling then 5 yearly	Annual to 2-yearly	Twice weekly to weekly	As required
Low	On first filling then 10 yearly	2-Yearly	Monthly	As required

Note: Dam owners may undertake a review to determine if a reduced or increased frequency of inspection is acceptable. The review should be carried out by an Approved Dam Engineer and take into account such matters as Regulator requirements, dam hazard and risk, type size of dam, dam failure modes and monitoring arrangements.

Different types of dam safety inspections should be undertaken for different purposes:

8.5.1 Routine Inspections

Purpose: To identify physical deficiencies of the dam.

The Standing Operating Procedures (SOP) should outline the requirements regarding:

- the timing and frequency of the inspections
- who should be involved
- the reporting requirements

8.5.2 Periodic Inspections

Purpose: Generally carried out by a dam engineer with the purpose of identifying physical deficiencies of the dam by visual examination and review of surveillance data against prevailing knowledge.

The timing of the inspection depends on the regional weather pattern.

8.5.3 Special Inspections

Purpose: The examination of a particular physical feature or operational aspect of a dam for some special reason.

8.5.4 Comprehensive Inspections

Purpose: A periodic inspection of the dam and a review of the owner's whole dam safety management program.

8.5.5 Regulatory Audits

Purpose: These audits will generally examine compliance with development permit conditions dealing with dam safety and the outcomes of inspections and Safety Reviews.

9. REHABILITATION PRACTICES FOR SMALL DAMS

9.1 INTRODUCTION

Aging of embankment dams, updating of design standards and criteria and the development of conditions affecting the safety of dams have resulted in a need for re-evaluation and, in some instances rehabilitation of dams.

The shortcomings are identified during the execution of safety inspections discussed in the previous section.

Even if finance is not readily available, rehabilitation or improvement is necessary to protect the asset of the owner of the dam, but also to protect the owner against claims caused by dam breaches. In this section techniques are presented for rehabilitation of embankment dams.

9.2 TECHNIQUES FOR IMPROVING SPILLWAY SAFETY

9.2.1 Introduction

A 1981 survey of non-Federal dams in the United States, concluded that 81 % had dam safety shortcomings because their spillways were not adequate to pass the estimated maximum design floods. This often reflects the difference between present-day design flood and the criteria in vogue at the time the dams were constructed.

Embankment dams are particularly sensitive to failure caused by overtopping, both during construction and while in service. Overtopping of a dam often causes dam failures. National statistics show that overtopping due to inadequate spillway design, debris blockage of spillway, or settlement of the dam embankment crests account for approximately 34 % of all U.S. dam failures. Embankments compacted to Standard Proctor density standards provide an elastic structure and show less cracking problems – often embankment dams are not compacted to this standard. Dam owners sometimes raise the spillway structures temporarily with say sandbags or definitely with concrete without realizing the effect on the safety of the structure.

In South West of France, a survey of small embankment dams has been carried out in 1997-1999 on more than 200 dam less than 20 m high [Lautrin, 2003]. 43 % of the spillways of those dams have been raised, ranging from 0,1 to 1,2 m and thereby reducing significantly the spillway capacity. The raising is obtained by means of a wooden beam (theoretically removable) or by means of a concrete beam.

The most frequent type of spillway used with small embankment dams, in South Africa, is a spillway cut from one of the banks of the river. Usually the material excavated is used for the fill in the embankment. This type of spillway is known as a by-wash spillway. A spillway can be located in a saddle adjacent to the reservoir if located at the proposed full supply level. These spillways can be lined or unlined. Normally the following problems are identified for existing by-wash spillways.

9.2.2 Spillway Capacity

Inadequate flood handling capacity not meeting design standards is normally caused by underestimating the design flood, not acknowledging the effect of the upstream approach channel on the hydraulic gradient or not ensuring that the dam was constructed with its total freeboard over the entire crest length of the dam. The hydraulic control must be correctly defined for the design flow by determining the hydraulic flow and energy lines based on the correct sectional information. Furthermore, the design floods associated with the classification shown in section 7.2 can be used as a guide.

9.2.3 Backward Erosion Control

Backward erosion of a spillway channel occurs due to fast flowing water over soil and weathered rockfill materials. An example is shown in Figure 9.2.1.

Figure 9.2 1: Backward erosion of the by-wash spillway of toleni dam in south africa

This may lead to undermining of the hydraulic control structure e.g. a concrete sill or a weir if that structure is also founded on erodible material. Backward erosion can be prevented by providing a control structure e.g. concrete weir founded on a solid unerodible foundation. Erosion can be controlled by limiting the depth and the velocity of the water depending on the type of gravel or soil medium between grasses in the case of grass lined channels. In arid areas care must be taken not to use grass as the grass will die and provide no resistance to erosion i.e. it will not be effective. A "better practice" is to protect the soil with a liner or rockfill as described in the following sections. The erodibility of spillways in unweathered to moderately weathered rock can be evaluated in accordance with water power and the rock strength using a chart (Van Schalkwyk, 1994). The following formula applies:

$$P = \rho w * Q1:100/b * s$$

Where

P = Power per unit area (kW/m2)

ρw = Density of water in kg/m3

Q = Discharge in m3/s

b = Width of channel in m

s = Slope of channel (m/m)

9.2.4 Spillway Chute or Spillway Return Channel Alignment

Change of direction in the spillway chute or spillway return channel during supercritical flow conditions provides problems regarding erosion of materials. Water in supercritical flow does not change direction and tends to carry on straight except if the floor is elevated correctly. Straight chutes for supercritical flow conditions must be provided.

9.2.5 Deformation of Spillway Structures Provided Over Embankments

It is possible to construct the spillway over the embankment, with two conditions:

Expected settlement of the foundation must be very low or must occur mostly during construction of the original embankment (short-term settlement).

Expected settlement of the original embankment must be very low, that means good compaction of the material.

A concrete spillway constructed over an earthfill embankment on soft foundations will therefore settle with movement of the concrete panels. Failure may be caused due to ingress of overflowing water and uplift of concrete panels. Although often expensive, (or costly) minimum risks and best solutions for the improvement of spillways include the "tried and tested" approach of location of spillways on solid foundations.

Nevertheless, construction of an additional spillway over the embankment of an existing dam may be a solution to consider, as settlement has often ceased after some decades.

9.2.6 Erosion of the Abutment Wall between the Embankment and the Spillway

The solution recommended for the transition between the embankment and the spillway is a concrete wall i.e. mass gravity or cantilever (depending on foundation, height and economics).
If the protection of an earthfill abutment (or the training wall of one side of a spillway return channel) comprises a layer (or layers) of rock (or rip-rap), it may be damaged (or washed away) after cracking of soil and erosion by storm water or high powered flowing water. Materials used for this type of erosion protection must be carefully designed to accommodate the unit power of the flowing water. The protection must also be designed to accommodate the safety evaluation check discharge.

9.2.7 Gabions Used as Sill in by-pass Channels

Gabion baskets must be considered with care as they have shortcomings:

Very often the galvanized steel wire that makes up the rectangular (or square) panels of the baskets corrodes and results in individual stones in the baskets being exposed to flowing water. Flowing water with high power (kW/m2) can remove stones if too small. Where there has been poor maintenance, total breakdown of a Gabion will take place if most of a panel has corroded and "broken down" before repair (or rehabilitation to its original functional condition)

- Stones and the "upper panels" of Gabions may be removed by vandalism.

A cut-off structure connected to rock should be considered.

9.3 TECHNIQUES FOR IMPROVING EMBANKMENT SAFETY

9.3.1 Techniques to Overcome Flood Handling Problem

9.3.1.1 Parapet Wall

A parapet wall can be used to raise the crest of a consolidated embankment crest. The concrete parapet must be designed to accommodate water loads including uplift but the safety factors against overtopping and shear failure if exposed to the PCC 3 (Potential Consequences Classification) floods can be selected close to 1 in this case. An impermeable membrane must be designed between the impervious core of the embankment and the base (or underside) of the parapet wall. The footing normally have to be imbedded into the embankment crest.

9.3.1.2 Raise Embankment Crest

Generally (but not always) the lowest cost option to overcome inadequate flood handling problem is to increase the total freeboard of the dam by raising the crest of the embankment. This is normally achieved by adding earthfill to the downstream face of the embankment starting from the toe of the embankment (see Figure 9.3.1). In addition to the fact that the spillway must be designed to accommodate higher "recommended design and safety evaluation check floods", respectively, the three (3) topics that are listed below must be addressed in the context of the structural integrity of the embankment:

The stability of the downstream slope (and "in-situ" foundation) must be analyzed taking into account that the existing embankment (or portions thereof) are structural zones (or components) of the raised embankment. The engineering properties of the earthfill materials of the existing embankment and the "new" earthfill section should be determined with reasonable confidence (i.e. a representative (or appropriate) sampling and laboratory testing program) and used in any analyses.

A "sand chimney drain" must be considered at the interface between the downstream face of the existing embankment and the "new" earthfill section (or zone). This will draw down the phreatic surface and intercept any possible seepage through the embankment.

A new toe drain possibly connected to the existing toe drain must be considered.

An example is shown in Figure 9.3.1.

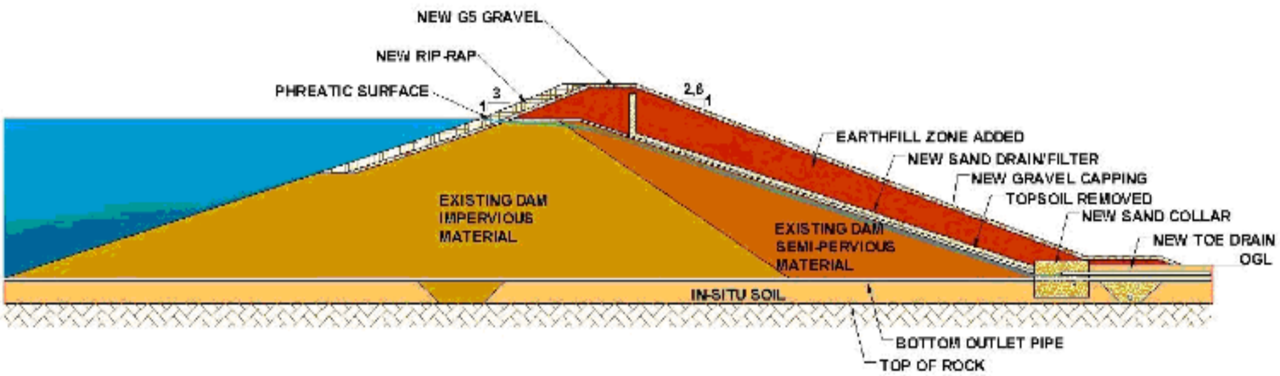


FIG. EXAMPLE OF RAISING OF EMBANKMENT :CROSS SECTION AT BOTTOM OUTLET

Figure 9.3 1: Section through an embankment showing rehabilitation with a positive effect both on stability, piping control and seepage control

Furthermore, to extend an earthfill core a High Density Polyethylene Sheet may be used to raise the core by anchoring it in a trench excavated in the top of the core. This method is especially applicable where ice conditions occur where the top of the core can be damaged by low temperatures.

9.3.1.3 Protection of Embankment from Erosion During Overtopping

Erosion as a result of overflow during flood events is a principal cause of embankment dam failure. Consequently, the approach to solving this problem has been not to allow overtopping. Preventing overtopping of existing dams to accommodate today's current estimates of inflow floods often requires costly modifications to the spillway or raising of the embankment. For a large number of potentially unsafe dams with inadequate spillway capacity, permitting overtopping during large or infrequent floods would result in significant benefits.

If overtopping of an embankment is allowed, the owner of the dam must ensure that the downstream slope is sufficiently protected to prevent erosion. Two erosion protection systems i.e. articulated concrete blocks (ACB) and roller compacted concrete (RCC) are often used successfully for the protection of the downstream face of embankments during floods.

(a) Articulated Concrete Blocks

An auxiliary spillway constructed with small cable tied concrete blocks system laid on a geotextile base is capable of withstanding velocities less than 8 m/s and provides a low cost alternative to reinforced concrete, as can be seen in Figure 9.3.2 and 9.3.3. Care should be taken to design for lower velocities of up to maximum 5 m/s (say) to accommodate deteriorating (or ageing) conditions during the life of the dam.

Figure 9.3 2: Installation of the articulated concrete blocks in a channel

Figure 9.3 3: Lateral slopes protected with concrete blocks and grass

This is the primary conclusion drawn from a series of tests undertaken in the UK by the Construction Industry Research & Information Association (CIRIA). The research has been prompted by the Reservoir Act 1975 which requires local authorities to enforce safety provisions on reservoirs with capacities greater than 25 000 m3.

These trials have also attracted interest from all over the world with inquiries coming from Australia, Israel, Peru and Chile, and, most interestingly, the US Bureau of Reclamation (BuRec) which is facing similar upgrading problems to the UK.

Figure 9.3 4: Testing under way on the left-hand of the ten channels at jackhouse reservoir near blackburn, uk

Figure 9.3 5: One of the cable-stayed block systems remains totally intact after a water test

A cross section of the dam overtopping protection with the ACB (Articulated Concrete Block) system and the downstream basin is shown in Figure 9.3.6

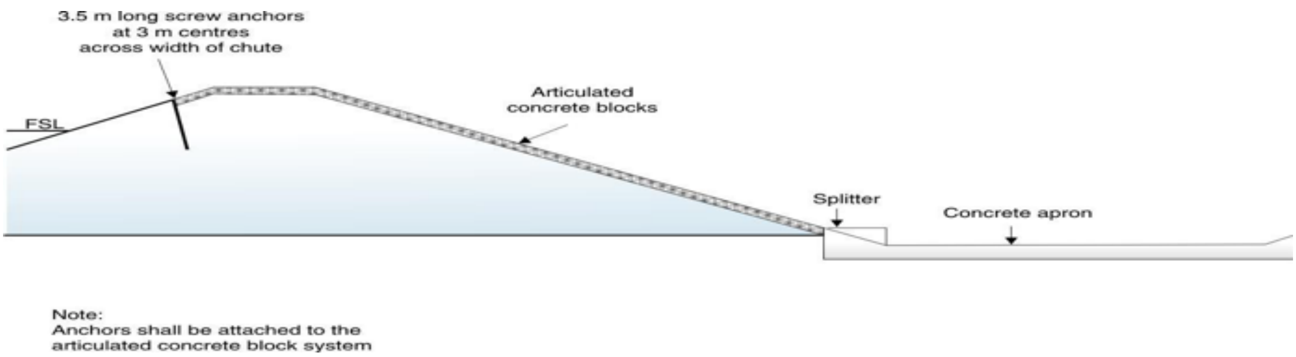


Figure 9.3 6: Dam overtopping protection with the acb system and the downstream concrete chute blocks. (Anderson and henrich/2005)

(b) Roller Compacted Concrete – RCC

Roller-compacted concrete (RCC) has been used as a spillway or overtopping protection on consolidated embankments for more than 130 dams in US as pointed out by Abdo and Adaska (2007), over the past 27 years. Primary reasons for the popularity of RCC with designers and owners are simplicity, speed of construction, strength and durability, and economic advantages compared with alternative methods. The unit discharge is normally limited to a maximum of 20 m2/s.

Comparative tests on soil-cement, RCC, and conventional concrete showed RCC to have a greater resistance than conventional concrete of higher strength, primarily because of a greater percentage of aggregate in the mixture and less paste. Several U.S. Army Corps of Engineers research projects have confirmed the excellent abrasion resistance and durability of RCC, as can be seen in the two following examples i.e. Lake Tholocco Dam and Red Rock Detention Basin Inlet Spillway.

Lake Tholocco Dam (USA)

Constructed in the 1930's, Lake Tholocco Dam is an earth embankment owned by the US Army and located in Fort Rucker, Alabama, US. The dam is 732 m long with a maximum height 13,7 m, in which the service spillway is 15,2 m long reinforced concrete structure with a fixed ogee crest. A 1979 Phase Inspection Report under the National Dam Safety regulations showed that the dam did not meet current standards due to insufficient spillway capacity. Since its construction the dam's earthen emergency spillway was operated regularly, causing severe erosion. Major storms in the 1990's breached the emergency spillway twice. The emergency spillway failed for the second time in July 1994 during tropical Storm Alberto. After that the reservoir remained empty for six years.

The USACE Mobile District investigated several upgrade alternatives and concluded that the most cost-effective solution would be to construct an RCC auxiliary spillway with a collection channel in the embankment adjacent to the reinforced concrete service spillway. Approximately 19 900 m3 of RCC were placed in the spring of 2000. The RCC mixture contained 163 kg/m3 Portland cement and 30 kg/m3 fly ash.

On-site USACE personnel reported that the spillway has been overtopped at least twice. The first time was during hurricane Ivan on September 16, 2004, and the second on March 27 and 28, 2005. Figure 9.3.7 was taken during this overtopping event. Figure 9.3.8 was taken in May 2007 and shows the excellent condition of the RCC steps after the two overtopping events.

Figure 9.3 7: A view of principal and auxiliary spillways at lake tholocco dam

Figure 9.3 8: Excellent condition of rcc steps at tholocco lake dam (Abdo and adaska/07)

Red Rock Detention Basin Inlet Spillway (USA)

Red Rock Detention Basin, in Nevada, is one of five detention basins and is part of a master plan for providing flood protection and erosion control in Las Vegas Valley. It's complete construction was in 2001, including an RCC inflow spillway, a holding reservoir, and three outflow spillways (service, auxiliary and emergency).

The RCC structure consists of an approach apron starstepped chute, stilling basin and training walls. The spillway is 12,6 m high, and the slope of the chute is 3H:1V. The steps are 0,6 m high. The RCC mixture contained 216 kg/m3 cement and 44 kg/m3 fly ash.

The RCC spillway operates during every rain event generally a flow in the wash, normally a few times a year. Figure 9.3.9 shows the inflow water during a 2004 storm. The storm carried heavy sediments loads, evidenced by the presence of a large pile of sediment in the basin of the RCC structure. In 2005 a strong storm deposited boulders up to 91 kg on the RCC steps and distributed the sediment pile throughout the detention basin (Figure 9.3.9).

Although RCC is still a relatively new method of construction for this application (i.e. spillways constructed on embankments or overtopping protection embankments) performance data is limited. Structures that have been overtopped show strong evidence that the RCC has performed satisfactorily when subjected to hydrostatic pressures and flows containing very abrasive sediments, as reported by Abdo and Adaska in 2007.

In addition to proper structural design, the primary factors that have contributed to the successful performance of these structures are related to the RCC mix design and sound construction methods, namely;

Figure 9.3 9: 2004 Storm overview at red rock detention basin spillway (abdo and adaska, 2007

Figure 9.3 10 Boulders deposited on the rcc steps during 2005 storm (abdo and adaska, 2007)

- Proper mix proportioning. This includes the use of a well-graded aggregate to ensure that the volume of coarse aggregate in the mix is minimized without segregation while still providing an adequate amount of paste;
- Use of the hardest aggregate available;

- Sufficient cementitious content in the RCC mix, proper compaction to achieve adequate strength and high density, especially where the RCC is subjected to repeated freeze-thaw cycles or frequent overtopping;
- Forming and high-density compaction of the steps to limit erosion at the exposed edges of lifts; and
- Proper bonding of RCC lifts, especially at the upper few lifts and where energy dissipation occurs in the vicinity of the stilling basin.

9.3.2 Slope Stability

A dam embankment must be structurally stable and the materials must have adequate strength and be sufficiently impermeable to prevent slope failure. The method of rehabilitation and design of the final cross section of the dam for rehabilitation should be decided by means of a comprehensive evaluation of the engineering properties of the embankment materials, the configuration of the existing dam, the foundation of the dam, geology, and examples of the rehabilitation of similar dams etc.

The rehabilitation of an existing dam can include repairing the existing embankment, widening of the embankment at the level of the downstream toe to crest level (and above if additional freeboard is required) discussed in Section 8.3.

9.3.3 Piping Control Measures

The remedial measures to be adopted to control seepage depend on many factors including quantity and rate of change of flow, phreatic levels, dam configuration, zoning and foundation conditions. The objectives of the remedial action are to reduce seepage flow and to control and prevent piping. In some cases, the downstream face or foundation area downstream of the dam can be treated to control piping, but not reduce seepage. Impervious materials should never be placed against an area of seepage downstream from the impervious zone of a dam because excessive pore pressures or uplift and a reduction in stability would result. To control piping, filters must be placed against the affected areas to prevent migration of soil, and free-draining materials should be placed against filters to convey seepage water. The grading (or particle size distribution) of the filter and drain zones are determined by strict adherence to established filter criteria.

In an emergency situation when large quantities of turbid water emerge from the dam or foundation, the first response is to lower the reservoir as rapidly as possible. At the same time three remedial actions should be attempted: (1) bulldoze large volumes of soil, gravel and rock in the vicinity of the source of leakage in the reservoir if indicated by vortices, (2) add bales of straw on the upstream side and (3) dump filter and free draining material onto the areas of discharge.

Increasing flow through abutment or foundation materials should be viewed as a serious problem that can worsen with time. Increasing flow, especially if turbid, indicates loss of supporting materials. Erosion of pervious foundation/abutment layers, leaching of soluble layers (e.g. calcium carbonate or gypsum), or subsurface collapsing due to breakthrough into caverns can cause the overlying embankment to undergo differential settlement and cracking. Seepage at the foundation embankment interface is especially serious as loss of embankment material by piping could result.

If seepage rate is increasing and piping is suspected, the first follow-up action is to trigger emergency plan and to start lowering the reservoir and follow the steps required by the emergency action plan.

9.3.4 Seepage Control Measures

Most embankment dams experience seepage to some degree. Under certain conditions, excessive, or even moderate uncontrolled seepage can lead to progressive internal erosion or piping of the embankment or foundation materials. There are many case histories that describe seepage and internal erosion which went undetected for significant time periods before the problems were recognized and seepage control measures implemented as remedial measures (or rehabilitation).

It is sometimes difficult to determine how, or if, seepage is adversely affecting the safety of a dam, and whether or not remedial measures are required to control seepage. Monitoring tools and techniques for detecting potential seepage problems are covered in other sections of this report. Once it has been determined that a dam requires modifications to control seepage, the options that are available fall under two broad categories, as follows:

Seepage cutoff or barrier systems (reduce quantity of seepage);

Seepage interception and conveyance with engineered filters and drains (filters prevent movement of soil particles under seepage forces; and drains relieve excessive pressure, intercept seepage pathways, and provide for safe collection and conveyance of seepage from the dam, its foundation and/or its abutments).

It is common to use those two categories of solutions in combination.

9.3.4.1 Seepage Barriers and Cutoff Systems

This section provides a brief overview of commonly used seepage control systems. Seepage cutoffs and barriers are intended to reduce the amount of seepage and minimize downstream pressures and exit gradients. Depending on the site and foundation conditions, seepage cutoff systems may provide either:

- Positive cutoff that completely penetrates pervious zones and ties into impervious soil layers or bedrock in the dam foundation and abutments, or
- Partial cutoff, which is sometimes used for deep pervious foundations to lengthen the seepage path and reduce exit gradients to acceptable levels.

There are many different types of cutoff systems that are used for dam rehabilitation. The primary objective is to provide a low permeability element within the dam and/or its foundation, either as the primary seepage barrier or to supplement existing barriers. Cutoffs may be classified according to their stiffness, the type of materials used, or construction methods used.

The terms cutoff “wall” or “diaphragm” are often used to distinguish seepage barriers that are thin compared to the surrounding embankment. Barriers employed for dam rehabilitation may be constructed through the dam and foundation from the existing dam crest (diaphragm or wall).

Alternatively barriers may be constructed using a liner system on the upstream face of the dam in combination with vertical or horizontal seepage barriers extending from the upstream toe.

Some of the seepage barriers that are commonly used for dam rehabilitation are listed in Table 9.3.1 .Most of those solutions require lowering of the reservoir level significantly, in order to reduce hydraulic gradients during rehabilitation and the risk of wash-out of cement and bentonite. In case of construction of rehabilitation works on the upstream face of the dam, the reservoir must be emptied.

Table 9.3 1: Common seepage cut-off measures identified for alluvium and earthfill foundations of small dams

Type of cutoff	**Method**	**Materials used**	**Remarks and References**
Core trench:	Open trench is excavated and, if stable, filling with one of the materials is done	Earthfill Soil-bentonite Concrete-bentonite	Depth limited to some metres
Slurry trench: Slurry wall, concrete wall, tremmy	Open trench by extended backhoe, kept open and stabilized with bentonite slurry	Concrete by tremmy pipe. Primary and secondary panels are inserted once sufficient strength has been identified	
Sheet piles curtain	Inserted by vibratory and hydraulic pile drivers Steel	Steel	Curtain not taken to rock foundation i.e. a non-positive cutoff.
Tube-a-manchettes injection system for grouting	Drilling with special packer rod tube and grouting through rubber sleeve	Cement	Only for rock foundation
Grout curtain	Drilling holes, primary, then secondary and tertiary holes. Grouting of holes with cement grout.	Cement	Only for rock foundation

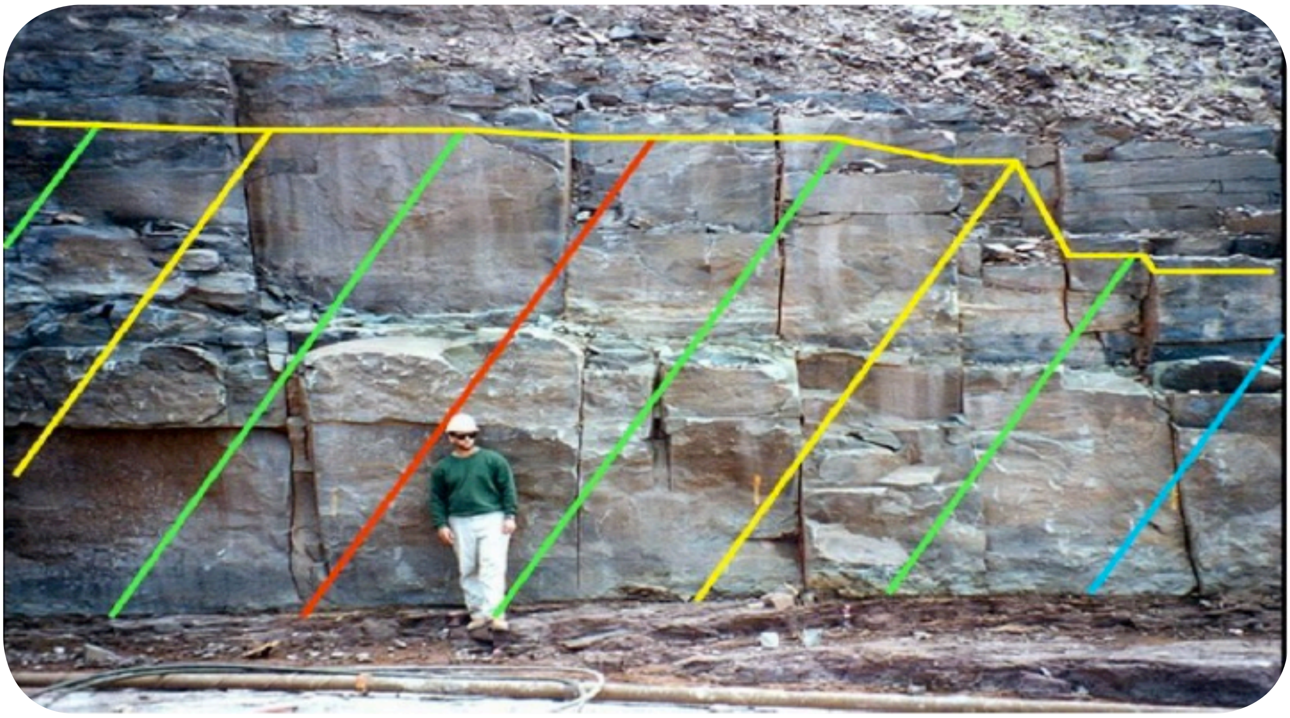


Figure 9.3 11: Vertical rock joints shown with angled grout hole orientation (Penn forrest dam rehabilitation, usa)

Figure 9.3 12: Grout holes are drilled at optimized angles for rock joints patterns (image from www.Advancedconstructiontechniques.Com/).

9.3.4.2 Seepage Interception and Exit Control With Filters and Drains

Filters are typically the first line of defense against piping and internal erosion. Filters and drains are often incorporated as remedial systems to control seepage in existing embankment dams.

For small embankment dams that can be dewatered, it is possible to install central vertical filters in open trenches. In the southwestern US, for example, hundreds of small homogeneous earth dams were constructed for flood control under a federal government watershed protection program that was initiated in 1954. Many of these flood control structures were subsequently found to have developed severe cracking caused by desiccation shrinkage, differential settlement, and collapse on inundation of metastable soils in the dam foundations. However, recognizing the potential risks associated with piping and internal erosion failure modes in cracked homogeneous embankment dams, NRCS retrofitted a large number of the cracked dams with central sand and gravel filters, as shown on Figure 9.3.13. NRCS also performed landmark research studies that advanced the general understanding of the performance and design requirements for granular filters (Sherard et al., 1984a, 1984b; Sherard and Dunnigan, 1985, 1989).

For water retention dams, or dams that are too high for open trenching from the crest, the filter and drain zones can be constructed as weighted berms and wedges of fill on the downstream side of the dam.

Figure 9.3 13: Central sand filter installed by open trenching from the crest of an existing low-height flood control dam in southwestern usa

9.3.5 Slope Protection Measures

9.3.5.1 Vegetal Protection

Grass protection is commonly used in very wet areas where sufficient rain occurs for good maintenance of the grass.

Climate and soil features have a major influence on the establishment, growth and survival of various grass species. Temperature influences evapotranspiration, seed germination and plant growth. The main soil features to be considered when selecting a suitable grass species to be grown are water holding characteristics, chemical properties and erodibility.

Grasses can be divided into annual and perennials. Annuals die off in summer each year and re-establish in the autumn from seeds which had set in the previous spring or summer. Perennials keep growing from one year to the next.

Grasses can be divided into turf forming (Kikuyu, Couch) and bunch grasses (Phalaris). A uniform, turf forming grass having a denser relatively deep root system provides the greatest protection against scour. Permissible velocities for bunch grasses and other non uniform grasses are lower because bare soil is likely to occur between plants and because they cause more disturbances to the flow.

9.3.5.2 Rip-rap or Gravel Protection of the Downstream Face of the Embankment

Low maintenance gravels (normally a 200 mm – 300 mm thick layer of -75 mm gravel) are used in South Africa to protect the surface from erosion damage caused by storm water.

9.3.5.3 Techniques for the Rehabilitation of Bottom Outlets on Embankment Dams

This section summarizes some preferred methods for rehabilitating or abandoning deteriorated bottom outlet conduits. Techniques that are considered "best practices" for rehabilitation of conduits through embankment dams are presented in a comprehensive technical manual that was prepared by the US Federal Emergency Management Association (FEMA 2005). A DVD version of the manual is available which also contains a bibliography with links to most of the cited references contained on the DVD for access by the user. Many of the same rehabilitation techniques described in the FEMA manual for conduits through embankment dams also are appropriate for conduits under concrete or masonry dams.

9.3.5.4 Renovation of Conduits

Due to the potentially high costs and construction difficulties associated with replacing aged and deteriorated outlet conduits, a variety of techniques have been developed for renovating existing outlet conduits in place. Renovation requires that the existing pipe be structurally sound, although hydraulic performance may be impaired due to cracking, spalling, corrosion or other deterioration of the conduit (US Army Corps of Engineers, 2001). Consideration of the age of the structure may be very important in selecting an approach to renovation of outlet conduits. For example, structural design standards have changed, and improvements have been made in steel reinforcing and concrete materials since reinforced concrete pipes were first used in dam engineering in the US beginning in the early 1900's, as shown in Figure 9.3.14.

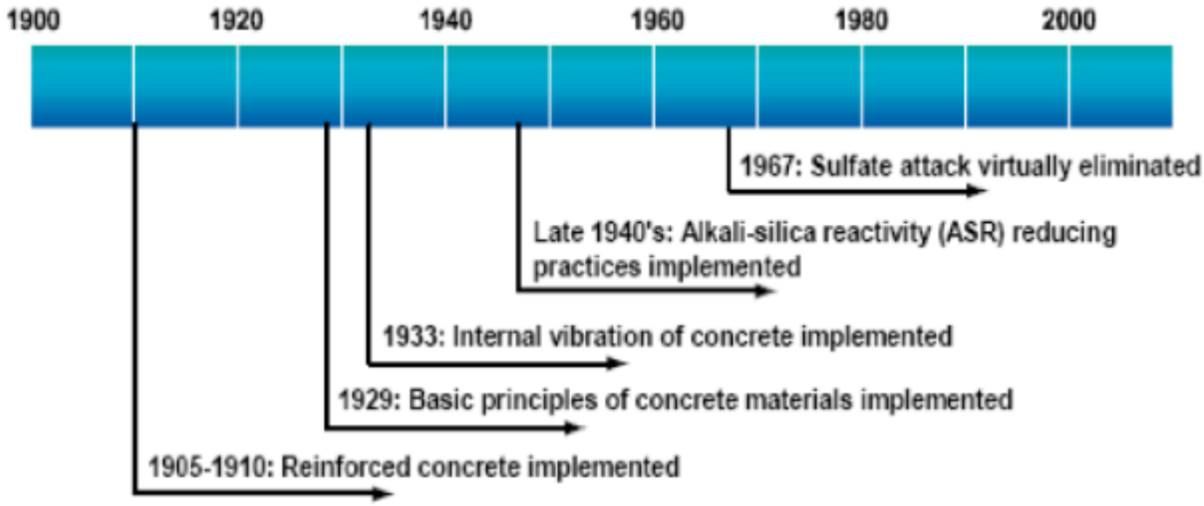


Figure 9.3 14: Example of the historical timeline used by us bureau of reclamation to assist in the structural evaluation of existing outlet conduits (from fema, 2005)

Removal and replacement of defective outlet conduits is the traditional method for rehabilitation. However a variety of "trenchless" technologies are now available for renovating deteriorated outlet pipes. Table 9.3.2 summarizes some of the various trenchless technologies, the conditions under which each method is appropriate, and the associated pros and cons.

Sliplining is one of the most common methods used (Figure 9.3.14). Understanding the existing seepage conditions around a defective outlet is a critically important consideration when evaluating whether or not sliplining should be attempted.

If the control valve is located upstream of the conduit, the conduit is not always pressurized and, if not watertight, can play an important drainage role for the surrounding embankment. In such a case, rehabilitation of the pipe by sliplining may alter the seepage flow regime and in some cases actually exacerbate adverse seepage conditions around the outside of the pipe. Figure 9.3.15 shows seepage conditions prior to sliplining, when seepage through the dam embankment is entering the defective (or leaking) conduit. After sliplining, drainage into the pipe is prevented, and pore pressures within the embankment may increase.

It is important to provide adequate filter protection around the outside of the pipe near its downstream end to intercept seepage around the outside of the pipe. Further details for evaluation, design and construction of sliplined conduits are provided in FEMA (2005).

TABLE 9.3-2: TRENCHLESS TECHNOLOGIES (USACE 2001) REFERENCE GUIDE FOR TRENCHLESS TECHNOLOGY

		Conditions	Pros	Cons	Comments
Pipe Rehabilitation	SLIPLINING	1. Unblocked and have some structural capacity. 2. Any pipe diameter and circular shape.	1. Simplicity of the technique (insertion of one pipe in another) 2. It can be accomplished on-line without diverting existing pipeline flow.	1. Reduction in existing pipeline diameter. 2. The need for insertion and receiving excavations. 3. Normally, annulus grouting is needed between the sliplining and the existing pipeline.	This technique is useful for renovating pipeline with deteriorated interior walls which have reduced flow capacity. This method is also useful for renovating structurally unsound pipelines.
	MODIFIED SLIPLINING	1. Unblocked and having sufficient structural capacity to aid the new lining. 2. Up to 1 600 mm (63 in.) dia for PE pipe and up to 330 mm (13 in.) dia for PVC pipe.	1. It is a close-fitting pipe liner that helps to minimize existing pipe diameter reduction. 2. Annulus grouting is eliminated between the liner and existing pipeline.	1. It does not hold up against high external loads from structurally unsound pipe. 2. The expanded spiral wound method allows an influx of water between the new sliplining walls and the existing pipe walls.	This technique is useful for renovating deteriorated pipeline that has remaining structural capacity. It is also recommended where manholes exist, so that insertion and pulling excavations may be eliminated.
	SPRAY LINING	1. Unblocked, very clean, and structurally sound/ 2. Almost any diameter, and most commonly circular shape, with rare exceptions.	1. It inexpensively provides corrosion protection for iron pipes. 2. Decreases pipe wall roughness in deteriorated iron and concrete pipe.	1. It is only useful for iron and concrete pipelines. 2. It is only useful for corrosion protection, providing little structural enhancement.	This technique is useful for corroded iron pipe interiors and deteriorated concrete pipe interiors. It is recommended mostly for iron pipes because of corrosion reduction.
	CURED-IN-PLACE LINING	1. Unblocked, dry pipe walls, some structural capacity to aid new lining. 2. Almost any diameter, yet larger diameters become less cost effective, can accommodate non-circular shapes.	1. It provides renovation of existing pipelines with minimal diameter reduction. 2. Pipeline hydraulic capacity is most likely increased. 3. Renovation timing can be varied using optional curing techniques.	1. The existing pipeline will need to be blocked during the renovation. 2. Traffic management will likely be required because of on-site support vehicles. 3. Closed-circuit television inspection will be required immediately prior to renovation.	This method is useful in reducing excavations, if not eliminating them. This method is most useful for existing pipeline with remaining structural capacity, yet deteriorated interior pipe wall.

Destruction and Replacement	PIPE BURSTING AND PIPE SPLITTING	1. May be partially blocked; existing structural capacity should be less than the bursting forces. 2. Common diameter is 305 mm (12 in.), yet up to 1 194 mm (47 in.) diameter is possible.	1. It can replace existing pipe with larger diameter pipe. 2. Existing pipe is broken and left surrounding the new pipeline.	1. This method must be modified by adding pipe splitting techniques to repair non-brittle pipe. 2. This technique may not burst reinforced or previously repaired pipelines.	This method is most useful for increasing existing pipe diameters. If they are deteriorated or not. Careful consideration should be taken in determining if this technique will be able to burst the existing pipe.
	PIPE EATING	1. Useful for deteriorated and unreinforced clay, partially reinforced concrete, and highly deteriorated ductile iron pipe. 2. Any diameter.	1. It allows for upsizing existing pipe diameters. 2. Existing [pipe materials are removed from the ground. 3. Realignment of the newly installed pipeline is possible.	1. Only old clay pipe without reinforcing, asbestos concrete with or without mesh reinforcement and some highly deteriorated ductile iron pipe may be replaced. 2. Substantial insertion and	This technique is useful for eliminating the need for open-trench pipeline replacement for select existing pipe types. Where applicable, this technique can be very useful in upsizing and realigning a new pipeline.
New Pipe Installation	HORIZONTAL DIRECTIONAL DRILLING	1. Appropriate for flexible polymer pipe installation. 2. Useful for almost any diameter pipeline.	1. Open trench and pit excavations may be eliminated altogether. 2. Flexible pipeline, drilling machine, and monitoring instruments allow for controlled direction and depth during installation.	1. Some drilling machines require excavations to place the machine at the desired pipeline depth. 2. Only flexible pipes, such as polyethelene or steel, may be installed with this method.	This technique is most useful for flexible-type pipe, which commonly is used for cable and electric lines. Gravity pipelines are less common, but are becoming more common with the use of direction and depth monitoring systems.
	MICRO-TUNNELING	1. Appropriate for high compressive strength pipe types only, such as clay, reinforced fiberglass, and concrete pipes. 2. Useful for any diameter, 152 mm (6 in.) or greater.	1. It can perform installation of pipe at great depths and tunnel through various soils and rock sizes. 2. Gradient and alignment of newly installed pipe is precisely controlled.	1. Substantial excavations will be required for insertion and extraction of the micro-tunneling machine. 2. Unexpected ground conditions can lead to a blocked machine, requiring an expensive removal.	This technique is useful for situations where open-trench installation of pipe is not economical or possible. Also, it is very useful for installation of gravity pipelines that require strict control of gradient and direction.

Figure 9.3 15: Example of sliplining – insertion of plastic (high density polyethylene) pipe into larger diameter concrete pipe (from fema, 2005)

Need filter at downstream when sliplining Seepage before slipliner Seepage after slipliner (without filter) (with filter) Filter diaphragm

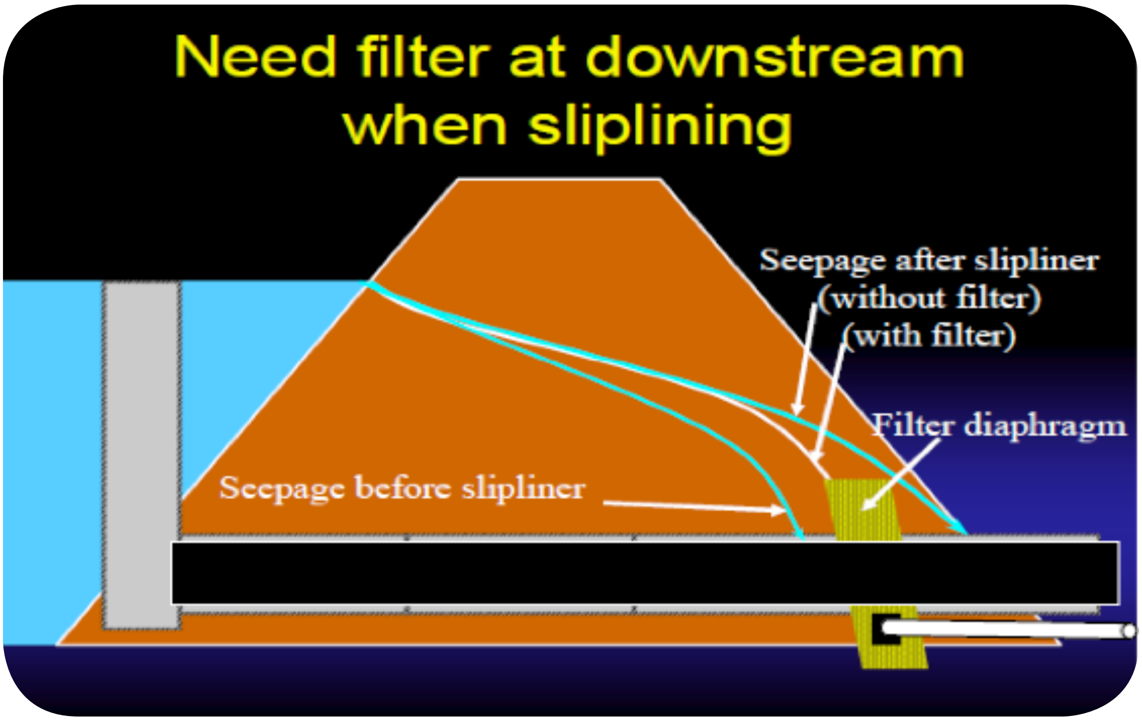


Figure 9.3 16: Possible effect of a slip-liner on pore pressure in an embankment dam, illustrating the need for a downstream filter diaphragm (courtesy of maryland (usa) dam safety division)

9.3.5.5 Filter Around Conduits

Filters are generally accepted as the preferred method to protect against seepage or leakage around the outside of conduits that penetrate the dam or its foundation. Use of anti-seep (cutoff) collars on conduits is no longer considered acceptable practice due to the high incidence of historic failures attributed to poor compaction control around these protrusions on the conduits. Filters used in conjunction with conduits generally fall under three categories: chimney filters, filter diaphragms, and filter collars. Detailed design recommendations for filters around conduits are provided in FEMA (2005), and some key points are summarized in this section. Embankment dams that have properly designed and constructed chimney filters normally do not require a separate filter zone to control seepage around the outlet conduit. When full hydraulic height filters are installed as a seepage remedy for the embankment (see Figure 9.3.16 and Figure 9.3.17, for example), care should be taken to incorporate the filter zone around the outlet conduit as well.

A filter diaphragm is a zone of engineered filter sand placed around the conduit to specified dimensions. The dimensions shown in Figure 9.3.17 are based on guidance provided by NRCS (1989, 1990). Normally a minimum width of 1 m around the conduit is provided. Other details for positioning and designing filter diaphragms are provided in FEMA (2005). Filter diaphragms are economical and effective seepage rehabilitation features that can be incorporated on dam embankments that do not have chimney filters, or which may have defective filter zones.

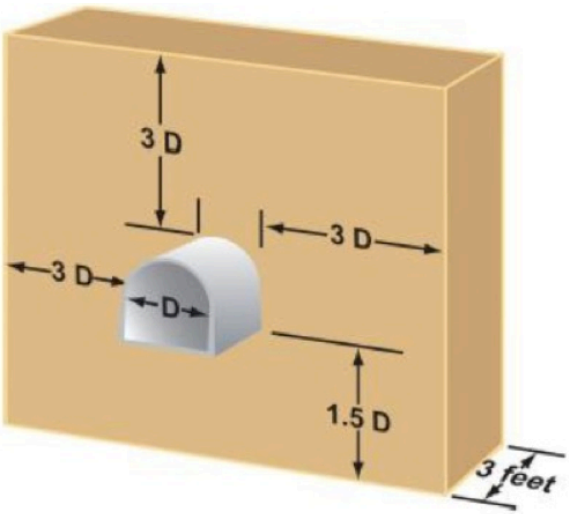


Figure 9.3 17: Typical configuration of a filter diaphragm for conduits through embankment dams (after fema, 2005; nrcs 1989, 1990)

A filter collar is a zone of filter material (typically sand) that completely surrounds a specified length of the outlet conduit. The lateral dimensions beyond the outside of the conduit are limited in extent compared to a filter diaphragm. It is recommended that filter collars be used only for special conditions when the only seepage considered likely is flow around the periphery of the pipe, and soil conditions are less conducive to erosion.

Sometimes it is difficult to construct the collar in the "downstream zone" of an embankment when the reservoir is full, or partially full. In such cases the conduit pipe can be lengthened and covered with a berm of soil on the downstream side of the dam. Special attention must be paid to the control valve at the downstream end of the existing outlet pipe if it is to be left in place "as a spacer" so that sufficient space is added for the connection by bolts.

9.3.5.6 Pipe Abandonment

Outlet pipe must sometimes be abandoned, such as where the conduit has deteriorated to the point that it is no longer safe to operate. It is often considered technically and economically more feasible to abandon the conduit in place, rather than excavate and remove it. If it is necessary for an "abandoned conduit" to remain in place in the dam or its foundation, the conduit must filled with grout. A filter diaphragm should be installed around the grouted conduit to protect against possible seepage (or flow) around the periphery of the grouted (or sealed) pipe/conduit. Some considerations relating to the abandonment of conduits include (FEMA, 2005):

Complete or partial filling of the conduit with grout: Normally, complete backfilling of the pipe with grout is recommended. However, it may be difficult to completely fill the pipe full of grout if the pipe is long and access is limited to one end of the pipe. It may be possible to access the pipe by drilling through the dam from the crest to intersect the conduit.

There are inherent risks associated with hydraulic fracturing both from drilling into the dam and injection of grout from the crest of the dam. These potential risks must be weighed against the desire to completely fill the pipe with grout. If drilling into the dam is necessary to fill the pipe, the preferred method is dry augering to avoid risk of hydraulic fracturing due to drilling fluids.

If water is flowing into the conduit and cannot be stopped by lowering the reservoir, it may be necessary to seal the pipe with a temporary bulkhead or inflatable bladder.

9.3.6 Techniques for Rehabilitation of Hydromechanical Components

All valves and hydromechanical equipment used for releasing water from a dam must be properly maintained. It is important to remember that under certain circumstances an unsafe condition could develop at a dam and there may be a requirement to empty the reservoir (or draw down the reservoir to a "safe level") as quickly as possible. It is important to service and exercise the valve and other mechanical components regularly to ensure that they are operational at all times. For example, it is recommended that for a dam with basic outlet works, that the control valve be operated at least four times per year to make sure it is functioning properly.

If a valve is difficult to operate, for example a tight handwheel, or one which requires excessive force to turn it for its full range, this is a clear indication of a developing problem that requires attention. It is possible to exercise a valve without losing water stored in the reservoir when the spillway is flowing because water is being passed downstream anyway. This "mode of operation" is recommended at the tail end of flow over a spillway because opening a valve at the beginning of spillway discharge may worsen the natural increase in discharge downstream of the dam.

Figure 9.3 18: What are the chances that this valve will work in an emergency? (Smec/2006)

Provision should be made at the discharge end of an outlet pipe to reduce the velocity of the water and to dissipate energy. Heavy (and suitably dimensioned) rock placed on a layer of crushed rock will minimize erosion. Construction of an effective concrete outlet structure e.g. "Impact-Type Stilling Basin" (Design of Small Dams, 1987) will dissipate the energy of the water and minimize erosion. Design of suitable erosion protection (or an energy dissipator) will depend on the flow (m3/s) and the pressure head of the water.

9.3.7 Restoring reservoir capacity in case of siltation

During reservoir operation, siltation may become so big in volume and adversely affect reservoir storage capacity over time. Restoring reservoir capacity in case of siltation is always costly. Usually, strict environmental procedures have to be followed to dump the removed reservoir siltation to adjacent areas of dam. Cost of desiltation can be reduced if it possible to dry the material and to reuse it, for example for improving the embankment. The following method of removal of silt from a reservoir, processing the material, and then adding to the downstream face of the embankment was used at a dam in Japan in such a situation, with the following steps:

1) Drain the reservoir.
2) Apply cement type hardening materials to the muddy soils that have accumulated in the reservoir and mix them.
3) Crush the mixture of muddy soils and cementitious materials after hardening.
4) Use the crushed material that has been produced as dam embankment material or counterweight fill material.

The process is explained in Figure 9.3.19.

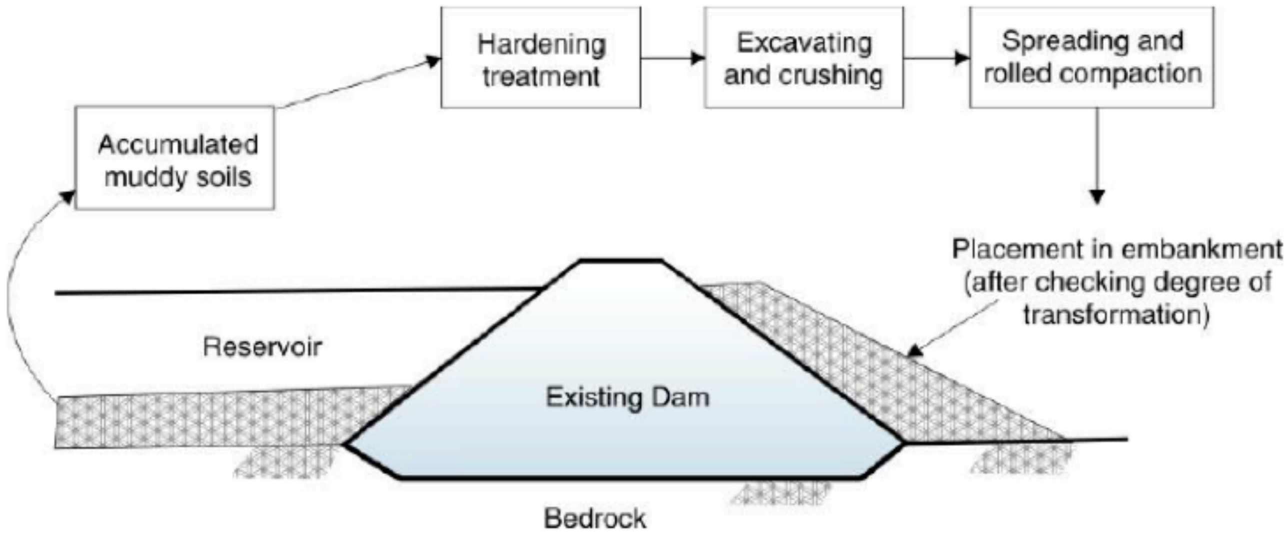


FIGURE 9.3 19: EFFICIENT USE OF ACCUMULATED MUDDY SOILS AFTER HARDENING

10. EMERGENCY ACTION PLAN

10.1 INTRODUCTION

The Emergency Action Plan (EAP) is a useful tool intended to minimize the consequences of a dam failure or a malfunction on the properties or population living downstream. In the last 30 years, several countries became aware of the new approach requested by modern society and developed regulations on this matter, creating a dam classification system according to potential downstream impact and establishing the EAP for dams with a high significant risk

Initially the EAP was applied to medium and large dams, and nowadays it is also applied to small dams as a consequence of the great number of such structures and that 87% of the victims related to dam failures have been caused by small dams, according to the experience in the United States - Bureau of Reclamation, along the period from 1970 to 1997. It is interesting to point out from this experience, that 99% of the victims were situated along the first 24 km downstream of the dams.

The EAP typically falls under the preventive measures to be taken regarding dams safety. For many years these aspects have been taken into account in different regulations and more demanding safety criteria have been established for those dams in which failure or malfunction, even with a very low probability, may cause severe consequences to the downstream population.

An EAP is essential for small dam owners and first responders in the event of a dam failure resulting from a natural event (such as a big storm or earthquake), from an accident or from a manmade event. It should include all pertinent instructions for a dam operator to follow during an emergency. It should be written in a clear and precise manner.

10.2 HOW TO DEVELOP AN EMERGENCY ACTION PLAN

The process of developing an EAP for a small dam generally follows the next steps:

- Determine the potential inundated area downstream in case of a dam failure. At least, Two scenarios have to be considered: inundation at maximum probable flood (PMF) through the spillway without dam failure, and inundation with a dam failure. Likely failures modes have to be identified and considered based upon, for example, dam type (i.e. overtopping or piping for earth embankments).
- Prepare inundation maps clearly indicating the flooded areas and the time of the wave arrival at the downstream endangered area.
- Determine and identify the situations and events that could initiate an emergency action and specify the corresponding actions to be taken and by whom. Develop clear instructions to explain how the operation staff must react before and during emergency.
- Identify all entities, jurisdictions, agencies and individuals, who will be involved in the emergency actions. Coordinate the development of the EAP with all parties. This interaction should include the discussion of an emergency operation center and discussion of the evacuation with destinations, priorities and procedures, post-flood actions (recovery and cleanup) and all other measures. These discussions must be concise and productive, and present visual information. It must be underlined that the development of an EAP does not mean that the dam is not safe. It is advisable that all parties and the public are invited to participate in a visual inspection of the dam because they can bring practical suggestions that could be taken into account. All parties, including the public, must feel involved in the EAP.
- Identify all primary and auxiliary communication systems to ensure continuous internal and external communication.
- List and prioritize all persons and entities involved in the notification process and create the notification flowchart.
- Develop a draft of the EAP. The list of the tasks must be complete and easy to understand so that it can be readily followed.
- Determine the responsibilities under the plan:
    - owner responsibility
    - responsibility for notification and warning
    - responsibility for evacuation (evacuation plan should be developed)
    - EAP coordinator responsibility
- The EAP should foresee actions prior to and following the development of emergency conditions. These actions include:
    - surveillance
    - access to the site
    - response during periods of darkness
    - response during periods of adverse weather
    - emergency supplies, resources and transport means
    - alternative means of communication
    - coordinating information on flows
- Review and discuss the EAP draft with all parties included in the notification list. The EAP must be clearly understood by everyone. The public (community) should also be invited to participate in the EAP revision meeting because their involvement helps in the elaboration of a better EAP and also gives support to the emergency actions.

- The EAP should be periodically updated to take into account changes in land use downstream and any changes to the catchment upstream.

10.3 EVALUATION OF POSSIBLE RISKS

In the event of a dam failure, the potential energy of the water stored even behind a small dam is capable of causing loss of life, serious property damage and an extended period of absence of the services that dams provide.

The EAP should be applied to all small dams with a high consequences potential, which is directly correlated to the population, properties and the environment downstream. It is important to analyze and evaluate the possible dam impacts/hazards, in order to perform a more rational approach to the EAP establishment.

10.3.1 Overtopping During Extremes Floods

Considering that about 90% of all small dams are embankment dams, and that 2/3 of those dams have failed as a consequence of extreme floods, it is very important to perform a thorough revision of the spillway or outlet structures, in order to evaluate its maximum discharge and compare it with the maximum flood calculated based on the latest data from the dam site and the dam catchment.

It is important to emphasize that most old farm dams have been built without a good evaluation of the maximum discharge, what explains the large number of small dams' failure during severe floods, in some areas of several countries. Therefore, when developing an EAP it is very important to initially check the real capacity of the dam outlet or spillway, in order to evaluate the probability of the dam to endure severe storms. It is also important to establish whether the dam has face protection (e.g. grass or blocks) that can resist overtopping events or not.

10.3.2 Piping and Internal Erosion

Several terms and classification systems have been used to describe failures and accidents caused by water flowing through or under an embankment. Piping occurs when internal seepage takes place through a soil causing the progressive removal of soil particles by percolating water, leading to development of internal channels. Animal burrows within an embankment could also initiate a piping failure.

An embankment dam can fail or experience serious distress if water flows without adequate controls through the embankment itself or through the foundation soil and bedrock on which it rests. Small dams are more vulnerable to internal erosion, related to large dams, as a consequence of the inexistence of internal filters and transitions, or poorly constructed filters. Internal erosion in a dam can breach the embankment when it creates a tunnel through the embankment that is large enough to empty the reservoir in an uncontrolled manner.

The analyses of the piping mechanism have to concentrate on the type of embankment and foundation soil and their current properties, the hydraulic gradient and the confining stresses, at the dam base and along the interfaces between embankment and the concrete structures.

10.3.3 Earthquake

Earthquakes are the second natural event that have to be considered in the application of an EAP, through the analysis of the location of the small dam over a seismological map. With such analysis it is possible to predict the intensity of the maximum probable earthquake, and its frequency along the time.

Considering that most small dams are of the embankment type, it is recommended to check the possibility of liquefaction of the dam foundation or of the dam embankment itself, which occurs more frequently with well graded sands and silts. In second place it is important to check the actual level of the dam crest and make a prediction of its settlement after a strong earthquake, to avoid dam overtopping in case of severe storms.

10.3.4 Cascade Dam Failures

In case of cascade of small dams, it is very important not only to consider the safety of a determined dam alone, but its safety as a consequence of the failure of another dam upstream. This scenario would usually take place during severe storms, that can cause dam overtopping if a reservoir that is already at full.

A probable solution to improve the safety of the downstream dam is to protect the downstream slope with gabion, Rolled Compacted Concrete or articulated concrete blocks.

10.3.5 Landslides in the Reservoir

Some large dams have been subject to severe damage as a consequence of landslide in the reservoir area. There is not much information about this problem regarding small dams; nevertheless in some specific places such as reservoirs in very mountainous regions, it is considered important to analyze the possibility of dam failure or severe damage caused by a landslide, and the big waves caused by it when hitting the dam.

10.3.6. Slope Instability

Another reason for dam failure is slope instability, but it happens more seldom than the two previous causes. Instability of downstream slope often occurs mainly when the pore water pressure inside the embankment is not controlled with an adequate drainage system. High pore pressures in the downstream part of the embankment may lead to circular rupture of this face, especially in case of a steep slope.
Instability of upstream slope often occurs after a rapid drawdown of the reservoir, when permeability of the material is too low to allow dissipation of pore pressure.

10.4 DAM SITE ACCESS DURING SEVERE STORMS

A very important point to be considered in an EAP for a small dam with a high impact rating is the access conditions to the dam site during extreme floods. Particularly for dams located in mountainous region, it is usual to observe slope failures or broken bridges during severe storms, which hinders accessibility to the dam site.

It is important to analyze the accessibility to the dam during severe storms, in order to detect all possible problems that can occur along the access roads, and what can be done in order to avoid them during future similar or worse climate conditions. For instance, in the case of slope failures it is important to evaluate the amount of soil and rock to be removed, how long this operation would take and what type of equipment are available in the nearby to do it.

When the existing bridges are not able to support great floods, the possibility of reinforcing them with a more resistant structure must be considered.

10.5 DAM BREAK AND INUNDATION MAP

The analytical requirements for an EAP include the information necessary to conduct dambreak analyses and to prepare inundation maps. The process of developing a workable EAP must necessarily begin with the knowledge of what areas will be flooded as a result of a dam failure, so that the jurisdictions and agencies and individuals involved in the implementation of the EAP can be identified.

The tools for identifying the areas flooded and developing the notification procedures are usually dambreak analyses and inundation mapping for large dams. Such technology can be applied only for the small dams with high potential impact classification, whilst using a more simplified way for the others small dams with less impact.

Several different inflow conditions may need to be investigated to determine the appropriate condition prevailing at the time of a dam failure in order to ensure that the EAP includes all communities that need to be notified. A "sunny day" dam failure, that means reservoir at normal full elevation, normal stream flow prevailing, is generally considered to have the most potential for human life loss, primarily due to the element of surprise. However, a failure of a dam during flood flow conditions will result in flooding downstream areas to higher elevations than during "sunny day" failure.

Experience has shown that the emergency management agencies will use the inundation maps to develop their evacuation procedures, using both the "sunny day" breach and also a failure during a flood level approaching the inflow design flood. Usually basic or hydrological maps of 1:10 000 scale are used for inundation.

Based on some accidents, it is found that for small dams with 5 to 15 m high the water level will be higher than 0.5 m, below that it is possible to consider that the damages will be minimum at the following distances downstream the dam:

- Dam 5 m high .... 4.7 km
- Dam 10 m high .... 7.0 km
- Dam 15 m high .... 7.0 km

These data are important in order to show us that in the case of small dams, we usually have to take care of persons, structures, highways and industries located up to 5 to 7 km downstream, in case of a dam break. In Annex 2, more accurate method to calculate the water depth downstream a dam in case of dam failure, based on Risk management for UK reservoirs. Also there are number of commercial software for dam break flood routing such as SOBEK suite, Mike 11, Tuflow, ISIS, Flow 2D can be used for accurate result.

10.6 RESPONSIBILITIES

The dam owner should:

- Develop and maintain a emergency action plan for all dams where there is the potential for loss of life in the event of dam failure;
- Determine the area, height, rate and timing of potential inundation from relevant dam break floods downstream of the dam;
- Establish and resource a warning / communication system for the timely notification, to operating personnel and emergency authorities, of impending / actual emergencies;
- Provide relevant State, or local, emergency management agencies with details of dam safety emergency response actions (e.g. water releases) and their downstream effects;
- Liaise regularly with emergency agencies to coordinate and maintain appropriate emergency planning arrangements. Ensure personnel with responsibilities under the plan have access to controlled copies of the plan; and
- Regularly update and periodically test the plan.

Regional State Disaster preparedness authorities should have separate guidelines for the preparation of Disaster Plans with focus on evacuation of affected people.

10.7 PERMANENT FILE FOR EMERGENCY PURPOSES

A logbook detailing the daily activities and maintenance of the dam must be kept by the dam owner and stored in a safe facility. All information related to operation and maintenance of the dam, monitoring, dam condition, specific incidents, and dam inspections must be recorded immediately on occurrence in the logbook. All regularly collected information recorded on prescribed forms shall be stored with this logbook. It shall be the responsibility of the dam owner to ensure that this logbook is correctly used, completed and maintained [5].

Anyone responding to a potential dam safety emergency would have access to the files and information specific to that dam. Such information, typically kept by the Owner and State Office of Dam Safety would include the following:

Summary information;

Design / "As Built" drawings; Previous Inspection Reports; Operation and Maintenance Manual Emergency Action Plan

Clearly, the EAP would become the key document if failure is believed to be imminent or in progress. EAPs are often filed also with the state emergency management agency or local state police and fire department.

Realistically however, an engineer or inspector will not have quick access to these files and in some cases, even when the files are available not all critical information may be included.

The following is a list of equipment which might prove useful when responding a top dam safety emergency, and then need to be prepared previously:

- Filed book and pencils
- Clip board

- Digital camera (extra batteries)
- Cell phone (with extra number of batteries);
- Measuring tape
- Flashlight (extra batteries)
- Rain gear and umbrella
- Calculator
- Laptop (with wireless or mobile internet connection)
- Stake or ruler for staff gauge
- GPS Unit
- Tracer dye.

Since the collection of such equipment takes time, it is generally a good idea to preassemble a “ready kit” or plastic tube with all necessary equipment so that it is ready to be loaded into a vehicle at a short notice.

Personal safety gear such as high-visibility clothing, flotation devices and hard hats should be available.

10.8 EMERGENCY EXERCISING AND UPDATING

Emergency incidents at dams and dam failures are not common events. Training exercises are necessary to maintain operation readiness, timeliness and responsiveness. This may be accomplished through development and conducting an emergency exercise program. This is to ensure that guidelines on developing and implementing EAP is practical, implementable and can be used in these situations.

Emergency action plan and disaster plan should be considered as “living” documents, this means that:

1. They should be reviewed each five years for small dams at the latest;
2. Review should include participation of personnel of the dam operating organization, and local authorities;
3. All updates should be made promptly.

Changes that may frequently require revision and update of emergency action plans include changes in personnel of involved organizations and changes in communications systems. As a minimum, review of office telephone numbers and appropriate personnel included in notification flowcharts should be conducted.

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Annexes

Annex -1

Recommended Embankment Slopes For Earthfill Dams

1. Experience of China

In China the completed earth dams are mainly of rolled-fill type, and comprise up to date over 95% of the entire large dams. There are many causes of slope failure. The major ones are the improper control of the placement water content, the low dry density and the low shearing strength.

Practice has shown that for a homogeneous dam of poor placement quality, sudden drawdown of reservoir level from full height to one third of the full height may easily cause sliding of the upstream slope, and that the high position of phreatic line during the first filling of the reservoir or the saturation of the embankment by submergence after continual raining may also lead to sliding of the downstream slope. In some cases, the dam slopes during construction or reservoir drawdown often slid along the weak layers interbedded in the foundation.

In the Hubei Province, the dam slopes are specified within the limit as shown in Table 1. This method provides convenience for the masses and also ensures the dam safety.

Table 1 – Range of embankment slopes for dams below 15 m.

Homogeneos Dams		Sand Clay		Silty loam	
Berm	Upstream slope (from top to bottom)	Downstream slope (from top to bottom)	Berm	Upstream slope (from top to bottom)	Downstream slope (from top to bottom)
Width of berms (m)	N° of Berms		Width of berms (m)	N° of Berms	
1.5	1	1:2.5 1:2.75	1:2.25 1:2.5	1:2.25 1:2.5	1:2.0 1:2.25

Sloping-core Earth Dams			Embankment		Sloping-core and cutoff trench		
Berm		Upstream slope (from top to bottom)	Downstream slope (from top to bottom)	Top thickness (normal to dam slope)	Bottom thickness (normal to dam slope)	Bottom width of foundation cutoff trench	Bottom width of abutment cutoff trench
Width of berms (m)				N° of Berms			
1.5	1	1:2.50 1:2.75	1:2.0 1:2.25	0.8 m	¼ of dam height	¼ of dam height	¼ of dam height

Earth dams with central core			Dam Shell		Central core and cutoff trench		
Berm		Upstream slope (from top to bottom)	Downstream slope (from top to bottom)	Top width	Slope	Bottom width of foundation cutoff trench	Bottom width of abutment cutoff trench
Width of berms (m)				N° of Berms			
1.5	1	1:2~1:2.25 1:2.25~1:2.50	1:1.75~1:2.0 1:2~1:2.25	1.5	1:0.2	¼ water head	½ water

2. Experience of United States

- **Homogeneous Dam**

The homogeneous dam is recommended only where the lack of free-draining materials make the construction of a zoned embankment uneconomical, with the further qualification that for storage dams the homogeneous dam must be modified to include internal drainage facilities.

Even in the construction of a homogeneous embankment, there is likely to be some variation in the nature of borrow material. It is important that the coarse and more pervious material be placed at the outer slopes to approach, as much as possible, the advantages of zoned embankment. It is also important to avoid segregation of the larger particles when the fill is dumped. Segregation leads to the formation of layers of much greater permeability than the other embankment; these layers tend to form drainage channels for percolating water and to increase the possibility of piping.

Because of the possibility of oversights during construction and of cracking, dispersive soil, etc., as discussed previously, consideration should be given to providing an inclined filter-drain to intercept any seepage along defects in the embankment.
The recommended slopes for small homogeneous earthfill dams are shown in Table 6 for detention and storage dams on stable foundations with and without rapid drawdown as a design condition. Where more than one soil classification is shown for a set of slopes, the table indicates that the dam can be constructed to the slopes shown by using any of the soil or combinations thereof.

- **Zoned embankments**

The zoned embankment dam consists of a central impervious core flanked by zones of material that are considerably more pervious. The recommended slopes for small zoned embankment dams are shown in Table 6.23. An excellent example of a zoned dam from the 1950 era is Carter Lake Dam. An excellent example of a more recent era is Ute Dam Dike. This type of embankment should always be constructed where there is a variety of soils readily available because its inherent advantages lead to savings in the costs of construction. Three major advantages in using zoned embankments are listed below:

- Steeper slopes may be used with consequent reduction in total volume of embankment materials;
- A wide variety of materials may be used;
- Maximum use can be made of material excavated from the foundation, spillway, outlet works, and other appurtenant structures.

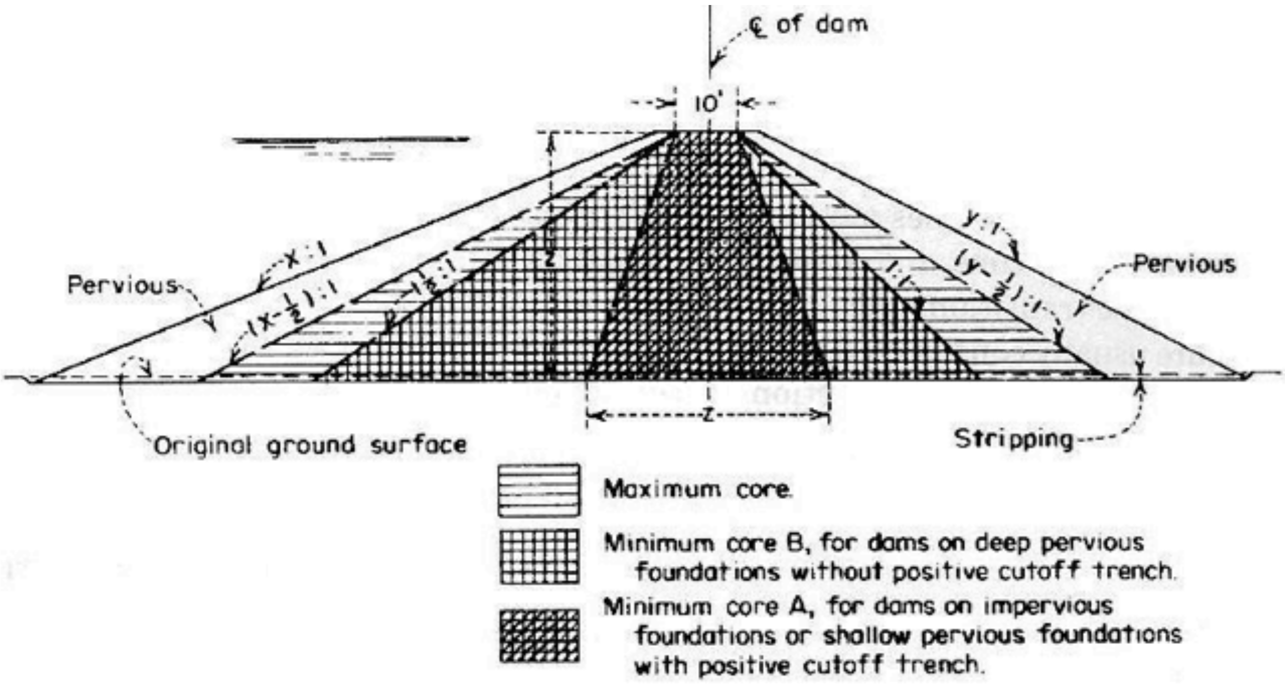


Fig. 1 – Size range of impervious cores used in zoned embankments.

Table 2 – Recommended slope for small homogeneous earthfill dams on stable foundations.

Type	Subject to rapid drawdown [1]	Soil Classification [2]	Upstream Slope	Downstream slope
Homogeneous or modified - homogeneous	No	GW, GP,SW, SP	Pervious, unsuitable	
		GC, GM, SC, SM	2.5:1	2:1
		CL, ML	3:1	2.5:1
		CH, MH	3.5:1	2.5:1
Modified - homogeneous	Yes	GW, GP,SW, SP	Pervious, unsuitable	
		GC, GM, SC, SM	3:1	2:1
		CL, ML	3.5:1	2.5:1
		CH, MH	4:1	2.5:1

[1] Drawdown rates of 15cm or more per day after prolonged storage at a high reservoir levels.
[2] OL and OH soils are not recommended for major portions of homogeneous earthfill dams. PT soils are unsuitable.

Table 3 – Recommended slopes for small zoned earthfill dams on stable foundation

Type	Subject to rapid drawdown [2]	Shell Material Classification	Core material Classification [3]	Upstream slope	Downstream slope
Zoned with minimum core A[1]	Not critical [4]	Rockfill, GW, GP, SW (gravelly), or SP (gravelly)	GC, GM, SC, SM, CL, ML, CH, or MH	2:1	2:1
Zoned with maximum core [1]	No	Rockfill, GW, GP, SW (gravelly), or SP (gravelly)	GC, GM	2:1	2:1
			SC, SM	2.25:1	2.25:1
			CL, ML	2.5:1	2.5:1
			CH, MH	3:1	3:1
Zoned with maximum core [1]	Yes	Rockfill, GW, GP, SW (gravelly), or SP (gravelly)	GC, GM	2.5:1	2:1
			SC, SM	2.5:1	2.25:1
			CL, ML	3:1	2.5:1
			CH, MH	3.5:1	3:1

[1] Minimum and maximum size cores are as shown on Figure 3.
[2] Rapid Drawdown is 15cm or more per day after prolonged storage at a high reservoir levels.
[3] OL and OH soils are not recommended for major portions of the cores of earthfill dams. PT soils are unsuitable.
[4] Rapid drawdown will not affect the upstream slope of a zoned embankment that has a large upstream pervious shell.

3. Experience of Australia

For homogenous dams the following batter slopes are recommended for embankments built of soils classified according to the Unified Soil Classification system.

Table 4 – Batter slope with soil classification and height.[4]

Case type		A Homogenous or Modified				B Modified homogenous			
Purpose		Detention or storage				Storage			
Subject to rapid drawdown		No				Yes			
Soil classification		GW	GC	CL	CH	GW	GC	CL	CH
		GP	GM	ML	MH	GP	GM	ML	MH
		SW	SC			SW	SC		
		SP	SM			SP	SM		
Dam Height (m) & Slope									
	u/s	P	2.5:1	2.5:1	3.5:1	P	3:1	3.5:1	4:1
	d/s		2:1	2:1	2.5:1		2:1	2.5:1	2.5:1
	u/s	P	2.5:1	3:1	3:1	P	3.5:1	4:1	4:1
	d/s		2.5:1	2.5:1	3:1		2.5:1	3:1	3:1
	u/s	P	3:1	3:1	3.5:1	P	3.5:1	4:1	4:1
	d/s		3:1	3:1	3:1		3:1	3.5:1	3.5:1

Notes: U/S = upstream slope and D/S = downstream slope
Rapid drawdown rates of 1.0 m/day.
"P" = pervious denotes soils which are not suitable.

4. Experience of Japan

The cross section shape of the dam shall be determined in accordance with calculations to ensure its stability.

The standard cross section diagram shown in Figure 6.17 and Table 6.25 shall be used in determining the cross section shape of the dam. The thickness of the paved portion is not to be included in the height of the dam when the top of the dam is used as a road.

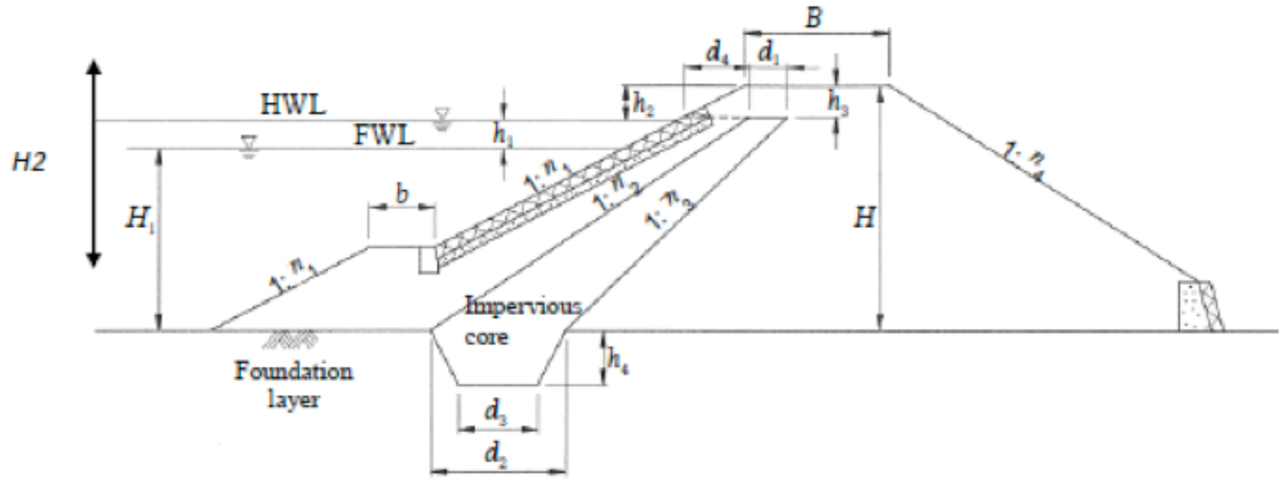


Fig.2 – Standard cross section view of inclined impervious zone

Table 5 – Reference dimensions for reservoirs with an inclined impervious zone

Height of dam H (m)		-5	5-10	10-15	Summary
Height from foundation surface to water level *H1* (m)		*-3,3*	*3,3-7,8*	*7,8-12,2*	Assumed from dam height
Planned overflow depth *h1* (m)		*0,3-0,5*	*0,5-0,8*	*0,8-1,2*	Major differences exist in accordance with position and design of spillways.
Freeboard *h2* (m)		Freeboard *h2* (m)	1,2-1,4	1,4-1,6	In accordance with formulas (3.3.5) and (3.3.6)
Dam crest width *B* (m)		2,0-3,0	3,0-4,0	4,0-5,0	In accordance with formula (3.3.7)
Front Slope	Gradient *n1* (%)	1,5-1,8	1,8-2,1	2,1-3,0	1.5-3.0
	Step width *b* (m)	0-1,5	1,5	0,2	Minimum of 1 m if dam is to be stepped
Impervious Zone	Distance from dam crest *h3* (m)	0,3-0,5	≥ 0,5	≥ 0,5	≥ 0.3 m
	Dam crest width *d1* (m)	1,5-1,8	1,8-2,4	2,4-3,5	1.5–3.5 m
	Distance from front *d4* (m)	1,5	1,5	≥ 1,5	≥ 1.5 m
	Upper excavation width *d2* (m)				Where n2□*n*1□0.1 *n3*□*n*2□0.2
	Lower excavation width *d3* (m)				*d3*□*1/2 d2*
	Depth of excavation *h4* (m)	1,1-1,3	1,3-2,1	2,1-3,2	Depends on soil quality of foundation surface. Figures here included only for reference.
Rear gradient n4 (10%)		1,5-1,8	1,8-2,1	2,1-2,5	1.5-2.5

Note: Construction of drains may be considered depending on on-site conditions.
(3.3.5) In case $R < 1.0$ m , $h2 = 0.05\ H2+1.0$
(3.3.6) In case $R > 1.0$ m , $h2 = 0.05\ H2+R$
Where R :run-up height of waves(m)
(3.3.7) In case $B \geq 3.0$ m , $B = 0.2\ H + 2.0$

5. Experience of Czech Republic

The cross section shape of the dam shall be determined in accordance with calculations to ensure its stability. The standard cross section diagram shown in Figure 2 and Table 6 shall be used in determining the preliminary cross section shape of the small dam

Table 6 – Recommended slopes for small dams in Czech Republic.

Dam sealing part (core) lies in the zone (Fig. 3)	Dam stabilization part lies in the zone (Fig. 3)	Categorization of earths: Dam sealing part (core)	Categorization of earths: Dam stabilization part	Slopes: Upstream 1:x 4)	Slopes: Downstream 1: y
A	DB,CE	GM,GC,SM	Quarry stone	1:1,75	1:1,15
		SC,GC,MG	GW,SW	1:2,8 [1)]	1:1,75
		ML-MI,CL-CI	GP,SP	1:3 [1)]	1:1,75
AB	D, CE	GM, SM	quarry stone	1:3	1,15
		GC, SC, MG CG, MS, CS	GW, SW	1:1,32	1:1,5
		ML-MI, CL-CI	GP,SP	1:3,4	1:1,75
CAB	D, E	GM, GC, SM, SC, MG, CG MS, CS	quarry stone GW, GP	as like as at a core position in the zone AB	1:2,0 [2)]
		ML-MI, CL-CI	SW, SP		1:2,2 [3)]
CABD	E			as like as at homogenous dams	as like as at a core position in the zone CAB
Homogenous dams [5)]		GM, SM		1:3	1:2
		GC, SC		1:3,4	1:2
		MG, CG, MS, CS		1:3,3	1:2
		MKL-MI, CL-CI		1:3,7	1:2,2

Legend to table:

1) At a very permeable material, possibly with respect to the velocity of the level drop, it can be increased up to 1:2,25.
2) If the dam subsoil contains a material with a shear strength of min. tg□sf = 0,74, it is possible to increase to 1:18
3) If in the dam subsoil contains a material with a shear strength of min. tg□sf = 0,74, it is possible to increase to 1:2
4) The stated decline for the upstream face will be used under the highest water level kept in the long term, above this level the slope may be carried out with a decline 1:(x-0,5).
5) At dams with the height up to 4 m, the decline of the upstream slope may be increased to 1:(x-0,5).

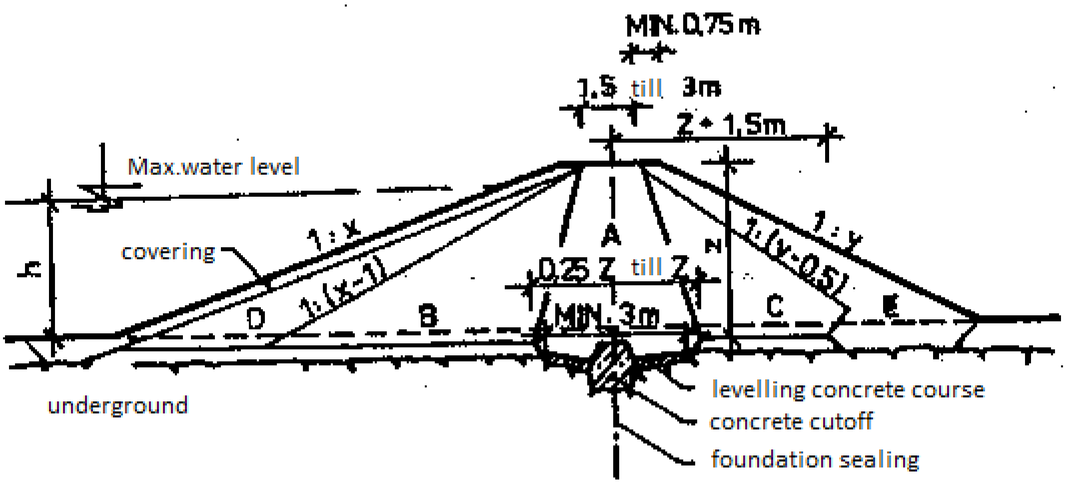


Fig.3 – Standard cross section of vertical core small dams.

Annex -2

Calculate water level downstream in case of dam failure

Historical data show that almost all fatalities caused by dam-break occur within the first 30 km stretch of inundation downstream of the dam (Graham, 1998). A shorter assessment length may be assumed if it can be shown that the floodwater will flow into a sea or a river able to cope with the discharge with minimal residual impact. In this case this flow is assumed to be corresponding to a water depth that does not exceed 0.5 m.

When reviewing fatalities from past dam-break events, it can be seen that half of these fatalities occur within the first 5 km reach downstream of a dam (Graham, 1998). Taking this into consideration, for small dams two valley areas have been defined for the assessment

- Near valley area 0-5 km downstream
- Far valley area 5-10 km downstream

Areas beyond 10 km or if the water depth reaches 0.5 m are not included within the assessment procedure as it is not anticipated that damages will occur at such conditions.

The calculation of flood level along the valley depends upon the features of the valley (width, slope etc.). Since a valley is never geometrically consistent along its entire length, an improved estimate of water level may be made by dividing the valley into zones. The zones should be selected according to topographic features, i.e. Zone 1: length of narrow valley, Zone 2 valley wider, Zone 3 coastal plain etc. Care should be taken at this stage since the selection of zones will affect the level of assessment work required! It is suggested that a maximum of four zones be identified for each of the near and far valleys. In some situations it may be appropriate to define only a single zone in each area. Following calculation of water levels in each zone, the impact assessment will be applied only to the whole near and far valley regions, not the individual zones.

Simple calculation of potential water level

Having defined valley zone(s) for both the near and far valley, an estimate of maximum floodwater level may be made. The most appropriate method for calculating water level will depend upon the availability of data and resources for performing the calculation. It has been assumed that limited data is available. The simplest technique is therefore to apply Manning's equation using an estimate of the peak discharge for the given location:

Qp = (A 5/3 S 1/2) I (n P2/3)

Where:

QP Peak discharge at the calculation point (m3/s) A Flow cross-sectional area (m2)
S0 Slope along the river valley
n Manning's roughness coefficient
P Wetted perimeter of valley section (m)

QP is initially estimated at the dam using the discharge equation detailed below.

Discharge prediction for an embankment dam

The rate of formation of a breach through an embankment dam, and hence the discharge, depends upon factors that include, for example, the volume of water stored. The recommended discharge equation is based on observed data records from breach events and offers a peak discharge based on the best fit to observed data (MacDonald Langridge-Monopolis, 1984).

To predict the peak discharge possible from a breach in an embankment dam apply:

Qp = 330 (BFF)0.42

Where:

Qp Peak discharge (m3/s)
BFF Breach formation factor:

and:

V-Storage volume of reservoir (Mm3)
H -Height of peak reservoir water level above the base of the dam (m)

To predict the time of failure to peak discharge, apply:

time to peak discharge, T P (s) = 120 H

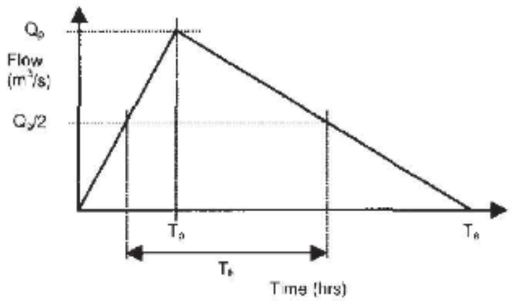
where:
H -Height of peak reservoir water level above the base of the dam (m)

The magnitude of Qp will reduce as the flood wave travels down the valley, due to attenuation. Therefore, to estimate the water level at the intersection between each valley zone, a new value of QP should be calculated. This may be done using the technique outlined below:

Where:
Qp(x)
Discharge at a location X m downstream of Qp(0) location (m3/s) Qp(0) Discharge calculated at an upstream location (m3/s)
X - Distance between zone intersections (i.e. length of the zone across which the calculation is being made, not the chainage downstream of the dam) (m)

And
Where:
K -Factor, with a possible range of 1 to 10. The suggested value is 2.5.
B - Estimated surface width of valley at estimated water depth (m) S0 Valley slope
n- Manning's n
Qp(0)- Discharge calculated at upstream location (m3/s)

and:
Th- Time period at half discharge as shown below.



Te may be calculated by ensuring that the volume under the hydrograph matches the reservoir volume V.

i.e. V=0.5(Qp x Tp) + 0.5 (Qp x (Te-Tp))

Note that B is estimated initially from engineering judgment. This should be compared with the value obtained when calculating the estimate of floodwater depth. If the values differ significantly the calculation should be repeated using the newly calculated value of B. It is likely that one iteration will be sufficient to provide a reasonable value of B.

Following calculation of an attenuated value of Qp the associated new value of Th may be found using the equation below. These new values of Qp and Th may then be used for subsequent attenuation calculations.

Th(new) = Th(original) x Qp(original)/Qp(new)

The value of Th therefore grows in proportion to the reduction in peak discharge. Therefore, to

calcu1ate potential water levels at the intersection between valley zones:
For Zone 1 (immediately downstream of the dam):

- calculate Qp at the dam and Th from the flood hydrograph
- estimate x-the distance from the dam to the end of the zone (i.e. the zone length).
- Consider the path that the flood wave would take rather than following (measuring) a potentially meandering river route. During a dam-break the flow will not follow or be contained within the river channel
- calculate the valley slope (for the zone) from map contours
- estimate Manning's n for the valley considering the possible high depth of water consider the potential flood of water and make an initial estimate the width of valley (B) that will be flooded
- calculate La using the equation above
- calculate Qp(x) using the equation above having calculated Qp, the average depth of water at the zone intersection may be calculated using Manning's equation ompare calculated and original values of B. If the difference is greater than 10 per cent recalculate La, Qp(x) and the average depth of water using the calculated value of B. Repeat until the difference in values is less than 10 per cent plot a flooded outline for the zone using the water depths calculated.

Now repeat the above process across each zone using the values estimated from the previous zone as the starting point for the next.

Accuracy

It should be recognized that the above method is only intended to give an order of magnitude estimate to allow the scale of potential flood impact to be assessed. The method is not intended, and should not be taken, as a substitute for full dam-break modeling.

Selection of Manning's n roughness value

The selection of this value affects estimation of flow depth and rate of flow progression along a valley. Although there is reasonable confidence in selecting appropriate values for river flow conditions (Chow, 1986), it should be recognized that dam-break flood conditions are very different. Under these conditions the extreme extent of flooding means that the flow is likely to pass through areas outside the normal flood plain, which may be heavily vegetated or developed. Under these conditions it is likely that then value will be between 0.05 and 0.1, although the true value will be site-specific.

Obstructions to flow

Large road and railway embankments often cross river valleys and can provide a significant obstruction to flow. These may withstand the flow, creating a secondary dam, thereby attenuating the flood wave, or collapse at some time during the flood.

Drowning of breach and valley conveyance

The estimation of discharge from the dam assumes free flow out of the reservoir. If there is a significant obstruction downstream, or the valley slope and size is insufficient to allow passage of the predicted discharge at a reasonable water level, then this discharge would not occur from the breach. If the predicted water level in the valley exceeds two thirds of the maximum reservoir depth at the dam, the water level in the valley would drown the flow from the reservoir and so reduce the peak discharge.

Annex-3

Environmental & social factors

It is essential and necessary to bring social and environment considerations early into the planning process. This will allow planners and designers to consider project locations and design elements that will better address environment and social requirements.

There are two ways of looking at environment and social issues in dam projects, i.e. impacts of dams on environment and society and impacts of environment and social factors on safety of dams

ICOLD B159, Committee on the Environment: Supplement to the position paper on dams and the environment, supplementary paper 2012 , in its introduction states the following:"Global problems such as climate change have impacts on dam safety. Dams can therefore be both affected by global problems and help provide solutions to them."

The following relevant environmental and social factors are summarized from the USACE dam safety guidelines and are also covered in the ICOLD recommended practices regarding environment and social issues of dam safety.

1. Water quality

Changes in water quality are likely to occur within and downstream of the development as a result of impoundment. Major issues include reduced oxygenation, temperature, stratification potential, pollutant inflow, and propensity for disease proliferation, nutrient capture, algal bloom potential and the release of toxicants from inundated sediments.

Adequate data collection and an EIA process that identifies potential problems prior to dam design are critical. Design and operational systems that minimize as much as possible the negative impacts within the storage and downstream; examples include multilevel off-takes, air injection facilities, aerating turbines, and de-stratification capability. While removal of vegetation from proposed impoundments is expensive, the potential benefits for water quality means that at least some removal should be considered. Working with local communities and regulatory authorities in improving catchment management practices can have significant water quality benefits for hydro reservoirs.

2. Sediment transport and erosion

The creation of a reservoir changes the hydraulic and sediment transport characteristics of the river, causing increased potential sedimentation within the storage and depriving the river downstream of material. Sedimentation is an important sustainability issue for some reservoirs and may reduce the long-term viability of developments. Reduction in the sediment load to the river downstream can change geomorphic processes (eg. erosion and river form modification).

Development proposals need to be considered within the context of existing catchment activities, especially those contributing to sediment inflow to the storage. Reducing reservoir sedimentation through cooperation with local communities and regulatory authorities in improving catchment management practices is an option. Specific actions, such as terracing or reforestation, may need to be considered. In some cases sediment by-passes, flushing systems or dredging should be investigated. Operational or physical mitigation measures to reduce erosion of downstream should be considered for both proposed and existing developments and appropriate objectives set.

3. Downstream hydrology and environmental flows

Changes to downstream hydrology impact on river hydraulics, in-stream and streamside habitat, and can affect local biodiversity. Operating rules should not only consider the requirements for power supply, but also be formulated, where necessary and practicable, to reduce downstream impacts on aquatic species and human activities.

Operating schedules should, where necessary and practicable, incorporate environmental water release patterns (including environmental flows) within the operational framework for the supply of power. Downstream regulating ponds and other engineering solutions may provide cost-effective alternatives to environmental flow releases directly from power stations. It is important that the environmental objectives of any flow release are identified in a clear and transparent manner. These releases need to be developed within the context of environmental sustainability and also take into account local and regional socio-economic factors. It is desirable that the environmental flow objectives be agreed with local communities.

4. Rare and endangered species

The loss of rare and threatened species may be a significant issue arising from dam construction. This can be caused by the loss or changes to habitat during construction disturbance, or from reservoir creation, altered downstream flow patterns, or the mixing of aquatic faunas in inter-basin water transfers. Hydropower developments modify existing terrestrial and aquatic habitats, and when significant changes cannot be avoided, mechanisms to protect remaining habitats at the local and regional scale should be considered in a compensatory manner.

Plans to manage this issue need to be developed prior to construction and options for mitigation identified and assessed. Habitats of critical importance should be identified (within a wider regional context) and impacts to these avoided or minimized as much as possible during the design phase. Targeted management plans need to be developed for species of conservation significance. Translocations or habitat rehabilitation may be options, along with identification of suitable habitat for 'reserve' management.

5. Passage of fish species

Many fish species require passage along the length of rivers during at least short periods of their life-cycle.

In many places the migration of fish is an annual event and dams and other instream structures constitute major barriers to their movement. In some cases the long-term sustainability of fish populations depend on this migration and in developing countries local economies can be heavily reliant on this as a source of income.

The passage of fish is an issue that must be considered during the design and planning stage of proposed developments (dam site selection) and adequate consideration should be given to appropriate mechanisms for their transfer (eg. Fish ladders, mechanical elevators, guidance devices and translocation programs). Large-scale downstream migration of some species may require mitigation measures to reduce mortality by passage through turbines. Appropriate and feasible options for facilitating passage are also an issue for existing developments.

6. Pest species within the reservoir (flora & fauna)

In some regions a significant long-term issue with reservoirs, irrespective of their use, is the introduction of exotic or native pest species. The change in environment caused by storage creation often results in advantageous colonization by species that are suited to the new conditions, and these are likely to result in additional biological impacts. In some instances, proliferation may interfere with power generation (eg. clogging of intake structures) or downstream water use through changes in the quality of discharge water (eg algal bloom toxins, deoxygenated water).
Identifying the risk of infestation prior to development should also help identify potential options for future management or mitigation. Shorter residence time of water is one viable mechanism for reducing risk. Downstream water uses must also be consideredwhen examining potential options for control.

7. Health issues

The changes brought about by hydropower developments have the capacity to affect human health. Issues relating to the transmission of disease, human health risks associated with flow regulation downstream and the consumption of contaminated food sources (eg, raised mercury levels in fish) need to be considered. The potential health benefits of the development should also be identified.
Public health and emergency response plans should be developed in conjunction with local authorities. These plans, and their associated monitoring programs, should be relevant to the levels of risk and uncertainty. The health benefits due to improved water supply, economic improvements and flood control should be recognized. Proper reservoir management can be highly effective in eliminating mosquito-borne illnesses such as malaria.

8. Construction activities

Construction needs to be carried out so as to minimize impacts on the terrestrial and aquatic environment. Where a new development is planned, there are a range of activities that can result in environmental impacts, both terrestrial and aquatic. Noise and dust may also be issues where the development is close to human habitation.

These issues should be adequately addressed during the EA stage and plans developed to manage these issues. Plans to manage specific issues may be required; e.g., rehabilitation of borrow pits, management of construction site drainage, storage and handling of chemicals. Similar plans to manage disturbance to terrestrial and aquatic fauna may also be required.

9. Environmental management systems

It is recommended that all hydropower schemes implement an independently audited environmental management system. An environmental management system should allow for effective management of the range of environmental issues associated with the on-going operation of the hydropower scheme.

The associated monitoring programs and environmental plans should ensure a program of continuous improvement in environmental management over the life of the project.

Managing social impacts

There are various issues that require management to ensure that change affecting communities and individuals is effectively managed during the planning, construction and operation of hydropower facilities.

Possible social impacts that require consideration are identified below.

1. Changes to resource use and biodiversity in the area of the proposed project and the impacts this may have on the local community.
2. Distribution of benefits among affected parties.
3. Effectiveness and on-going performance of compensatory and benefits programmes.
4. Public health issues that can result from the modification of hydrological systems, especially in tropical and sub-tropical areas, where water-borne diseases can be a significant issue. In some reservoirs, a further concern is the management of the temporary rise of mercury levels in fish.
5. The impacts of displacement on individuals and communities. These impacts include:
   - the physical loss of homes and lands;
   - the transition to alternative means of earning a livelihood, particularly for populations that rely heavily on local land and resources for their way of life or that have a traditional existence;
   - disruption of established community networks and loss of cultural identity.

Outcomes for new developments and strategies to achieve proposed outcomes are provided in the Annex--.

Environment and social factors that may affect the safety of dams

- **Climate change**

The sudden or unplanned release of impounded water in the event of a dam failure can cause much destruction of life and property. A climate change may give rise to more extreme floods and may require additional considerations in the design and construction of dams beyond the conventional design and construction process. However there is no proper understanding nor established methodology for predicting the impact of climate change on water structures. This is a global challenge as well as for the Eastern Nile region.

- **Land stability**

There are evidences that landslides have caused overtopping of dams. The sudden plunge of mass of rocks and soil in to a reservoir can displace the water causing overflow over the dam. This may result in dam failure and can also harm power houses downstream. Proper assessment of slopes and geological characteristics of land masses in the perimeters of the reservoir is necessary to ensure stability of abutments, islands within a reservoir and land around reservoirs and dams.

For reservoirs in very mountainous regions, it is considered important to analyze the possibility of dam failure or severe damage caused by the failure of a big slope along the reservoir border, and the big waves caused by it when hitting the dam.

- **Debris and vegetation**

Floating debris and vegetation or large trunks of tress may obstruct gates and outlets that may endanger the safety of dams due to blockage of gated spillways.

"Structural instability can occur due to falling/decaying tree/woody vegetation and root system growth. Large, seemingly stable and innocuous trees can easily be blown over or uprooted in a storm/flood and cause a large hole left by the root system. This in turn can shorten the seepage path and initiate piping, or a breach in the dam."

This may be countered through a proper environmental assessment studies which usually address issues of vegetation clearing, soil and land stability studies.

- **Water quality**

Sever water quality degradation in reservoirs may affect the integrity of gates and upstream dam facing material.

- **Sabotage**

Section 8.3.6 of ICOLD's document on Small Dams, Design, Surveillance and Rehabilitation, 2011 states, "This possibility is not usual in all places, but unfortunately it has to be considered in several countries nowadays. Nevertheless, this item will not be examined in this ICOLD publication."

Sabotage is a real possibility in the Eastern Nile region. Ill planned and executed resettlements without proper compensations may result in disgruntled communities that may threaten the safety of the dam with sabotage. Political instability and threats of extremists may require enhanced security assessment for dams in the Eastern Nile region.

The USACE dam safety guidelines recommends at a minimum , a security assessment should be conducted every five years, using established risk assessment methodology and dams should have physical security plans.

Annex-4

Dam Safety Vegetation Management

- **Purpose**

The establishment, maintenance, and control of vegetation pose Engineering, as well as routine maintenance considerations. This guidance establishes minimum requirements for maintenance/control of vegetation at dams, abutments, spillways, inlet/outlet channels, and other appurtenances.

- **Background.**

Vegetation is much more than an aesthetic consideration. Proper vegetation management is necessary to preserve the design functionality of critical project features. Requirements for mowing and eradication are documented in the project specific Operations and Maintenance Manual. Changes in vegetation management practices to promote project benefits such as recreation and environmental enhancement must be carefully evaluated from a dam safety perspective and coordinated with dam safety experts.

Vegetation that adversely impacts engineered structures or inhibits inspection, monitoring, and emergency response actions is not allowed.

1. Beneficial Vegetation

Beneficial vegetation, such as grass cover, can assist in preventing erosion, controlling dust, defining zones of use, and creating a pleasant environment. Uniform grass cover enhances visual inspection, allowing the detection of seeps, settlement, displacements, and other evidence of distress. Robust grass coverage along embankments and discharge channels can help deter the natural establishment of trees and other deep rooted species.

2. Undesirable Vegetation

Woody vegetation and aquatic plants (e.g. cattails) can obscure large portions of the dam, preventing adequate visual inspection, creating potential seepage pathways, reducing discharge capability, and can threaten the stability and integrity of a structure.

Structural instability can occur due to falling/decaying tree/woody vegetation and root system growth. Large, seemingly stable and innocuous trees can easily be blown over or uprooted in a storm/flood and cause a large hole left by the root system. This in turn can shorten the seepage path and initiate piping, or a breach in the dam.

Root systems may undermine concrete slabs, causing erosion of foundation materials and subsidence or heave. Additionally, root systems can interfere with interior drainage systems. Trees and aquatic vegetation in channels can restrict flow volumes, or become a source of debris which blocks releases. Trees in channels can also initiate uneven flow patterns and cause erosion that may divert discharges out of bank. All of these can ultimately threaten public safety.

- **Policy.**

The following areas shall remain free of trees and other woody vegetation such as shrubs and vines:

- The dam and dam toe area
- In or around seepage monitoring systems or critical areas for seepage observation
- Abutments and groins
- Emergency spillways and regulating outlet channels, including channel floors, side slopes and approaches
- Outlet works discharge channels

Inspections conducted either by project personnel, or engineering personnel must always consider the potential dangers from excessive or inadequate vegetation growth. Changes in surfaces, such as cracks, depressions, and movements must also be readily observable via controlled grass cover. Any evidence of seepage or erosion must be quickly identified, monitored evaluated and controlled to prevent flows that could become detrimental to the safety of the structure. Inspection of vegetation shall be part of each annual and formal periodic inspection for each project and shall be discussed in the respective reports.

The governing criteria for maintenance of vegetation on the dams, or areas adjacent to, or immediately downstream of dams is to provide ready and adequate visual observation.

Design and construction of landscape plantings, including irrigation systems, must be carefully devaluated and reviewed from a Dam Safety perspective and approved by dam safety experts.

Trees, brush, and weeds in spillways and inlet and outlet channels shall be maintained so as not to obstruct flows, or cause any threat or potential threat to areas downstream of the dam. Specified spillway and outlet works design discharge capacities must be maintained. Tree and vegetation removal from spillway discharge areas downstream of the crest or sill is required to avoid "head cutting" or causing flow concentrations.

- **Implementation**

Mowing/ clearing limits for each dam shall be identified by dam safety personnel within Engineering Division and documented on aerial photographs or plan drawings, as part of the project Operations and Maintenance Manual. The limits shall be site-specific and shall take into consideration the topography, phreatic surfaces within the structure and abutments, foundation characteristics and any historical problems with the structure.

The horizontal limits of clearing shall not be less than the entrance width of the spillway. In the vertical direction, no encroachment by woody vegetation of any kind can be tolerated up to the elevation of the inflow design flood profile. Dam safety personnel shall establish specific clearing limits for spillways based on project hydrological characteristics, and they shall be permanently and clearly marked in the field.
Riprap in all areas shall be maintained free of vegetation. This includes embankment slopes, discharge channel slopes, and emergency rock stockpiles.

- **Remediation Procedures**

Undesirable woody vegetation identified by Dam Safety personnel shall be removed. Removal of woody vegetation will require engineering judgment to determine if the root system has engaged water bearing regions of the dam and\or site specific geologic areas of special interests such as jointed rock formation which contain water at the toe or dam abutments.

Tree and woody vegetation growth on the upstream slope should be undercut to remove all stumps, root balls, and root systems. The undercut area must be thoroughly inspected to confirm that all major root systems (greater than about one-half inch in diameter) have been removed during the undercutting operation to prohibit regrowth. Suitable backfill shall be placed in the excavation and properly compacted to the dam remediation design limits. Backfill shall be similar to the in-situ embankment fill material and shall be compacted. Installing a slope protection system is recommended to reduce the potential for wave and surface runoff erosion.

Engineering judgment will be required to identify the depth and extent of stump and root ball removal, laying back the undercut slope and selection of backfill based on dam design.

Alternative methods, such as herbicide spraying, burning, or cutting trees flush to the ground surface and leaving roots in place may be considered, in consultation with dam safety experts. However, burning atop riprap is prohibited as this can weaken and degrade the rock.

The suggested dam remediation design and construction procedure suggested for complete removal of trees, stumps, root balls, and root systems consists of the following activities:

- Cut the tree approximately two (2) feet above ground leaving a well-defined stump that can be used in the root ball removal process;
- Remove the stump and root ball by pulling the stump, or by using a track-mounted backhoe to first loosen the root ball by pulling on the stump and then extracting the stump and root ball together;
- Remove the remaining root system and loose soil from the root ball cavity by excavating

the sides of the cavity to slopes no steeper than 1:1 (horizontal to vertical) and the bottom of the cavity approximately horizontal;

- Backfill the excavation with compacted soil placed in relatively loose lifts not greater than about eight (8) inches in thickness. Compaction of backfilled soils in these tree stump and root ball excavations typically requires the use of manually operated compaction equipment or compaction equipment attached to a backhoe.
- Procedure for total removal of trees near the toe is more complicated. Treatment of mature tree penetrations in a downstream slope may involve installation of a subdrain and/or filter system in the tree penetration excavation and backfill with compacted soil placed in maximum loose lifts of eight inches.

- **Establishment of Vegetation**

All disturbed areas must be protected by seeding and mulching. Timing must be considered to allow seed to germinate and develop roots prior to winter or heavy precipitation seasons.

Waivers to allow additional vegetation or alternate remediation techniques must be submitted in writing to the Dam Safety Officer. A multi-discipline team shall review the proposed change, assess potential dam safety impacts, and provide a written recommendation to either approve or decline by the Dam Safety Officer.

ENTRO is an autonomous organ established to implement the Eastern Nile Subsidiary Action Program within the framework of Nile Basin Initiative

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