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Report

A REVIEW ON DAMS AND BREACH PARAMETERS ESTIMATION

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Review dams

ABSTRACT

Nowadays, especially after the appearance of the global warming effects, water is becoming less and less available. Here appears the role of water resources engineering. That is; finding the mean through which we can collect water. One alternative for doing so is the storing of water behind dams. This is why this report will focus on dams' issues. This report is divided in two sections. The first section deals with the most common types of dams, the forces applied on them, the modes of failure of these structures, the environmental effects on the stream, the decommissioning and other technical matters. The second part focuses on the different methods used in order to estimate or predict the breach of the dams especially for the embankment type. These methods are applied to the case of the Timberlake Dam in Lynchburg, VA that failed in 1995 and was rebuilt in 2000.

ACKNOWLEDGEMENT

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Tony Atallah

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Introduction

A dam is a structure that is built across a river or stream for several purposes that are discussed in the following (see fig 1).

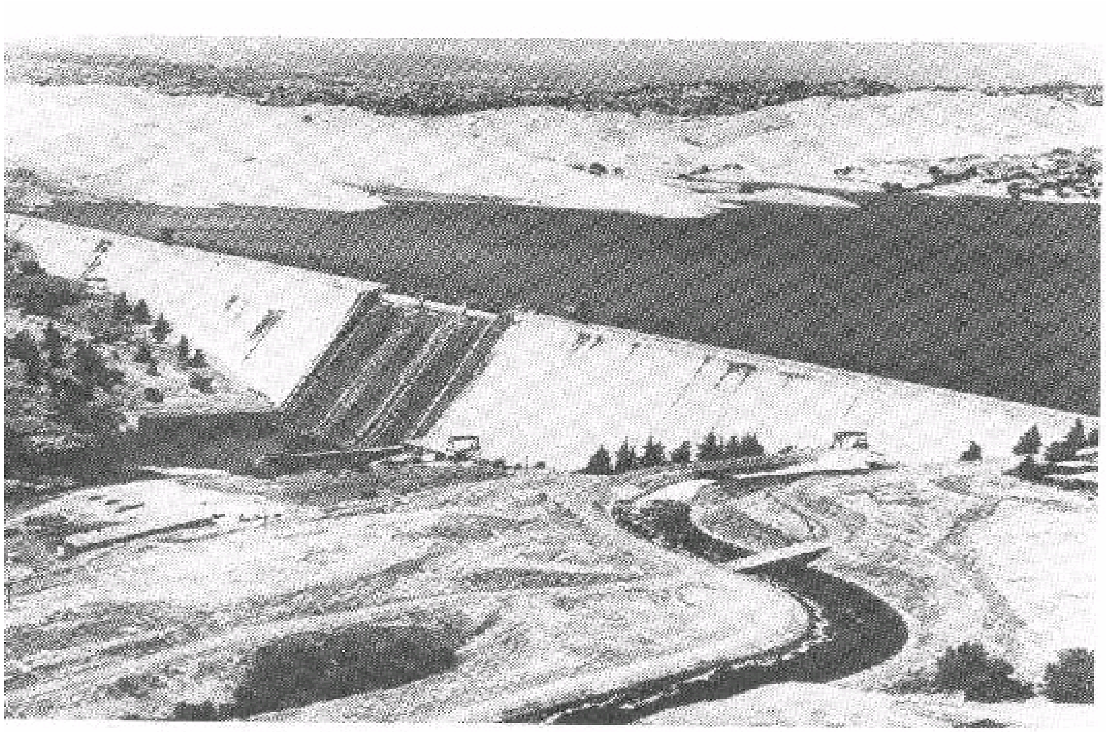

Figure 1. A dam ([1], p.283)

A dam is a structure that forms a "barrier" that obstructs the river and its flow. In order to distribute the water to the downstream side, there should be some outlet structures.

Dams have spillways (see fig 2) that are designed to pass water to the downstream side of the river safely (i.e. for dissipating huge floods, to maintain a certain quantity of water to reach the downstream side of the river for aquatic life or to protect the dam from being overtopped).

Figure 2. Comparison between a dam with spillway (top) and one without (bottom)

A dam is built to last for a very long time (50 to 150 years). Therefore it should be designed in such a way that it can sustain all possible problems it would face (i.e. different types of erosion, sustain against the biggest flood, sustain an earthquake and so on).

Purposes of Dams

Dams provide a life-sustaining resource for people. Dams represent a part of a nation's infrastructure. Dams are built for several reasons and purposes such as:

-Improvement of water supply for domestic and municipal uses

-Irrigation of agricultural areas

-Power generation (see fig 3)

-Water quality improvement

-Recreational improvement

-Fishery improvement.

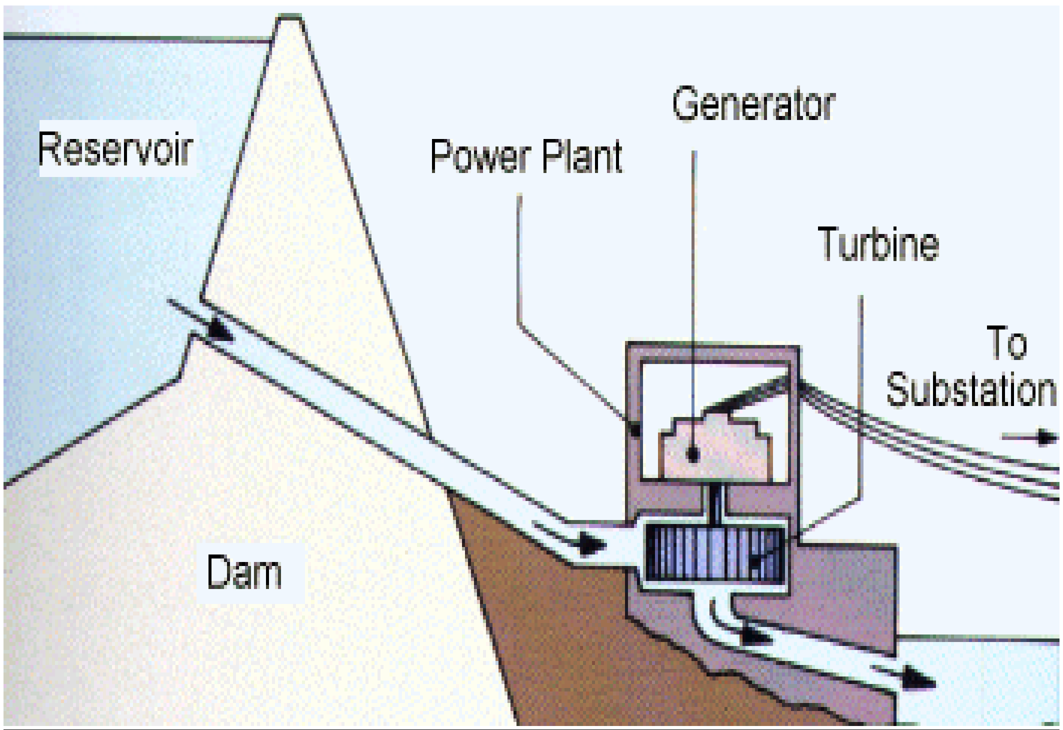

Figure 3. Example of dam used for power generation ([23])

The ranked uses of dams in the USA are listed below ([11]):

Recreation	31.3 %
Fire and Farm Ponds	17 %
Flood Control	14.6 %

Irrigation	13.7 %
Water Supply	9.8 %
Mine Waste Retention	8.2 %
Hydroelectric	2.9 %
Undetermined	2.3 %
Navigation	0.2 %

Many dams fulfill several purposes at the same time.

Design Flood Selection

In order to know the height of the dam and the peak discharge that can safely pass a design inflow hydrograph should be implemented and routed through the reservoir.

Criteria for Choosing the Inflow Design Flood

There are several methods to determine the inflow design flood ([6], p.11). The entire dam is assigned one IDF (intensity-duration-frequency), which is determined based on the consequences of failure of the section of the dam. This does not contradict with the design criteria for different sections of the dam that may be based on the effects of their failure on downstream areas ([6], p.11).

A flood less than the PMF (probable maximum flood) may be adopted as obtained from the design IDF in situations where the consequences of dam failure at flood flows larger than the selected IDF is acceptable.

Flood frequency and risk based analyses may be used to hold operation and maintenance costs to a reasonable level, to maintain public confidence in owners and agencies responsible for dam safety and to be in compliance with local, state or other regulations applicable to the facility ([6], p.11). Generally, it would not be an appropriate risk to design a dam having a potential for failure of a return period less than 100 years (T=100 years) ([6], p.11).

Design Flood Computations

There are two distinct ways by which the design flood is computed:

1-By using a rainfall–runoff model (or a deterministic model) .In this case, the whole analysis is based on the probable maximum precipitation (PMP).

2-By using a statistical analysis (or an extreme value analysis). This method is based on either floods or rainfalls having specified probabilities or return periods.

The procedure of each method is discussed in the following:

Rainfall Runoff Model

The steps to be followed in this procedure are ([2], p.45):

1-Divide the drainage area into sub areas

2-Implement a runoff model

3-Determine the PMP (probable maximum precipitation)

4-Order PMP increments into acceptable storm rainfall patterns

5-Estimate, for each time interval, the losses from rainfall due to detention and infiltration within the watershed

6-Subtract losses from rainfall in order to have rainfall excess

7-Apply rainfall excess values to a runoff model for each sub area of the basin

8-Obtain the flood hydrograph of each area

9-Route the flood of each area

10-Route the inflow through the outlets and spillways to obtain estimates of storage elevations, discharges of the dam and tail water elevations.

If the routing reveals that there is chance for overtopping failure of the dam, the resulting flood wave may be routed through the downstream valley to give a basis for assessment of damages.

Statistical Method

This method is mainly based on the analysis of an extreme value of peak annual floods. By fitting a distribution (i.e. Pearson's method) to these peaks, you can get the design flood that corresponds to a chosen probability of exceedence or T-year flood.

Classification of Dams

Dams are classified according to two criteria ([2], p.16):

1- Size: based on the height of the dam and storage capacity. There are 4 sizes of dams: small, medium, large and major (see table 1)([3], p.50).

Table1.Classification of dams according to their size ([3], p.50)

Size	Capacity (10^6 m^3)	Height (m)
Small	Below 1	Below 8
Medium	1-3	8-15
Large	3-20	15-30
Major	Above 20	Above 30

2- Hazard potential:the hazard potential is the possible adverse incremental consequences caused by the release of water or stored contents due to failure or misoperation of the dam or appurtenances ([6], p.5). It describes the consequences or effects of the dam's failure. It is assigned based on the effects of a failure during both normal and flood flow conditions ([6], p.5).

There are 4 categories of hazard potential, but no exact dollar amounts are taken as a scale by which one can choose the category ([6], p.5). These categories are: very low, low, significant, high (see table 2):

a) Very low: where loss of life is impossible and economical consequences are sustainable

b) Low: where failure or misoperation results in a zero probability loss of human life and low economic and environmental losses at the downstream side of the dam.

c) Moderate: where there is zero probability loss of human life but appreciable economic and environmental damages occurred at the downstream of the dam.

d) High: may cause loss of human life that may be sometimes catastrophic.

Table 2.Hazard potential classification ([3], p.50)

Hazard potential	Loss of life	Economic loss
Very low	Impossible	Minimal
Low	Impossible	Marginal
Moderate	Possible	Appreciable
High	Probable	Excessive

The combination of both hazard and size for dam is given in table 3.

Table 3.Classification of dams according to both size and hazard ([3], p.50)

Hazard /size	Small	Medium	Large	Major
Very low	4	3	2	1
Low	3	2	1	1
Moderate	2	1	1	1
High	1	1	1	1

Table 4 classifies dams according to the flood for which they are designed keeping in mind the conditions of (i) the peak flow without overtopping of the dam (no freeboard) and (ii) accommodating the design flood with normal dry freeboard allowance.

Table 4.Classification of dams according to their design flood and peak flood ([3], p.51)

Class	Peak Flood yr (i)	Design Flood yr (ii)
1	10000	2000
2	2000	500
3	750	250
4	250	100

Table 5 summarizes the classification of size and hazard potential in VIRGINIA.

Table 5.Size and hazard classification in VIRGINIA ([2], p.155)

Class of Dam	Hazard potential if impounding structure fails	Size Classification	Maximum Capacity (ac-ft)	Height (ft)	Spillway Design Flood (SDF)
I	Probable loss of life; excessive economic loss	Large Medium Small	>50000 >1000 and <50000 >50 and <1000	>100 >40 and <100 >25 and <40	PMF PMF 1/2PMF to PMF
II	Probable loss of life; appreciable economic loss	Large Medium Small	>50000 >1000 and <50000 >50 and <1000	>100 >40 and <100 >25 and <40	PMF 1/2PMF to PMF 100 yr to ½ PMF
III	No loss of life; minimal economic loss	Large Medium Small	>50000 >1000 and <50000 >50 and <1000	>100 >40 and <100 >25 and <40	PMF 1/2PMF to PMF 50 yr to 100 yr

In classifying the hazard potential of a dam, this classification should be based on the worst –case failure condition ([2], p.17). This classification can be assessed by field investigations and review of available data such as the topographic maps, performing a dam break modeling and running a gradually varied flow analysis ([6], p.6).

In simulating the dam breach if there is no loss of life downstream, the chosen design flood need not be very conservative ([3], p.51).

For dams where failure may cause loss of life downstream, the recommended guidelines suggest ([3], p.52):

1) Design for a 1000 to 10000 year flood and check the safety of the dam by routing a PMF through the reservoir.

2) Design for the PMF (probable maximum flood).

These criteria differ from one country to another. Here is a comparison between the flood selection criteria in the USA and that in UK.

A) In the United States, the recommended design floods range from 100years for a small dam with a small reservoir and no expected downstream losses, to the PMF for large dams with estimated significant human and economical losses.

B)-In the UK, if failure threatens the downstream life, the PMF is required, no matter how small the dam is. In addition to those requirements, dams are classified into 4 categories:

-A: Where lives in a community would be endangered by failure

-B: Where lives are possibly endangered by failure but not in community

-C: Where there is negligible risk to life and little damage

-D: Where there is clearly no loss of life and very little damage.

Moreover three levels of standard are recommended ([3], p.52):

1-A general standard (overtopping unacceptable)

2-An alternate standard (rare overtopping tolerable)

3-A minimum standard (economic analysis acceptable)

The acceptable design flood depends on both the category and standard of the dam.

How to Evaluate the Effects of a Dam Failure?

It is directly related to the extent of existing and future downstream development, size and type of dam. Careful considerations should be given to the following factors ([6], p.7):

- Quantities of stored water in the dam
- Reservoir inflow
- Size and shape of the expected breach
- Hydraulic head
- Time of breach formation

The extent of an affected downstream area by a flood wave resulting from a theoretical dam breach is a function of both the height of the flood wave and the downstream distance and width of the river at a particular location ([6], p.8). An associated and important factor is the flood wave travel time. These elements are not only a function of the rate and extent of dam failure, but also are functions of channel and floodplain geometry and roughness and channel slope ([6], p.8).
The flood wave should be routed downstream to the point where the effect of the failure will no longer have negative consequences.

There are several methods used for analyzing the dam break. These models will be discussed in the literature review section

Types and Forces on Different Dams

In this section, we will deal with all aspects of dams and the forces applied on them.

Selection Criteria for Dams

The dam’s choice at a certain location depends mainly on experience, judgment, topography and geology of the site. But, the existing conditions are the most critical in the dam’s selection, these conditions are ([9], p.40-43):

-Safety: not all types of dams are safe at any location: in other words, they are location sensitive.

- Cost of the hydraulic structure: this cost is mainly affected by the availability and cost of the needed construction materials. Additional funds vary enormously between one type of dam to another.

Types of Dams and Their Characteristics

The types of dams are ([9], p.40):

-Earth and rock embankments

-Solid gravity concrete dams.

-Buttress concrete dams

-Arched concrete dams

-Steel dams

-Timber dams

-Other types

In what follows, a review concerning each type of dam and the forces affecting it is presented ([9], p.40-43; [10]; [12]; [13]; [14]; [18]).

Embankment Dam

This type of dam is the most commonly built in the United States (see fig 5). If sufficient quantities of materials are present near the site, embankment can be constructed at a much lower cost than a concrete gravity dam. Usually, an earthen dam has a high ratio of length to height. It can be built at sites where the foundation is pervious. There are two categories of embankment: 1-earthfill dam that is made of fine materials, and 2-rockfill dam where the shells are made form rock.

An earthfill dam is feasible if:

(1) Suitable construction materials are available.

(2) An adequate amount of clay (for the impermeable core) is nearby.

(3) Usually built in a flat area.

A rockfill dam is feasible under the following conditions:

(1) The foundation is unreliable for sustaining the pressure on concrete dams.

(2) Suitable rock is nearby.

(3) An adequate amount of clay (for the impermeable core) is nearby.

(4) The dam’s site is wide enough for the manipulation of heavy earth moving machinery.

The problem of embankment dams is that they require large spillway for handling floods. Some spillways would require most of the length of the dam, leading to the infeasibility of the embankment.

If well maintained, an embankment dam should last for a long time.

This type of dams is also used in a region where it is required to preserve the natural look of the site because of its main components (earthen materials: clay, sand and rocks).

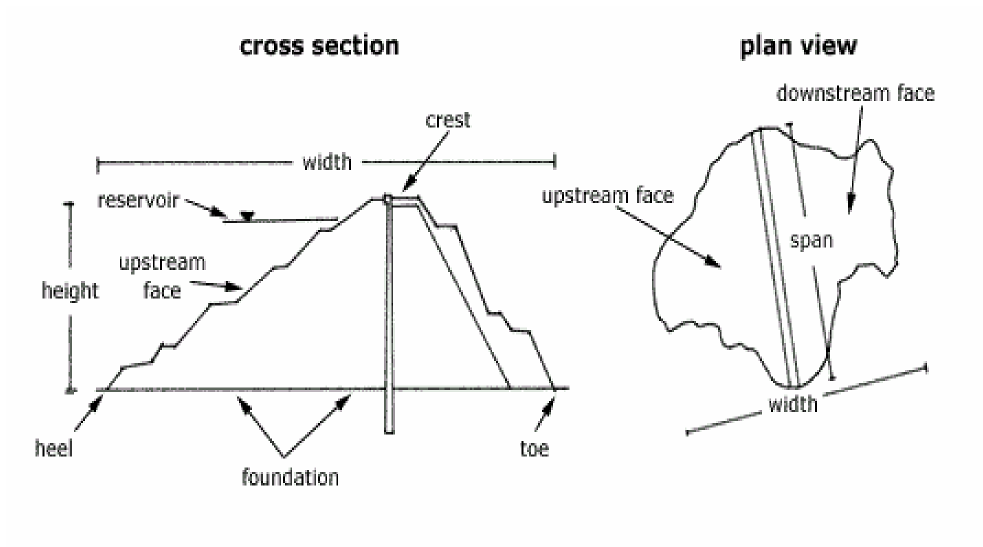

Figure 4.Embankment dam cross section ([11])

Embankment dams rely on their heavy weight to resist the force of the water (see fig 6). Inside embankment dams, there is an impervious region called core. This region has the role to stop the water from seeping through the dam.

The uplift force is directed upward, or in the opposite direction of the dam's weight. The problems facing this kind of dam (piping, overtopping, and erosion) will be discussed later.

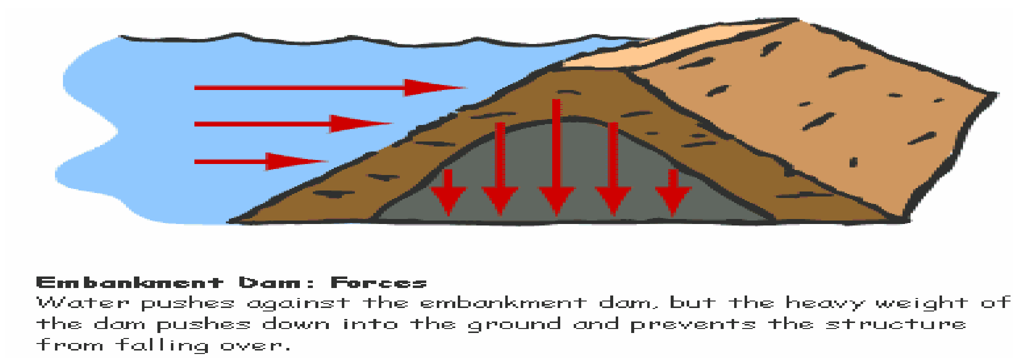

Figure 5.Forces applied on an embankment dam ([13])

Gravity Dam

The gravity concrete dam is the most common of all concrete dams and is considered the safest (see fig 7). The gravity dam resists the water entirely by its own weight. It should be well maintained in order to be effective and safe. Most gravity dams are expensive to build because they require huge amount of concrete.

A gravity dam can be built at any location, but its height is limited by the strength of the foundation. Therefore, if built on an earthen foundation, its height cannot be more than 30 m. A gravity dam is feasible if the length of the crest is at least five times the height of the dam .It has a concrete core mainly concentrated on its upstream face in order to reduce tensile stress due to bending and to obtain favorable gravity load. If the foundation is rock, and if the required materials are available, building an earthfill dam is more economical than a gravity concrete one.

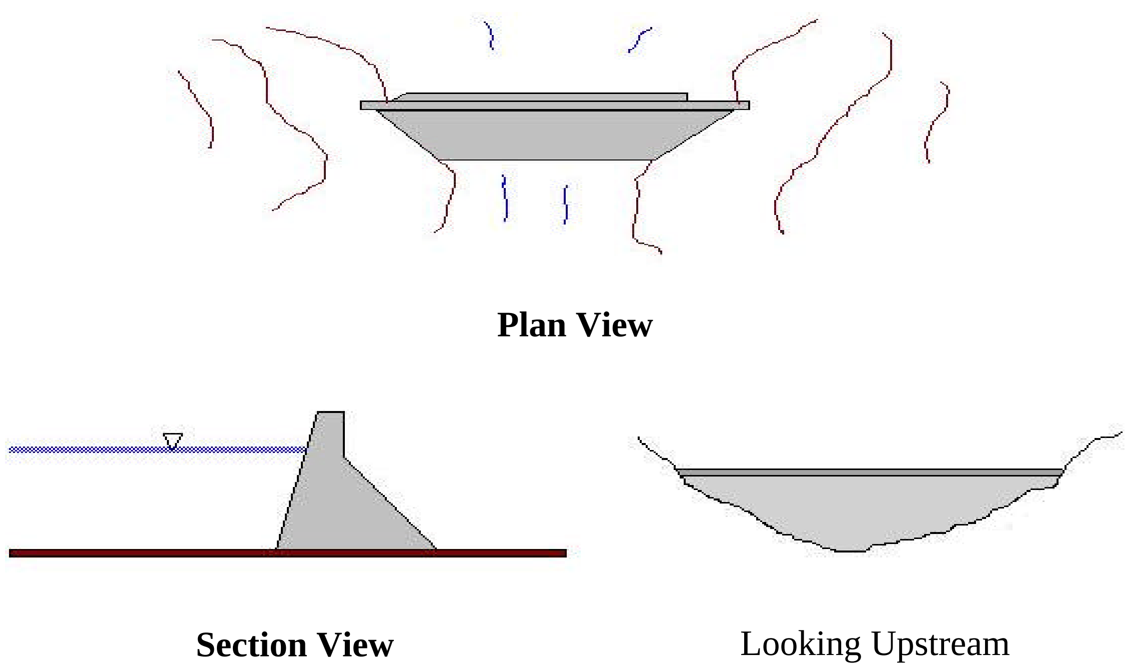

Figure 6.Gravity dam cross section ([19])

The forces applied on the gravity dam are (see fig 8):

- The thrust of water upstream
- The weight of the dam acting downward
- Sometimes the uplift force that is directed vertically upward.

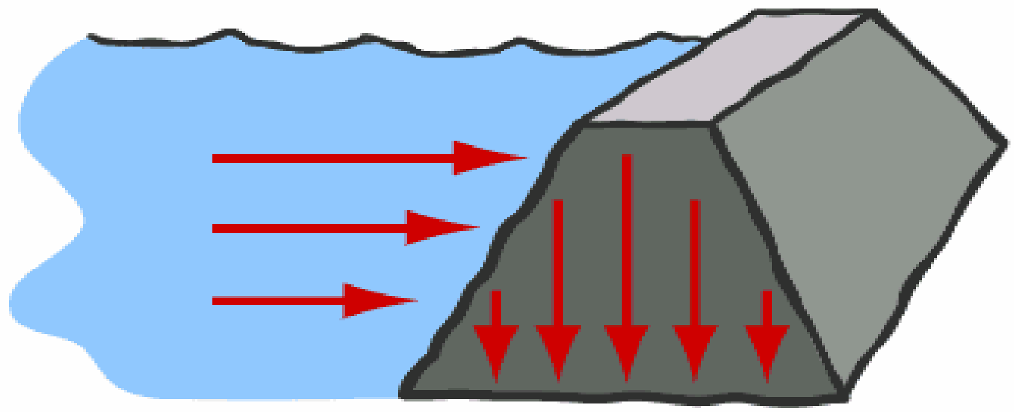
Gravity Dam: Forces
Water pushes against the gravity dam, but the heavy weight of the dam pushes down into the ground and prevents the structure from falling over.

Figure 7.Forces on a gravity dam ([13])

Arched Concrete Dam

This type is mainly used in narrow and deep valleys where the height is much larger than the dam’s length, and when the sides of the valley are made of hard rocks, which had to handle a large amount of stress (see fig 9).

Arched dams are not expensive because they require less material than a gravity dam but require good skills in order to place the formwork. The main difference between this type and the gravity dam is that the first one relies on the strength of the dam’s material as opposed to the latter that relies on the materials’ weight.

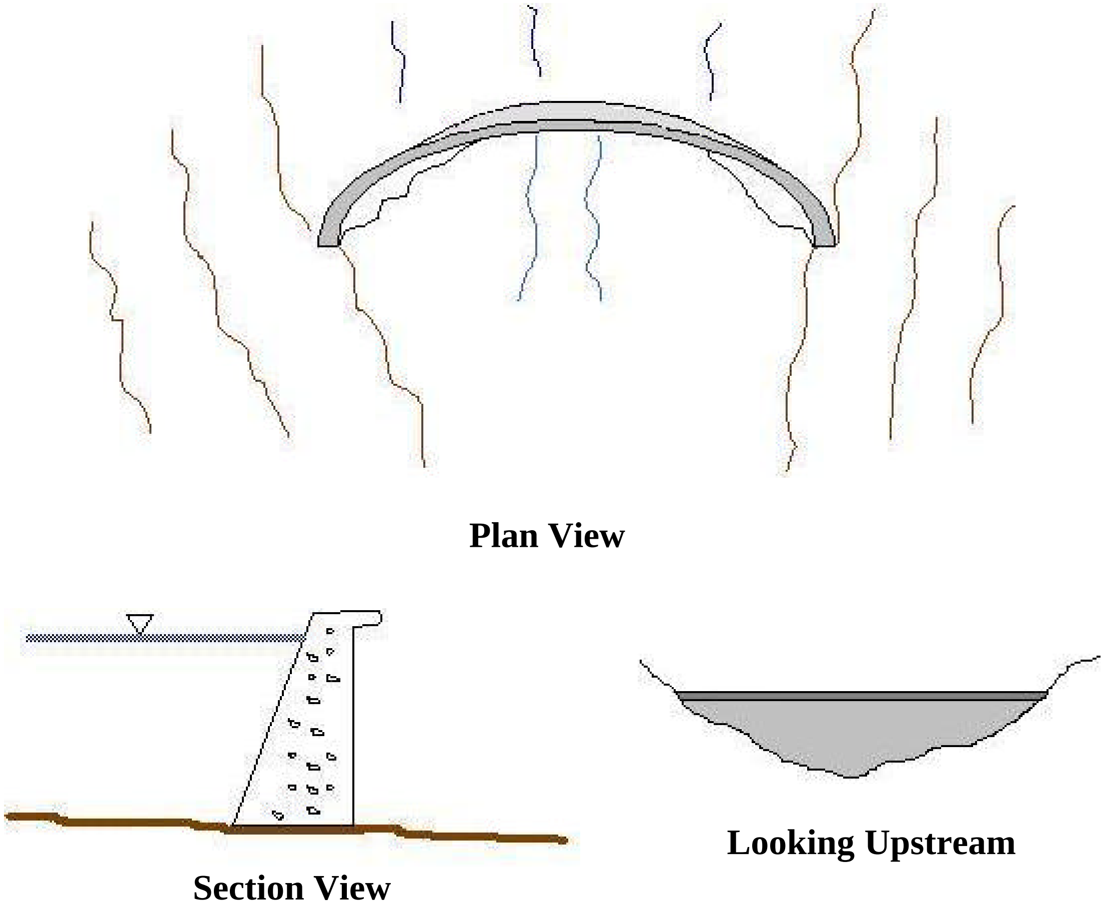

Figure 8.Arch dam cross section ([19])

This type uses the arch effect in order to resist loads placed on it. Therefore, its weight will not contribute enormously to the external loads resistance. That is why the uplift force on its base is not an important design factor (see fig 10).

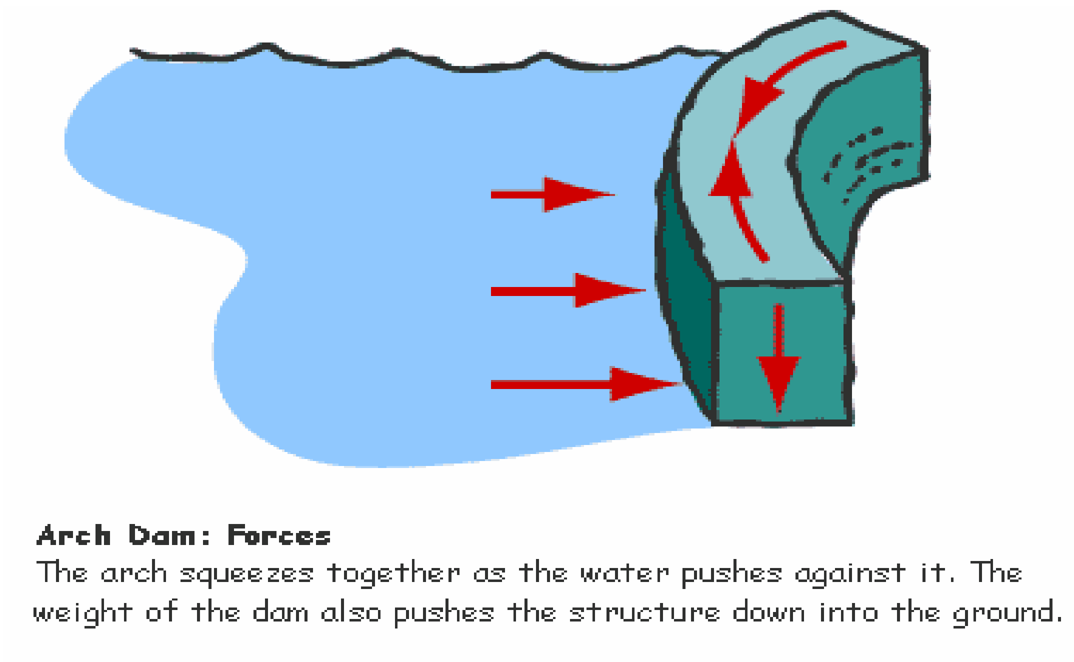

Figure 9.Forces applied on an arch dam ([13])

Buttress Dam

The buttress dam is often a combination of both the gravity and arch dams (see fig 11). It requires less concrete than a gravity concrete dam having the same volume; but needs more labor force. Buttress dams are much lighter than gravity ones, thus they exert less pressure on the foundations. Compared to an arched dam, a buttress dam does not require strong sides.

Its main disadvantage is the deterioration of concrete due to the stored water. This is not very frequent in a thick gravity dam.

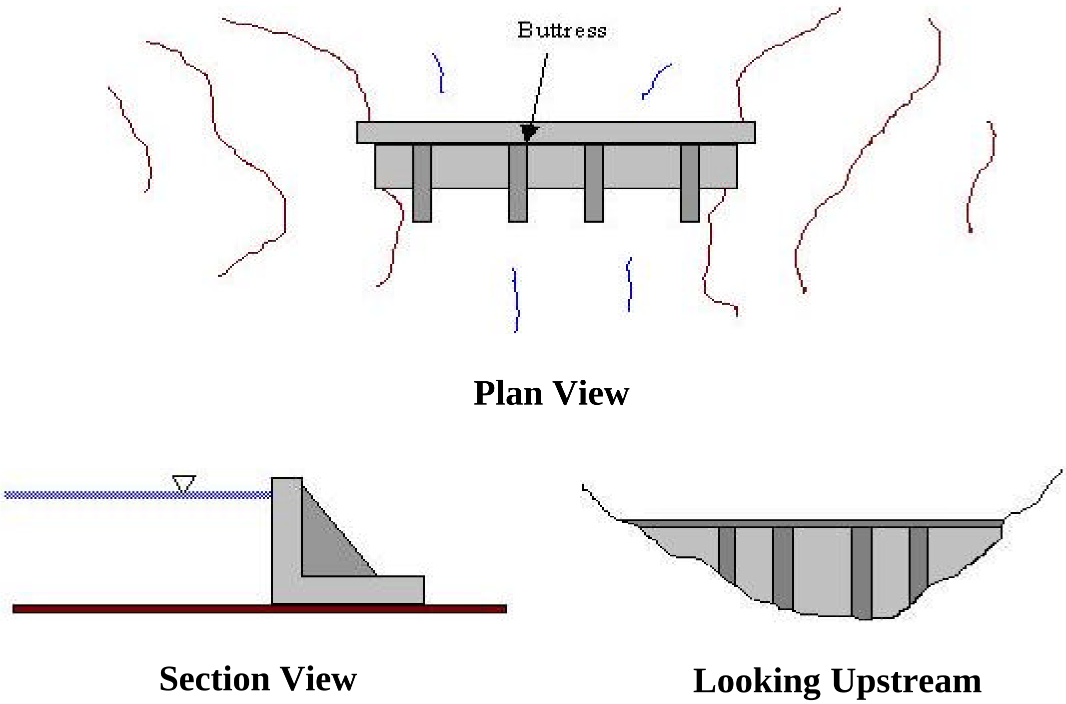

Figure 10.Buttress dam cross section ([19])

The uplift force at the base of a buttressed dam is negligible because of the effect of the buttress situated at the downstream side of the dam (see fig 12). One advantage of the buttress dam is that it does not overturn because of the batters.

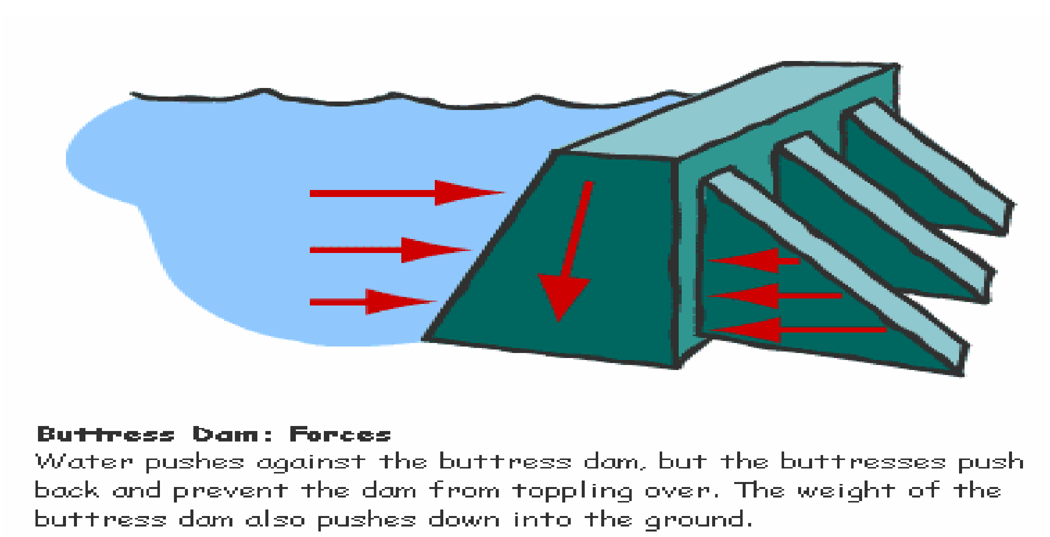

Figure 11.Forces on a buttress dam ([13])

Multiple Arch Dam

The multiple arch concrete dam is a combination of both buttress and arch dams (see fig 13). Its foundation need not be as strong as that of concrete arch dams. In building such a dam, less concrete quantities are needed than a buttress dam. If one part of this type fails, the whole structure will fail as well. From an economical point of view, both multiple arch dams and buttress ones are similar.

Concerning the uplift force, as in buttress dam, its effect is negligible due to the force exerted by the batter. Corrosion of these dams, like in buttress, is a major problem.

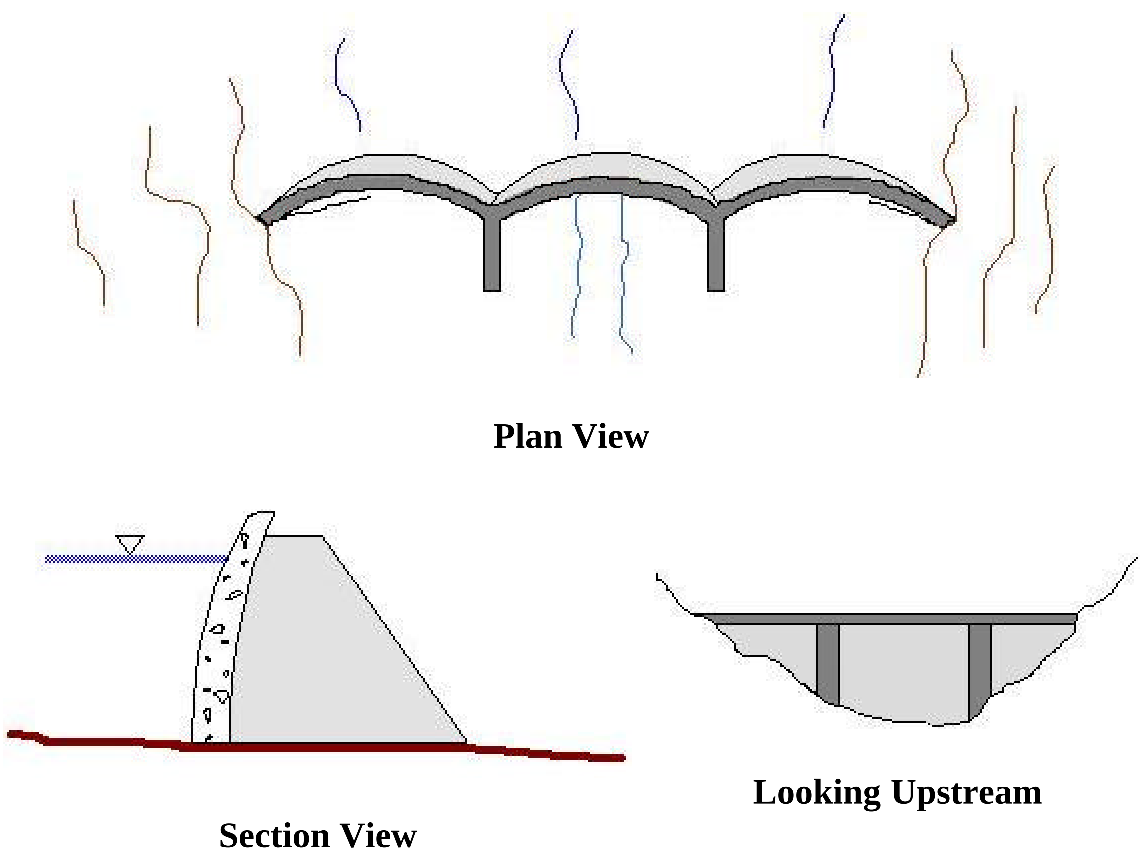

Figure 12. Multiple arch dam cross section ([19])

Steel Dam

This type is rarely used around the world. This type of dams is thought to be economical, but its problem is that it needs to be well anchored to the foundation ([9], p.43).

Timber Dam

The most temporary type, although if it is well designed, constructed and maintained, it may last more than 50 years. But its maintenance costs are very high ([9], p.43).

Hydraulic Fill Dam

Hydraulic fill dams are suitable in valleys of soft materials and are constructed by pumping soft material up to moderate heights up to 100 ft ([14]).

Composite Dam

Sometimes, due to geological and topographical aspects of the sites, one dam can be of different types ([12]).

Causes and Solutions of Dam Failures

The main causes of failures of dams are ([11]; [17]):

- Overtopping
- Sliding
- Piping

. Internal seepage

. Overturning

. Overstressing

· Cracking

. Bearing capacity

· Maintenance

. Rapid drawdown

The percentage of failed dam in the world was about 2.1 before 1950, and around 0.3 after 1950([17]).

On average around 40 % of dam failures occurred due to overtopping where the design flood has been exceeded (see table 6).

Table 6.Reported causes of dam failures ([17])

Cause	Middlebrooks (1955)	Bureau of Reclamation (1984)
Erosion, piping &conduit failure	38%	37%
Overtopping (inadequate spillway capacity)	30%	40%
Slope instability &slope protection failure	20%	23%
Unknown causes	12%	-

In the following, a discussion of these mechanisms and their solutions is given.

Overtopping

When water passes above the dam's crest, the dam will be gradually washed away for an embankment dam whereas it will be destroyed for a concrete dam. Another but rare cause for overtopping is when an earthquake hits the region where the dam is located creating a large water wave that can pass above the dam's crest.

This type of failure may occur in any type of dams, but it is mostly dangerous for embankment dams because it washes away or erodes very quickly the dam's materials.

In order to prevent overtopping, the dam's height should be designed in a manner that it can handle the maximum expected conceivable flood (i.e. PMF). Moreover, the difference between the height of the dam and the expected height of the water behind the dam should be between 2 and10 ft. This distance is called freeboard and it represents a factor of safety against unexpected events. Also, the design height should account for the highest expected wave in the dam that can be caused by wind or earthquake.

Sliding

One of the reasons of sliding is the uplift pressure that is applied on the dam by the water seeping below its foundations. This will cause the dam to be uplifted but the dam's weight will act against the uplift force and in the opposite direction.

Normally, the main forces that influence the sliding behavior are ([7], p.398) (see fig14):

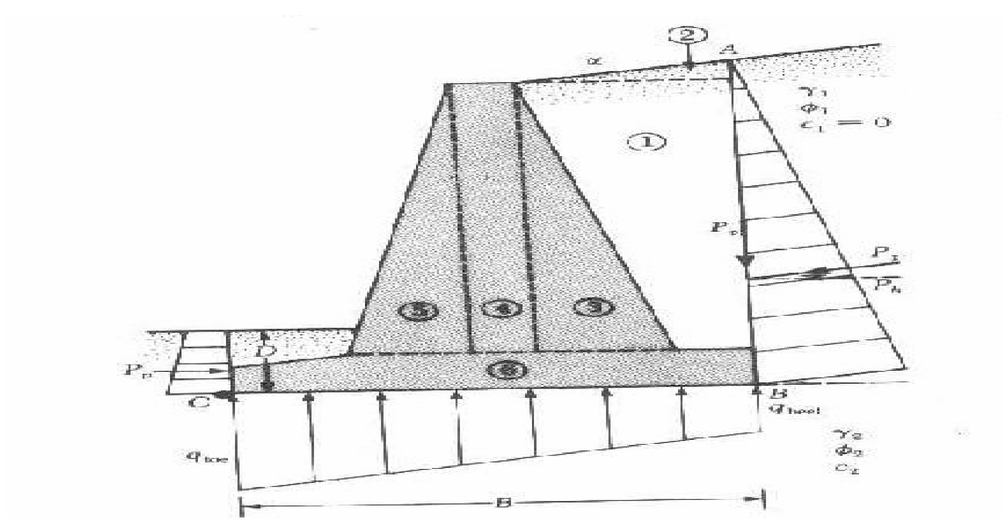

Figure 13.The main forces that may affect sliding ([7], p.396)

1-The active force, P_h that is exerted on the upstream side of the dam and is caused by the upstream water level and the upstream soil pressure. The active force is mainly a hydrostatic driving force for sliding.

2-The passive force that is applied on the downstream face of the dam is mainly represented as a hydrostatic force and /or a force applied by the soil. The passive force is a resisting force against sliding which acts in the opposite direction of the active force.

3-The weight of the dam that is concentrated at its center of gravity.

4-The uplift force exerted by the water seeping under the dam: this force depends on the height of the water stored behind the dam.

Therefore we can say that the higher the levels of water behind the dam, the bigger the hydrostatic force and uplift force is.

In order to decrease the effect of the uplift force that is caused by the uplift pressure, the design should either decrease the uplift pressure or increase the weight of the dam that usually acts in the opposite direction of the uplift force.

Most of the time, the first solution (decrease of pore pressure) is applied. This can be achieved by:

1- Introducing a drainage system (see fig 15):

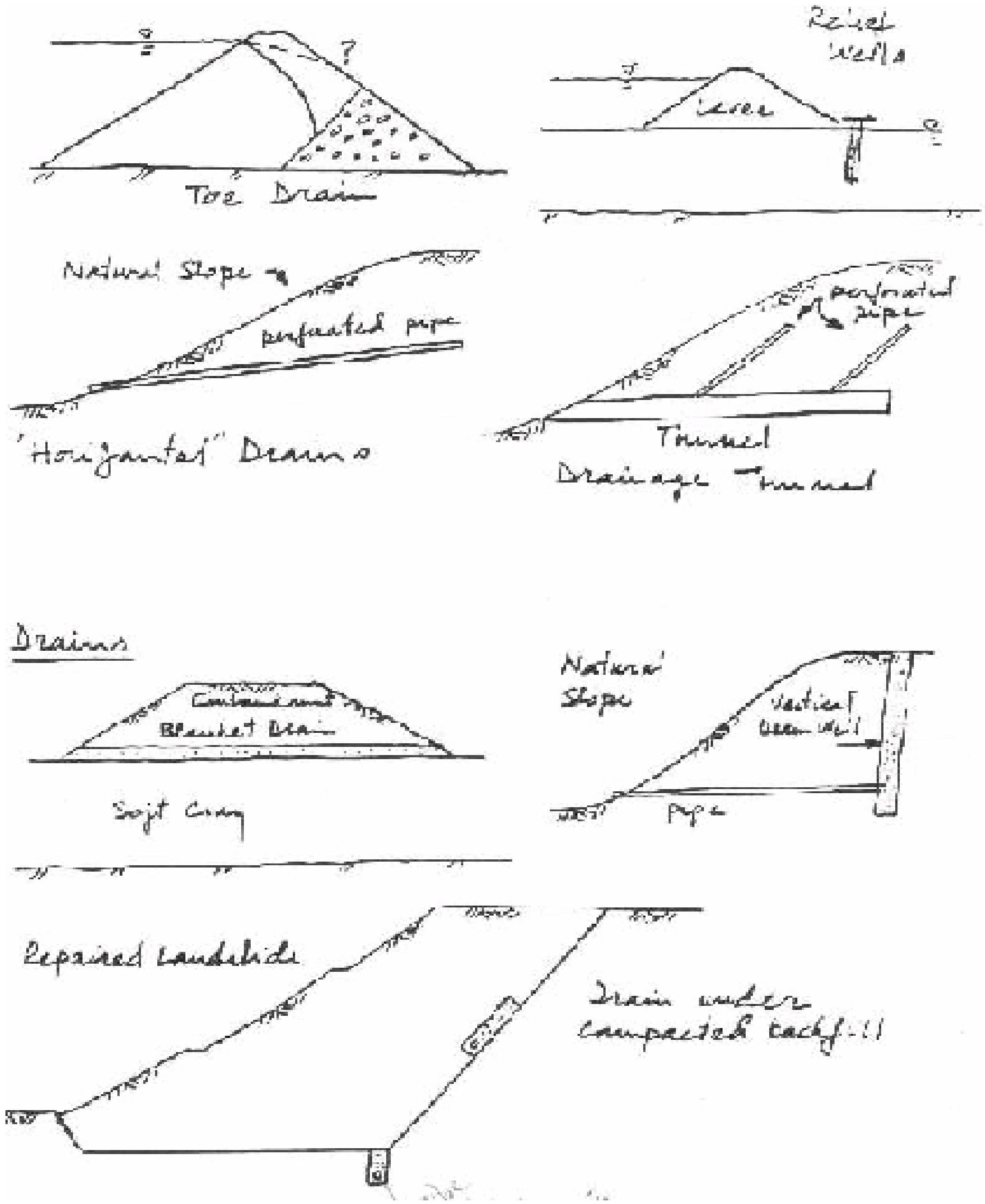

Figure 14. Drainage system ([17])

2- Introducing a deep impermeable sheet or cutoff wall at the upstream side of the dam that will lengthen the water path, leading to the decrease in the head consequently in the uplift pressure (see fig 16).

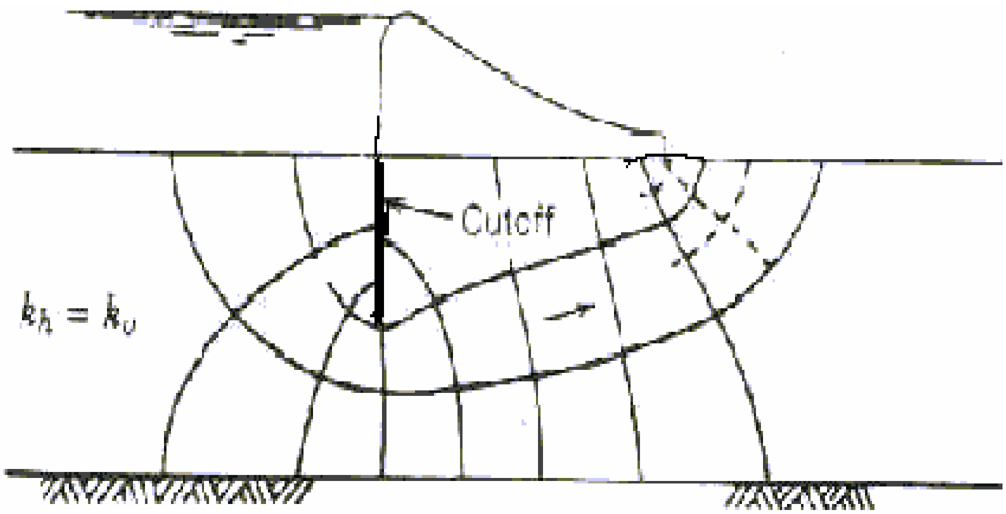

Figure 15. Sheet wall ([17])

For more information concerning sliding of dams, the factor of safety analysis against sliding is included in this report (see page 35).

Piping

When water seeps under the dam, with time, it may begin eroding the soil at the downstream side. As time elapses, this erosion may expand by moving gradually from the downstream to the upstream side of the dam creating a cavity (tunnel) under the dam (see fig 17). This tunnel, when expended, may cause failure of the dam.

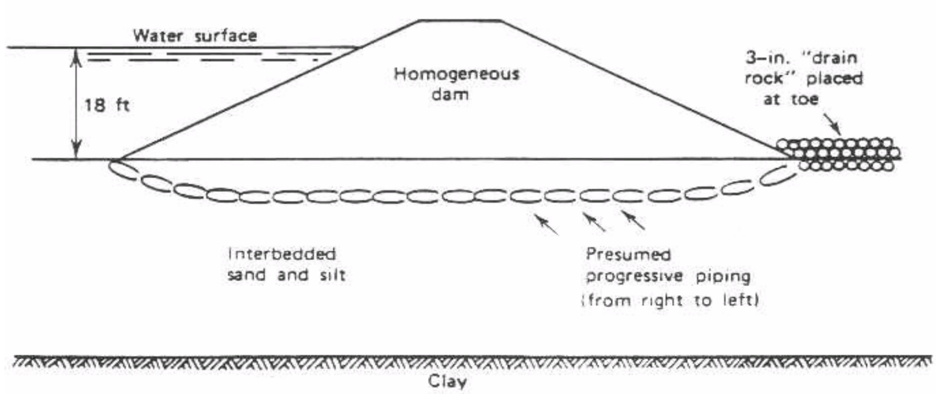

Figure 16.Piping mechanism and solution {filter) at the downstream ([17])

The extent of this seepage depends mainly on the type of soil underneath the dam, for example: a dam built on a rocky foundation will be safer against piping than a dam built on a sandy or loose soil. In other words, piping depends mainly on the hydraulic conductivity of the soil. This phenomenon may occur under any type of dam.

The most common solution (see fig 17) is the installation of a graded filter at this troublesome downstream region that enables the water to seep but, at the same time, it prevents the soil particles from being washed away.

Internal Seepage

This mode of dam failure depends on the type of dam.

For all types of dams, the internal seepage depends on the dam's construction quality, such as:

A-The conditions of the terrain: relief - flat, sloping, surface - smooth or rough, soft or hard.

B-The stiffness of the structure: the stiffer the structure is, the lower the seepage will be.

C-The skill and experience of the construction teams: the more experienced they are, the lesser the effects of seepage will be.

For concrete dams, the main causes of this internal seepage are:

1-The number of segments or junctions along the length of the dam: the less the number of junctions, the safer the structure is.

2-The measures used for sealing junctions: if junction sealing is not perfect, seepage between the junctions may take place.

In embankment dams, there is internal seepage (see fig 18). Sometimes, this internal seepage may cause erosion because water is able to seep through the dam's upstream shell to pass then through its relatively impermeable core. This erosion is mainly concentrated between the downstream shell of the dam and its core. This occurs, because the core's particles are much smaller than that of the downstream shell, thus the water passing through the core to the downstream shell may transport with it these tiny particles. This will cause the gradual loss of impermeable core's particles leading to its erosion. Therefore, with time, the core's particles will be washed away leading to the failure of the dam.

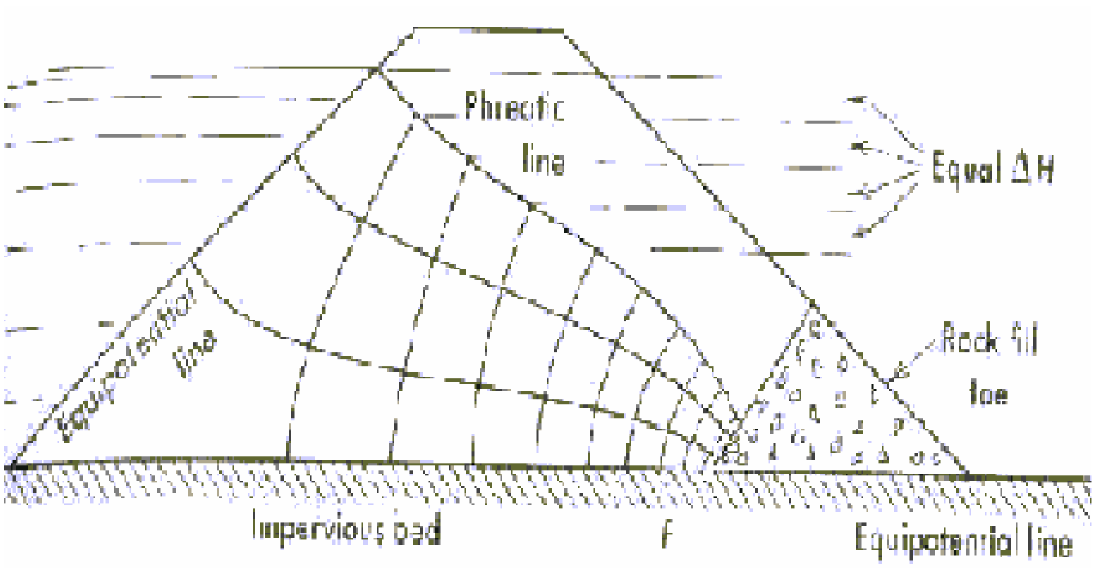

Figure 17.Internal seepage in an embankment dam ([17])

For all types of concrete dams, internal erosion may take place when there is, for a certain reason, an opening in the structure in which water can pass. With time, water will enlarge this opening by eroding the concrete and attacking the steel. The solution of this problem is by periodically maintaining the structure and closing these openings.

In a zoned embankment dam, in order to prohibit the core's fine particles to leave the core, a filter should be placed between the core and the downstream shell of a zoned dam (see fig 19). The role of this filter is to stop the fine core's particles to be transported but at the same it enables the water to seep as usual (see fig 20). If the dam is not zoned (refer to fig 18) a rock toe can be put at the toe of the dam in order to let water seeping through the dam go out.

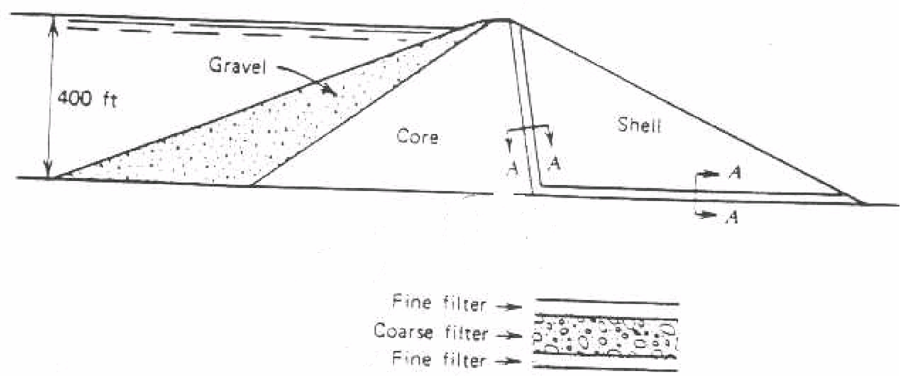

Figure 18.The filter in a zoned dam ([17])

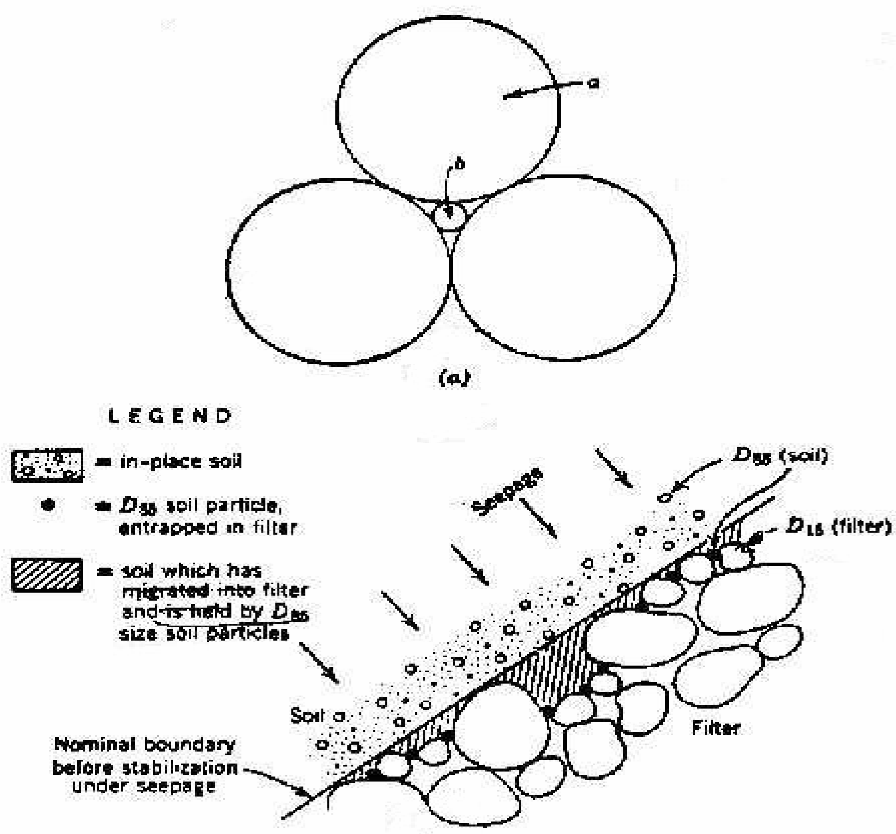

Figure 19. Illustration of how core’s fine particles are entrapped by the filter ([17])

For all types of concrete dams, internal erosion may take place when there is, for a certain reason, an opening in the structure in which water can pass. With time, water will enlarge this opening by eroding the concrete and attacking the steel. The solution of this problem is by periodically maintaining the structure and closing these openings.

Overturning

It is mainly caused by the imbalance of the acting moments on the structure. Design against overturning is not crucial for embankment dams because the structure does not behave as one body. Overturning is mostly frequent in gravity dams due to the materials forming it (concrete and steel), which make it one entity ([7], p.396). The forces discussed in the sliding section are the same that affect overturning (see fig 14).

In order to be safe against overturning, the resisting moments and the overturning ones should be balanced. The analysis for the factor of safety against overturning is discussed later in this report in page 35.

Overstressing

As flood flows enter the reservoir, the water level in the reservoir will rise, causing a sudden increase in the loading status on the dam ([6], p.5). If the Dam is not designed to sustain such event, either the whole dam or a part of it will be overstressed, leading to an overturning, a sliding or a failure of a specific structural components ([6], p.5).

Concerning the Embankment dams, they may be in danger if the increased water levels results in increasing pore pressures and seepage rates, which exceed that of the design ([6], p.5).

Cracking

This can be caused by movements such as the natural settling of a dam or due to an earthquake that hit the dam. In this case, the dam is weakened and cracks appear leading to the dam failure.

If a crack takes place in the dam, it should be directly cured and repaired in order to prevent its enlargement with time that may lead to the dam's failure.

Bearing Capacity

If the foundation on which the dam is built cannot hold the dam anymore for several reasons like it has become weak with time, the dam will be subject to failure. As long as the dam is in place, the foundation should be able to bear it. Otherwise the failure will occur.

Maintenance

If a dam is not well maintained (i.e. the clogging of the gates by the sediments) the dam will be subject to failure. Removing sediments from the stream, checking the outlet structures of the dam and other steps are the main solutions for maintaining a dam.

Rapid Drawdown

This is mostly significant in embankment dams, where its materials are made from disconnected soil particles (sand or rock). An example of rapid drawdown is the emptying of a reservoir at a very fast rate, leading to a landslide in the upstream face of the dam.

In order to prevent rapid drawdown, an embankment dam should not be emptied in a very abrupt way, it should be gradual.

How Each Type of Dams Fails?

Table 7 shows the likely failure modes fro the different types of dams (X means that this type of dams fails under this mode).

Table 7.Modes of failure that can hit different types of dams

	Embankment	Gravity	Buttress	Arch	Multiple Arch
Sliding	X	X	X		
Piping	X	X	X	X	X
Overturning		X		X	
Overtopping	X	X	X	X	X
Maintenance	X	X	X	X	X
Cracking	X	X	X	X	X
Rapid drawdown	X				
Internal erosion	X	X	X	X	X

Factors of Safety

The factors of safety are crucial in the design of dams. The most known factors of safety for dams are: 1-sliding 2-overturning 3-bearing capacity. For the sake of giving a general idea about the factor of safety, the discussion will be focused on a gravity concrete dam. But to be consistent, the factors of safety apply also to different types of dams (refer to table 4).

Forces on a Gravity Dam

The major forces acting on a gravity dam are given in figure 21([5], p.230-234):

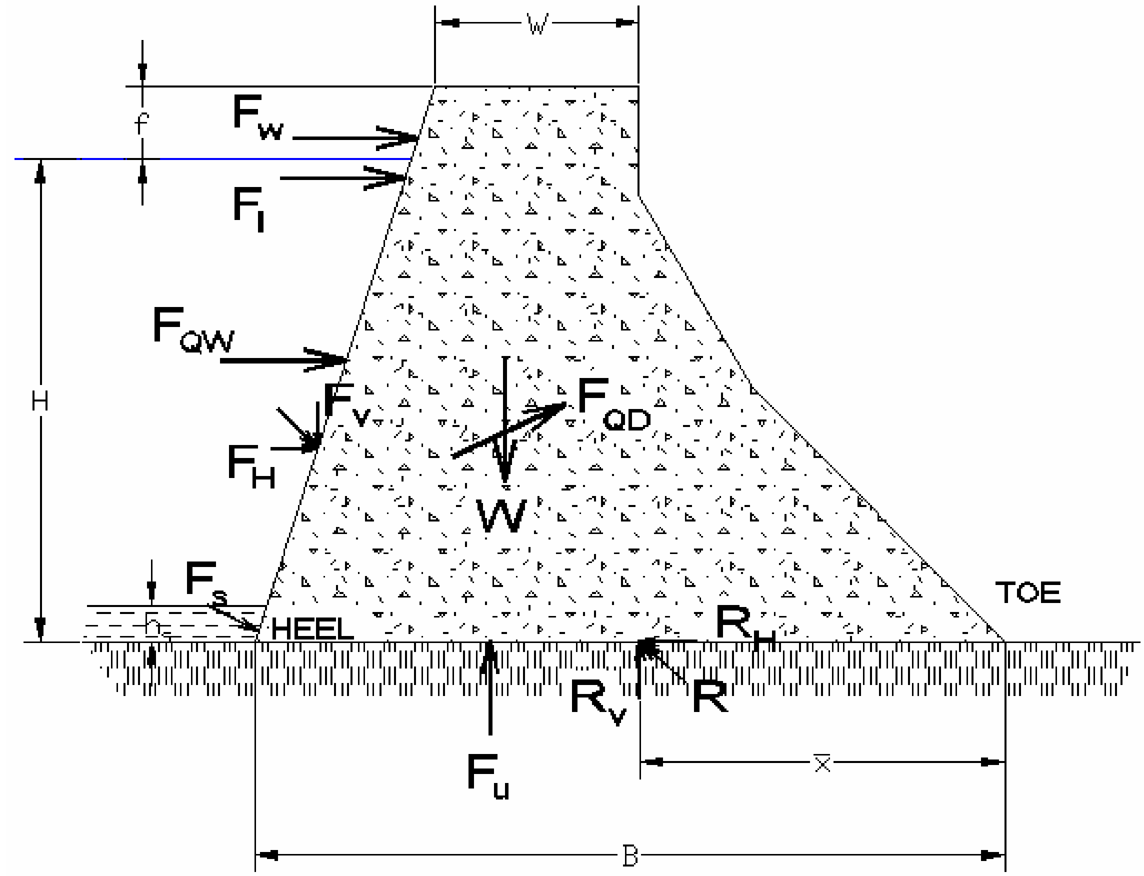

Figure 20. Forces on a gravity dam ([5], p.230)

Where:

1. W= the weight of the dam. In this case $W=1/2\gamma HBS$ where S= specific gravity of concrete.

2. F_U=the uplift force on the base of the dam which is found by using flow net analysis below the dam.

3. F_H =the horizontal force of hydrostatic pressure, acting along a line H/3 above the base $=1/2\gamma H^2$, where γ =water specific weight

4. F_V =the vertical force of the hydrostatic pressure = Weight of fluid mass vertically above the upstream face, acting through the center of gravity of that water mass.

5. F_{QW} = the earthquake force due to acceleration of water behind the dam

6. F_I= the force caused by ice on the surface of the lake against dam which is approximately equal to $5000h_I$, where h_I is the freezing depth.

7. F_S= additional hydrostatic force due to silt deposits near the heel, which is approximated by $1/2\gamma\ (S_s\text{-}1)h_s^2$ with h_s is the depth of silt and S_s is the specific gravity of the water-silt mixture.

8. F_{QD} = force caused by an earthquake applied on the dam

= (W/g)a, where the acceleration is usually taken as 0.1g in the horizontal direction, and about (1/12)g in the vertical direction, acting the center of gravity of the dam.

9. F_{QW} = the earthquake force due to acceleration of water behind the dam.

10. F_I= the force caused by ice on the surface of the lake against dam which is approximated to be $5000h_I$, where h_I is the freezing depth.

11. R = reaction of the ground.

From the horizontal forces equilibrium we have:

$R_H\text{-}\ F_H – F_S – F_{QD} – F_{QW}\ \text{-}\ F_I = 0$

In which: R_H = the horizontal component of the ground reaction R.

And for the vertical forces equilibrium, we have:

$R_V – W – F_V – F_U – F_{QD} = 0$

In which: R_V = vertical component of the ground reaction.

In what follows, in order to simplify the analysis, some minor forces will be dropped. The ground reaction R acts at a distance "x^-" from the toe that needs to be determined. When there is impending motion, there is frictional resistance $R_{H,\ friction} = R_V m$ where R_V is the normal force transmitted across the surface of contact.

In order to check the stability of a dam, the dam's design should include the calculation of the factor of safety against sliding, overturning and bearing capacity.

Here we will only focus on both sliding and overturning.

Sliding

The factor of safety against sliding is a measure of the forces needed to overcome the frictional resistance. It is defined as ([5], p.232):

$$FS_{sliding} = \frac{mR_V}{R_H}$$

Or in geotechnical engineering, it can be written as ([7], p.398):

$$FS_{\ sliding} = (W-F_u)\ tand\ /F_H$$

Where δ is an angle dependant on the foundations 'characteristics. On average tanδ has a value equal to 0.4.

In order to be safe against sliding, the factor of safety against sliding is recommended to be bigger than 1.5.

Overturning

The factor of safety against overturning about the toe is defined as the ratio of the resisting moments to the overturning ones.

The resisting moments are all moments with counterclockwise direction. In this case (see fig 21):

$M_{resisting} = Wa + F_v b$ (2)

Where a and b are the distances from the toe to the lines of action of W and F_v respectively.

The overturning moments tend to topple the dam about its toe. These moments are all clockwise moments .In this case:

$M_{overturning} = F_H Z_H$ (3)

Where Z_H is the vertical distance from the point of application of F_H to the toe.

If the dam were to overturn, R would move to the toe and will not have any moment about the toe due to R (and in particular due to R_V). Therefore, the factor of safety against overturning is given by:

$$FS_{overturning} = \frac{M_{resisting}}{M_{Overturning}}$$

Or

$$FS_{overturning} = \frac{Wa + F_v b}{(Wa + F_v b) - R_v x}$$

Usually, the factor of safety against overturning should be between 2 and 3 in order to be safe.

Environmental Effects of Dams

The human interference near or on a stream, may introduce a drastic change to the ecology and species living there. The more the development on a river, the less rich will the river be in organisms and nutrients. Therefore, building a dam in a river will change significantly the river's biology and ecology. On the other hand, there is a tight link between the environment and the population's social needs. These effects are discussed in the following.

Loss of Aquatic Habitats and Fish

By definition, a habitat is a place where a plant or animal grows or live naturally. By building a dam, the habitat in stream channels will be inundated leading to the extinction of some aquatic species. Most often, after building the dam, different types of fish and aquatic species will substitute the original ones ([1], p.73).

Wildlife Habitats Loss

Many wild species living in the streams eat from the vegetation that is along and near the stream or from trees and brush ([1], p.73). When a dam is built, all those food sources will be inundated, consequently lost. This may lead to the extinction of animals feeding from these sources.

Enhancing other areas along the stream may solve this problem, but this is very expensive.

The Change of the Channel's Geometry

Usually, a decrease in the peak flow of a river will cause a decrease in the river width. This usually occurs due to ([20]):

- The low floods that pass the river that are unable to scour its sides

- The sediments transported by the tributary channel that will coalesce and encroach to the sides of the main channel.

Sometimes, if tributary channel are present downstream, in addition to the narrowing of the river, there will be is a decrease in its depth (see fig 22). This occurs due to the accumulation of sediments (resulting from the tributaries) in the river where they cannot be transported further due to the low flow.

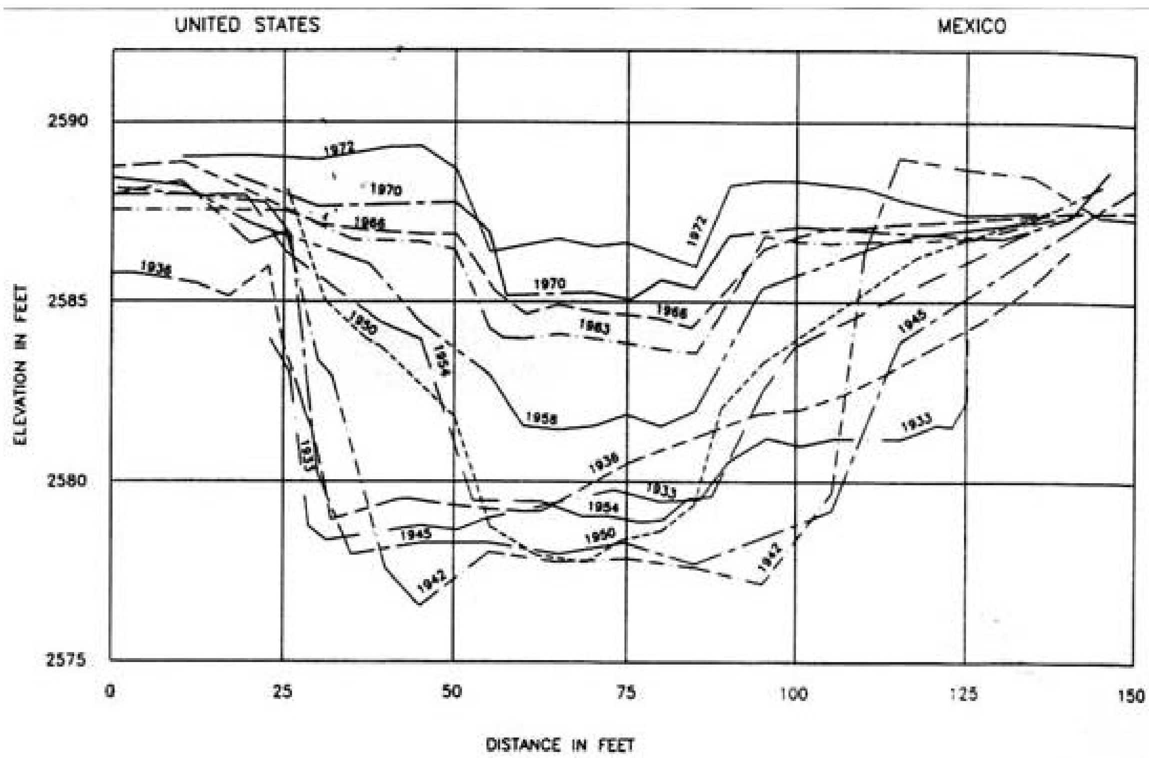

Figure 21.Example of the change, with time, of the cross section of a river downstream of a dam ([20])

This decrease in the river's geometry will affect negatively the people living on its sides. The negative effect arises if a high flood passes the river; in this case, the narrow channel will not be able to handle this flood that may cause the inundation of the area.

Bed Degradation

Because of the dam, most often streambeds are scoured and their elevation is decreased (see fig 23)([20]). Near the dam, high velocity of water is the main cause for scour. Further downstream, the deficiency of needed sediments that are accumulated behind the dam will cause scouring. Because river needs sediments, and these sediments are not available, it will scour its streambed as a way for compensation.

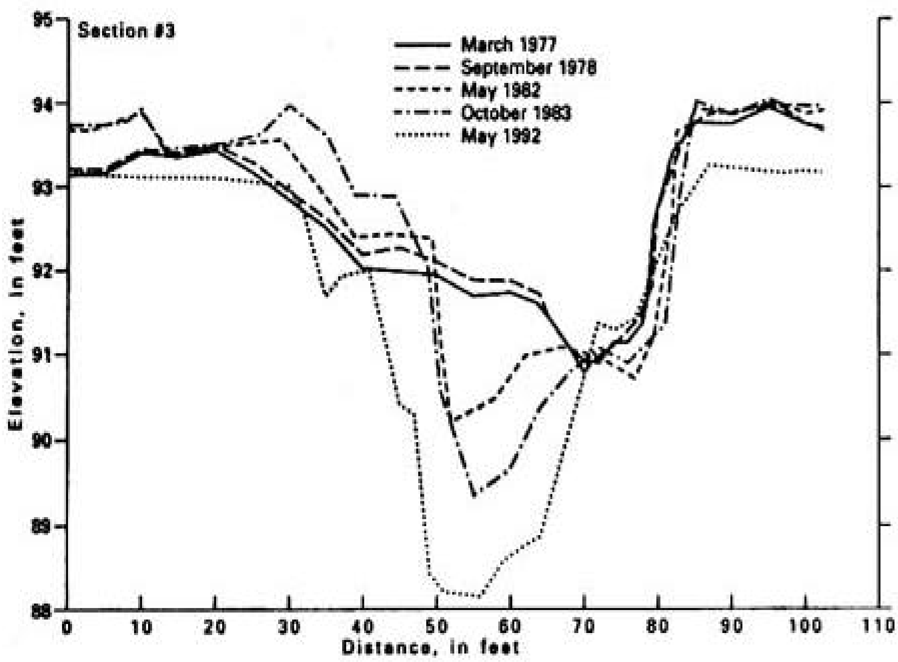

Figure 22.An example of narrowing and deepening of a channel in 15 years time span ([20])

This phenomenon causes a problem for bridges that have foundation in these eroded streambeds. It seems that the biggest degradation occurs near the dam and as you go downstream, degradation will decrease. This means that the decrease of the elevation near the dam is bigger than that of the downstream. It leads to a decrease in the river slope.

Bed Armoring

Building a dam will cause degradation of particles: the average river particle size will increase ([20]). This size will be transported downstream, whereas everything bigger than that will accumulate. These accumulated particles will, with time, stick together and form an armor, with very high strength, on the river bed. For example, On Bear Creek in Colorado, the mean particle size increased from 0.48 millimeters before the dam, to 30 millimeters after only 5 years of dam operation.

Bed armoring is not always bad because there are some examples where those new rigid beds can decrease appreciably the effect of bed erosion. On the other hand, the bad effect of bed armoring is the formation of rapids. These rapids, by the reduction of peak flow, will become more stable. More stable rapids lead to more water turbulence. This issue is unsafe if the main channel is connected to tributary channels. Those tributary channels transport, to the main channel, relatively big particles that will be deposited there because of the low flows. These large particles (i.e. boulders) will be very dangerous for rafters. Moreover rapids will become more dangerous with time.

Loss of Access to Minerals

A mine, located in an anticipated reservoir site, would be closed after the construction of this dam. This action would cause an economical loss ([1], p.74).

A typical example of environmental loss is the inundation of deposits within a reservoir site that are not currently used but will be of potential value and need in the future. Studies projecting the future use of these resources should be conducted before building the dam depending on both the future possible needs for these minerals and their availability elsewhere.

Loss of Flat Areas in Mountainous Terrain

Usually, in mountainous regions, dams are built on flat areas. This represents both an economic and environmental loss, because people would use later such flat areas for different purposes ([1], p.74).

The Inundation of Historical and Archeological Sites

Most of the important archeological sites have been found on relatively high altitudes because the ancient rulers preferred to construct these sites in such places in order to be more able to resist the enemy ([1], p.75). But in order to have sufficient water supply for surviving, these sites were mostly located near rivers. The construction of a dam on these rivers will perhaps inundate part or most of these archeological sites. Therefore at such sites, an archeological survey should be conducted in order to know if this reservoir site is of an archeological value. Small artifacts may be taken to museums, but large archeological features may create a big problem for the construction of the dam.

A typical example is the construction of the Aswan dam on the Nile River in Egypt that was supposed to fully inundate great monuments of the old Egyptian civilization. The most important of these large monuments were removed to locations that are slightly above the maximum expected headwater of the reservoir.

The Inundation of Important Geological Formations

A reservoir site may inundate fully or partially some important geological features such as ([1], p.74): Waterfalls, large springs, geothermal displays and caves. A typical example is a reservoir in the Colorado River that was not constructed because part of the Grand Canyon national park would be inundated.

Aquatic Life Reduction

During the construction of a dam, the river will be diverted causing the reduction of the water level downstream, which in its turn reduces the downstream fish and aquatic habitats ([1], p.75). If all water is diverted, the river's downstream will be completely dry causing the death of many species in the river.

On the other hand, dam can, in dry seasons, be a relief for such species where the river is dry and the release of water from dams will let these species able to survive.

Reduction of Flushing Flows in the Stream

When a dam is built, most large flows are controlled and blocked by the dam ([1], p.75). Usually, without the presence of a dam, these large flows flush and transport the sediments the farthest possible downstream. But after blocking these flows by the dam, this flushing capability is lost. Several tributary streams connected to the river and carrying sediments, will reach the main stream and will deposit these sediments in it. These sediments, with time, will accumulate there because of lack of flushing flow.

Change of Water Quality

The parameters of water quality that are mainly affected by the building of a dam are: oxygen content, organic matter content, turbidity and temperature ([1], p.75).

1-Oxygen content: dissolved oxygen in the stored water behind the reservoir may deplete due to the decomposition of organic substance in the water. Deep zones of the reservoir have less oxygen content. The amount of available oxygen depends on organic materials present in the reservoir.

2-Organic matter content: decomposition of organic matter in the reservoir enhances the nutrients in water. This decomposition forms gases that may lead to localized pollution in the reservoir.

3-Turbidity: water stored in a reservoir contains initially sediments that vary from fine to coarse particles. The coarse particles settle much faster than the finer ones.

Under normal conditions, water released from a dam is slightly clear because it is drawn from its bottom where all particles have settled. But if a flood takes place, the stored water will become turbid.

The solution for this problem is the use of selective level outlets that may stabilize the turbidity and water temperature of the reservoir's released water.

4-Temperature: in a reservoir, the water at the top is warm and becomes colder as you go down. But the released water from the reservoir has a different temperature than that of the natural flow. Usually, water is drawn from low depth from the reservoir, thus the released water is colder than the natural stream flow.

Blocking the Way for Anadromous Fish

Anadromous fishes are the ones that migrate from the sea or ocean to a river and vice versa ([1], p.77). Dams are built in many of these streams. These structures represent a barrier for such fishes because they block their migration routes. The best solution for this problem is Fish hatcheries that are constructed at or near dams in order to maintain the fish in numbers at least equal to those existing under previous natural conditions. Another case is the fishes that migrate within the same stream going from its upper part to its lower one and vice versa. The same problems discussed for anadromous fish will occur. Also the hatcheries represent a suitable solution for such problem.

Blocking the Migration Routes of Species

Several animals like deers migrate, according to the season, in order to search for food ([1], p.77). Sometimes the migrations routes pass through a stream or tributary channel. But after building a dam on these streams, the route will be disturbed. For example, many animals can cross a river with low water level but they cannot cross a reservoir with high water level. Therefore, building dams may change or disturb the life style of some animals.

Unsightly Excavation and Waste Sites

Not all dam's construction materials are available on site. Therefore, these materials will be imported from other sites ([1], p.78). Also there will be a lot of cut materials (wastes) due to the construction of a dam such as foundations excavation .The cut materials should be properly disposed.

Erosion Caused by Temporary Roads

Building temporary roads near a dam is usually done without satisfying the minimum roads' requirements such as the presence of a drainage system ([1], p.78). If these temporary roads remain in place after the dam is completed, they will cause a lot of erosion because no minimum provisions are applied for the roads. Moreover, roads constructed on steep slopes near the dam abutments may cause landslide or rocks' slide. The solution for these problems is the construction of roads satisfying the minimum requirements such as the presence of a drainage system.

Changes in Vegetation Due to the Reduced Downstream Flows

When a dam is constructed, many downstream types of vegetations will be replaced by new ones ([20]). Usually, in such cases, vegetation increases due to several reasons (see fig 24):

-The narrowing of the river: there will be more available area on which plants can grow.

-The reduction of large flows: usually a large flow destroys or removes the plants across a river and it creates floodplain scour that washes away the roots of plants. Therefore, reducing large flows enables more plants to grow along a river.

- The increase of low flows: these flows usually make the soil more saturated and raise the water table which enhances the growth of new vegetation.

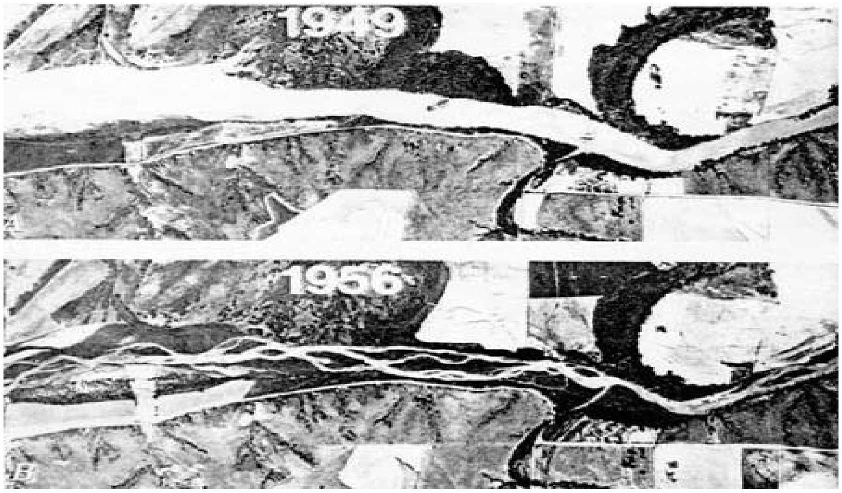

Figure 23.A comparison of the downstream vegetation of a river before (1949) and after the building of a dam (1956) ([20])

Dams Decommissioning

The decommissioning of a dam means the deactivation of some of its key functions such as ([15]; [22]; [25]; [26]):

-No more stored water: this is achieved by opening the gates forever

-Partial removal of the structure by breaching a part of the dam

-Complete removal.

In this section of the paper, the focus will be on the complete dams' removal.

After a long time of using the dam, its damages to the ecosystem and communities may outweigh its benefits. Dam removal may become a desired alternative for many deteriorating, unsafe or abandoned dams. In many cases, dam removal is more economical than its repair. All around the world, a new axiom "dams are not forever" is spreading.

Most of the removed dams were in the US, where more than 75000 dams over 6 feet high exist. In the past 75 years, several hundreds of dams have been removed all around the US.

Causes of Dams Removal

The main causes for removal of a dam are discussed in detail in this section. These include:

Better Understanding of Dams' Bad Impacts

Many studies have demonstrated that, with time, the cost (the harmed stream ecology) of the dam will outweigh its benefits (water storage, navigation purposes, power generation…). Therefore, after several years of using a dam, it will be more beneficial to remove it.

The Substitutes of Dams

A dam can be also removed if there is an alternative that is able to offer the same functions for which the dam has been built. For example, instead of building a hydropower plant for supplying power to a region, another source of power (i.e. nuclear) may satisfy this demand. Restoring wetlands, maintaining riparian buffers and relocating homes and business far from the floodplain can substitute the dams used mainly for flood protection. Renovating irrigation systems can reduce significantly the dependence on dams.

The Aging of Dams in the US

In the US, one reason for the increase in decommissioning activities is the poor condition of the nation's dams, where approximately 1800 of them are officially predicted unsafe. By 2020, 85% of the US government owned dams would be at least 50 years old that is the typical life span. Many of these dams have lost their main purposes and do not have any useful role. Some old dams require much more maintenance than before, therefore removing these old structures would be more economical than maintaining them.

The Relicensing Procedure

Another reason for dam removal is the license renewal given by the Federal Energy Regulatory Commission (FERC) for private hydropower dams (around 2400 all around the country) that enables them to operate for another 30 to 50 years. At least 500 such licenses will expire by year 2010. The main purpose of license renewal is to reevaluate the effects of dams, like the protection of endangered species and existing aquatic life and the quality of the environment. In many cases, the removal of unsafe or not useful dams will be the best solution for river management.

The Restructuring of Power Sources

The shift to more efficient electrical resources (i.e. the nuclear and solar power) will diminish significantly the use of hydropower sources. Moreover, as the licenses of some dams should be renewed by the Federal Power Act, new environmental constraints will be imposed where few or none had previously existed on these dams, making those dams uneconomical.

The Media's Concern in Dams Removal

Lately, actual dam removals cases were covered in almost all major newspapers. The focus of the media on this issue has offered to the public some knowledge about the economical and environmental benefits of removing dams that have lost their intended functions

Decommissioning Methods

These methods depend on project's characteristics like the size, type, dam location, river characteristics and intended objectives of decommissioning (fisheries restoration, land reclamation and recreation). Dam decommissioning is thus highly site-specific. The different decommissioning methods are:

1-Complete removal: which is often accomplished by first diverting momentarily the river, and destroying the dam

2-The creation of a Breach in the dam: that enables the river to flow around the dam's structures. Heavy machinery is usually used to breach earthen parts of dams located in relatively wide river corridors. It is advised to have a breach for partial dam removal, and this option is an inexpensive decommissioning option for larger structures, if it can be done.

3-The use of explosives: which is mostly used for destroying concrete dams.

4-The combination of heavy machinery and explosives: that is especially needed for large projects.

Removing Accumulated Sediments

Dams catch huge quantities of river sediment. It is estimated, on average, that each year, around 0.5% to 1.5% of the reservoir's storage capacity is filled with sediments. Therefore, if there is a dam removing plan, a special care concerning the accumulated sediments should be taken. Sediments removal represents the most costly and technically intensive job in removing large dams.

Specific sediment removal techniques are used according to the quantities and types of the available sediments, age of the dam and the effectiveness of periodical flushes. The sediment removal techniques must be accurately conducted because intensive removals may destroy some habitats in the river downstream side. For example, on Elwha River, a gradual sediment drawdown had been used in order to preserve the habitats of juvenile Salmon. A problem that may occur, when sediments are flushed, is the spreading of the accumulated contaminants or hazardous wastes into fisheries or water supplies leading to the disturbance of the aquatic life. For instance, after the removal of a 9-meter-high dam on New York's Hudson River in 1973, several tons of accumulated toxins spread downstream and killed a big part of the aquatic life in the river.

Another possible problem that can be faced when removing a dam is the spreading of hazardous waste present in sediments that are very dangerous to aquatic life and water quality. Thus a careful planning against spreading of the wastes should be taken care of when removing a dam.

Environmental Benefits of Decommissioning

The most important benefits when a dam is removed are:

1. Normalization of sediments and energy transports
2. The stabilization of the temperature in different part of the river
3. The enhancement of water clarity
4. Re-connection of important seasonal fish habitats
5. The enhancement of the concentrations of dissolved oxygen
6. The Help in establishing more biological diversity.

Examples of Removed Dams

Hundreds of dams have been removed all around the world, but most of them took place in the USA.

In what follows, some examples of removed dams in the US are discussed.

Woolen Mills Dam, Wisconsin

In 1919, a concrete gravity dam 18-foot high was built in the Milwaukee river for the Wisconsin Power and Electric (see fig 25) to produce electricity. By 1959, the dam was not profitable. The company abandoned the dam and the City of West Bend became the new owner. In 1988, the dam was removed because it became structurally unsafe and the cost of removing it is much lower than that for its rehabilitation.

Figure 24.Woolen Mills Dam before removal ([15])

The reservoir behind the dam became very shallow due to the accumulation of sediments. Water quality was bad, oxygen content was at its lowest level, and the water was turbid, aquatic life decreased and a large amount of pollution in the sediments was there due to the impoundment from a near landfill. There were no appreciable recreational uses near the dam.

The cost of rebuilding the dam was estimated to be 3.3 millions dollars, whereas the estimated removal cost will not exceed 5 % this amount. This is why the city proposed to remove the dam .The financing of this action is from the Federal government.

The dam ‘s removal cost was $86,000 (see fig 26). Both the state and city paid for seeding the previous reservoir area, design and engineering work, stabilization and vegetation.

Figure 25. Milwaukee River after dam removal ([15])

The river was restored to a rock-bottomed channel with meanders, riffles, pools and rapids. After the removal:

- The water quality has enhanced gradually and the water was well oxygenated.
- The city has restored 61 acres of land. This area became a park.
- The aquatic and fish life have improved significantly.
- The value of properties along the previous impoundment has increased significantly.
- The recreational opportunities near the site have greatly increased.

Lewiston Dam, Idaho

This dam was constructed in 1927 in the Clearwater River in Idaho. It was a semicircular earth dam, 45 feet high, with a1060 feet long concrete spillway and a powerhouse of 10 megawatts capacity .At the time of removal; Washington Water Power was the owner of this dam. This dam was removed in 1973.

The main problem of the Lewiston Dam was its bad impact on anadromous fish migration and their passage in the Clearwater River. A lot of different species also died due to the presence of this dam. In 1967, the Army Corps of Engineers decided to build the Lower Granite Dam that is located at the downstream side of the Lewiston Dam. The Lower Granite Dam was supposed to interfere with the impoundment of the Lewiston Dam. This project requires the removal of the Lewiston Dam. All the parties accepted the deal, and the structural removal cost was $ 633428 and began in December 1972. Moreover, Washington Water Power was paid $2.7 million in compensation for the loss.

The removal procedure was as follows:

1-Emptying the reservoir by opening the spillway gates

2-The bridge and spillway gates were removed

3-The concrete spillway was destroyed using dynamite (see fig 27).

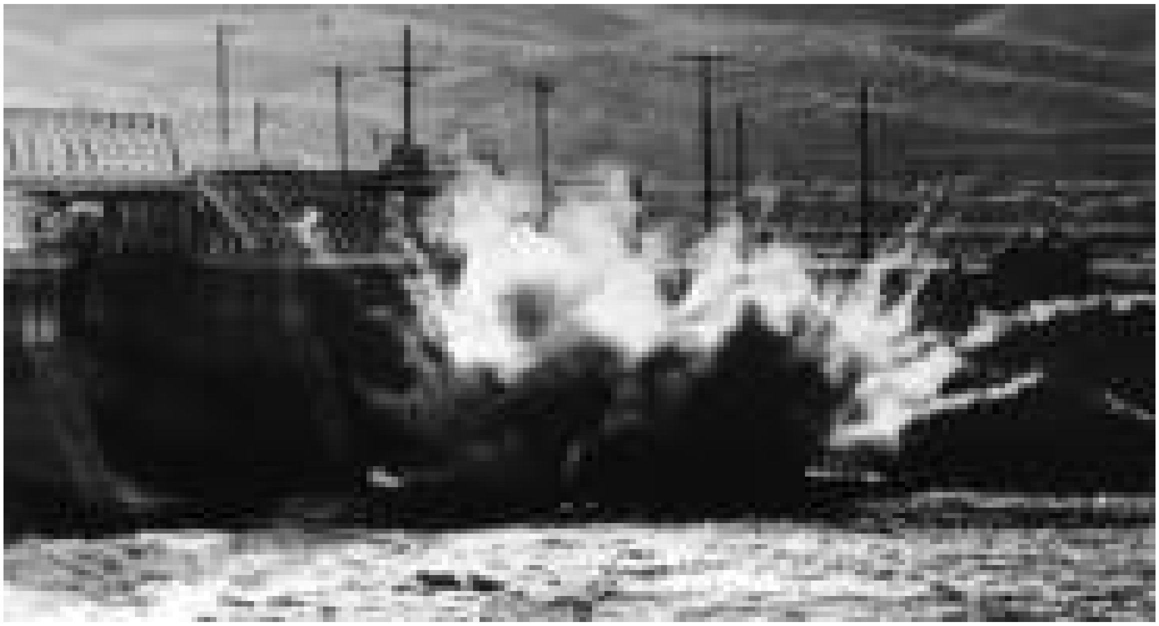

Figure 26. Dynamite is used for removing the Lewiston Dam ([15])

All the waste, without the steel, was deposited on the north side of the river and covered with soil and vegetation. The removal of sediments behind the dam was critical because of the fear that the huge quantities of sediments will flow downstream the removed dam and will reach the Lower Granite Dam. The used procedure was to remove the dam in a period of the year in which there is a low flow of water in the river that is unable to transport the sediments downstream. The removal was terminated in April 1973.

The benefits of this project are:

-No more maintenance for an unused structure

-No more obstruction for recreational boats

-The migration all along the river, for several types of fish, has been restored.

Chapter 2
Literature Review of Dam Breach

Introduction

This part of the report focuses mainly on the breach failure of embankment dams. Here, the focus is on how to predict the breach in an embankment dams and the effect of this failure on downstream.

Types of Models Available for Breach of Dams

There are two tasks that should be performed in the dam breach analysis, these tasks are ([16],p.3):

1-The prediction of the outflow hydrograph, which can be done by:

a) Predicting the breach characteristics such as the shape, depth and width of the breach

b) Routing the reservoir storage and inflow through the predicted breach form. Most computer models use different 1D routing methods.

2-Outflow hydrograph routing through the downstream valley: most models perform these routing procedures but each one uses different 1 D routing techniques. However, each model treats the breach simulation differently.

Breach Parameters

In this section, breach parameters are explained in detail. These parameters are ([16], p.7):

Breach Width

It refers, depending on each model, to the top, lower or average width of the breach.

Breach Depth or Height

This depth is commonly known as the distance from the dam crest to the breach invert.

Breach Side Slope Factor

Usually it is referred to it as Z where the side slope of the breach is given by Z h: 1v.

Breach Initiation Time

The breach initiation time begins with the first flow over or through a dam that will initiate warning, evacuation, or heightened awareness of the potential for dam failure. The breach initiation time ends at the start of the breach formation phase ([16], p.8).

Breach Formation Time

The time of failure as used in DAMBRK is the duration of time between the first breaching of the upstream face of the dam until the breach is fully formed. For overtopping failures the beginning of breach formation is after the downstream face of the dam has eroded away and the resulting crevasse has progressed back across the width of the dam crest to reach the upstream face ([16], p.8).

The engineer should always remember that there are 2 phases for dam failures where the first phase represents the breach initiation time whereas the second phase represents the warning time available for evacuating the downstream community ([16], p.8). The early researches had mainly focused on the breach formation time, and neglected the breach initiation time that they usually include in the definition of the beach formation time. This is the reason why breach initiation time has not been usually reported for many failed dams.

Available Approaches

Most methods are either based on 1- case study data from past failures or on 2- physical models that do not account for the actual true erosion mechanism and flow regimes that a dam breach may face ([16], p.5, [3], p.101). Here is a discussion about these two methods:

1-Case study methods are not very accurate because they are mostly based on small database of failed dams, especially of small dams. Case study data are not good in predicting the initiation time of a breach, breach's rate formation, and the total time required for failure. But under case studies, there are 3 methods:

1.1-parametric models: they first predict time of failure and ultimate breach geometry and compute breach outflows using hydraulics principles, second they simulate breach growth as being time dependent ([16], p.5).

1.2-predictor equations: these equations are most of the time empirical and they estimate peak discharge based on case study data ([21], p.90).

1.3.analysis by comparison: if the dam under study has characteristics similar to that of another actual failed dam with a well documented failure, the breach characteristics and hydrograph can be determined by comparison. In other words, this method neglects the process of breaching and is only based on comparison with a similar breached dam (comparative methods)([16], p.5).

2-Physically based models such as BREACH give more extensive information but suffer from their limited accuracy ([16], p.5). The current models are mostly based on geotechnical concepts and sediment transport relations that are not applicable or are not well tested on dam's breach ([3], p.151).

Other physical models like DAMBRK simulate the breach of the dam and the resulting reservoir outflow. The geometry and time of formation of the breach should be given to this program as an input, and the output will give the breach enlargement as function of time (e.g., linear increase of breach dimensions). The required input parameters should be found from either comparative methods or from prediction equations or other physical models.

Problems of These Approaches

The problems of these three approaches are ([16], p.8):

A-Comparative analysis: this analysis is only appropriate to small dams, because most case studies in this approach are based on small dams.

B-Predictor equations: the same restriction of the comparative analysis applies for the predictor equation method. Therefore the regression relations based on the available data have high uncertainty.

C-Physical models: the main flaws from which this method suffers are due to insufficient understanding of breach development; breach and high erosion dominating dam breach.

Breach characteristics

When a small variation in one of the breach parameters (width, depth, failure time and overtopping head) occurs, large changes in peak flows will take place especially for reservoirs with relatively small storage. In 1984,Singh and Snorrason used some models such as DAMBRK and HEC-1 on 8 hypothetical breached dams to assess which breach parameter affects mostly the peak outflow ([16], p.6).

Failure Time

They found that if failure time were reduced by half its initial value, the peak outflow for a PMF hydrograph would increase by 13 to 83 %. But for large reservoirs, the change in peak outflow was much smaller showing a variation of only 1 to 5 % ([16], p.6).

Breach Width

It seems that the changes in breach width is more effective for large dams because it produced larger changes (35-87%)in peak outflow and smaller changes (6-50%) for small reservoirs ([16], p.6).

Breach Depth

If breach depth is changed, little change in peak outflow has been identified, leading to the conclusion that the change in peak flow is not really dependent on the reservoir size ([16], p.6).

Other studies conducted by Petrascheck and Sydler (1984) also proved that a change in the breach width and breach formation time would significantly affect the outflow peak discharge, inundation levels, and flood arrival time. For locations not far from the dam, both breach width and breach formation time will have a great influence ([16], p.6).

Some critical results have been found by Wurbs(1987) which are([16], p.7):
In large reservoirs, the peak outflow takes place at the moment when the maximum depth and width of the breach are attained. Changes in reservoir head are relatively slight during the breach formation period.
In small reservoirs, a huge change in the level of the reservoir takes place during the formation of the breach; consequently the peak outflow occurs sometime before reaching the final breach. Here, the formation rate of breach is crucial.

Historical Overview

Before the late seventies, no dam failures were recorded in a detailed way. During the eighties, many researchers began gathering detailed breaches of dams in order to simulate models that are able to predict the effects and mechanisms of breach and estimate peak outflows. Among those are SCS in 1981, Singh and Snorrason in 1982, MacDonald and Langridge-Monopolis in 1984, Costa in 1985, Froehlich in 1987 and 1995 and Singh and Scarlatos in 1988. Later, other scientists used these models to develop guidance on breach outflow and parameters. The most known of those are FERC in 1987; Reclamation in 1988 and Von Thun and Gillette in 1990.All these methods will be discussed later ([16], p.10; [3], p.151; [21], p.90).

Each of these methods is based on about 15 to 60 dams, mainly for small dams because of lack of failure data for high dams (more than 75 ft).

Empirical Models for Predicting Breach Parameters

Using case study data, many researchers developed formulas that enabled them to predict breach parameters like time of breach formation and breach geometry. In the following, a discussion concerning each method is given.

Johnson and Illes (1976)

They were the first to predict failure shapes for earth, gravity, and arch concrete dams. For earth dams, their proposition was that the breach shape begins as a triangle and ends as a trapezoid ([27]; [16], p.13). They also realized that failure width (general) B is given by:

$0.5h_d \quad B \quad 3h_d$ for earthfill dams

Most other studies assume that the breach shape of earthen dam is trapezoidal.

Singh and Snorrason (1982,84)

Their study was conducted on 20 case studies and they came up with the following ([28]; [3], p.101; [16], p.14). The breach width is constrained by:

$2h_d \quad B \quad 5h_d$

Where B=breach width (general) and h_d=dam height with

$0.15\ \text{m} \quad d_{ovtopp} \quad 0.61\ \text{m}$

Where $d_{overtopp}$=the maximum overtopping height above the crest of the dam before failure

0.25 hr t_f 1.0 hr

Where t_f=failure time

MacDonald and Langridge-Monopolis (1984)

Based on 42 case studies, they suggested that most of the breach side slope are approximately 1h: 2v and that the breach shape could be trapezoidal or triangular and this depends on whether the breach has reached the bottom of the dam or not ([29]; [16], p.13-14). They also estimated the quantity of eroded embankment materials V_{er} (m^3) for earth dams based on time of failure t_f.

$V_{er} = 0.0261(V_{out}*h_w)^{0.769}$

Where V_{out}=volume of water discharged through breach

And h_w=hydraulic depth of water at dam at failure above breach bottom (m)

$t_f = 0.0179(V_{er})^{0.364}$

On the other hand, for nonearthfill dams they came up only with estimation for volume of eroded embankment material V_{er}.

$V_{er} = 0.00348(V_{out}*h_w)^{0.852}$

They could not predict the failure time for nonearthfill dams because sometimes the failure of such dams may be caused by structural problems instead of erosion ([16], p.14). They also found it crucial that the estimation of breach parameters and outflows should be conducted using several iterations.

FERC (1987)

Ferc proposed ([30]; [16], p.13):

Usually $2h_d<B<4h_d$

But B can range $h_d<B<5h_d$

Where B is the breach width

$0.25 < Z < 1$ (engineered, compacted dams)

$1 < Z < 2$ (non-engineered, slag or refuse dams]

Where Z =horizontal side slope factor (Z horizontal: 1vertical) for breach opening

$0.1<t_f<1$ hours (engineered, compacted earth dam)

$0.1<t_f<0.5$ hours (non-engineered, poorly compacted)

Froehlich (1987, 1995)

In his research, he used 43 case studies ([31]). He used nondimensional analysis in order to create equations that estimate the average breach width, side slope and the time of failure ([16], p.14). These equations are:

$B^*_{avg}=0.47K_0(S^*)^{0.25}$

Where B^*_{avg} is the nondimensional average width= $(B_{top}+B_{bottom})/(2h_b)$

And h_b=height of breach and S^*=dimensionless storage=(S/h_b^3)

K_o = constant=1.4 if there is overtopping, else 1

$Z=0.075K_c(h^*_w)^{1.57}(W^*_{avg})^{0.73}$

Where Z is the side slope factor, h^*_w=dimensionless height of water above breach bottom (h_w/h_b)

W^*_{avg}=average dimensionless embankment width= $(W_{crest}+W_{bottom})/(2/h_b)$

K_c = constant=0.6 if there is a core or 1.0 if no core is present

$t_f^* = 0.79(S^*)^{0.47}$

Where t_f^*=dimensionless breach formation time=$t_f/(gh_b)^{0.5}$

These equations were based on very specific dam characteristics like the presence of core, height of water above breach bottom, the extent of overtopping and so on ([16], p.14). He also realized that overtopping causes the most breach extension and erode at a higher rate than any other failure mode.

In 1995, 8 years after his first study, he published new and revised equations based now on 63 case studies ([16], p.14). This time, the new equations are not nondimensional. These equations have better estimated coefficients. These new equations are:

$B_{avg}\,(m)=0.1803K_0V_w^{0.32}\,h_b^{0.19}$

K_0=constant=1.4 if there is overtopping and 1 if else.

$t_f=0.000254V_w^{0.53}\,h_b^{(-0.9)}$

Z=1.4 if there is overtopping, if not Z=0.9

Reclamation (1988)

They develop these equations for earthen dams where ([16], p.15):

$B = 3h_w$

t_f(hours) = 0.011B and B is in meters

Where h_w=height measured from the initial reservoir water level to the breach bottom elevation which is assumed to be the streambed elevation at the toe of the dam. Reclamation uses these formulas in the SMPDBK model. The suggested formulas are conservative, and thus they represent a factor of safety for the hazard classification procedure.

Singh and Scarlatos (1988)

Their study is based on 52 case studies ([3], p.101). They found that the top width is 106% to 174% larger than the bottom width with an average of 129% and an acceptable standard deviation of 18 %. Whereas, they found that the ratio of the top breach width to dam height was widely distributed. The breach side slopes were inclined 40^o to 80^o with the horizontal. Moreover, most failure times were less than 3 hours.

Von Thun and Gillette (1990)

They have used the data of Froehlich (1987) and MacDonald and Langridge-Monopolis (1984) in order to develop some breach parameters ([32]; [16], p.15). In their work, they assumed that side slopes of breach are 1H: 1V except for dams that have cohesive shells or very wide cohesive cores, where slopes of 1:2 or 1:3 (H: V) are more acceptable.

The relation proposed by Von Thun and Gillette is for the average breach width, and it is given by ([16], p.16):

B_{avg} (m) $=2.5h_w+C_b$

Where h_w = the depth of water at the dam at the time of failure, and C_b is dependant on the reservoir storage (see table 8):

Table 8.Values of C_b according to the reservoir size ([16], p.15)

Size of reservoir (m^3)	**C_b (m)**
$<1,23*10^6$	6.1
$1.23*10^6$-$6.17*10^6$	18.3
$6.17*10^6$-$1.23*10^7$	42.7
$>1.23*10^7$	54.9

They plotted the volume of the eroded embankment versus water outflow volume and water depth above the breach invert, with upper bounds of reasonable breach geometry estimates.

These methods are dependent on the amount of erosion that occurs ([16], p.15):

t_f(hr) = 0.020h_w + 0.25 (erosion resistant)

t_f(hr) = 0.015h_w (easily erodible)

Where t_f should be in hours and h_w in meters.

Moreover, they have suggested other equations that estimate the time of failure using the average lateral erosion rate (the ratio of the final breach width to breach formation time) and depth of water above the breach invert. They conclude that there is a better estimation using these equations than the first ones that they developed. These new equations are ([16], p.16):

$$t_f = \frac{B_{avg}}{4h_W} \quad \text{(Erosion resistant)}$$

$$t_f = \frac{B_{avg}}{(4h_w + 61)} \quad \text{(Highly erodible)}$$

Empirical Models for Predicting Peak Outflows

Some other researchers have conducted studies to determine the peak outflow as a function of the breach parameters (dam height, reservoir storage volume). A discussion of each of these methods is given below.

Kirkpatrick (1977)

Using data from 13 failed embankment dams and 6 other hypothetical failures; he related the peak flow versus the depth of water behind the dam at failure. This equation is written as ([33]; [16], p.16):

$Q_{P=}f\ (h_w)$

But the flaw of this method is that among the case study failures he used is the St. Francis Dam in California, which was a concrete gravity dam ([16], p.16).

SCS (1981)

The Soil Conservation Service used the 13 cases studied by Kirkpatrick in order to develop another method, for earth dam, that relates the peak dam failure outflow to the depth of water at the dam at the time of failure ([34]; [35]; [16], p.16). The equation is given by ([21], p.93):

When H_w>31.4 m

$$Q_p=16.6H_w^{1.85} \quad (1)$$

Where H_w=the height of water directly at the reservoir before breach measured from the bottom of the final breach.

And Q_p=peak outflow through the breach (m^3/s).

When H_w<31.4m

$$Q_p=0.000421\ (V_wH_w/WH)^{1.35} \quad (2)$$

Where V_w=reservoir water volume at the time of failure (m^3)

W=average width from the bottom of the final breach to the top of the embankment.

H=distance from the bottom of the final breach to the top of the embankment

But the flow calculated in (2) should not exceed the value given by (1) and not less than

$Q_p=1.77H_w^{2.5}$

From the plot of the results of this method with that of the observed flows, It appears that there is a good matching between calculated and measured peak flows except at the low peak flows ([16], p.16).

The problem of this method is that it does not provide a way for determining a peak outflow that provides a factor of safety when evaluating downstream flooding.

Reclamation (1982)

Used the work done by SCS and proposed a similar envelope equation for peak breach outflow using case study data from 21 failed dams ([16], p.16).

Singh and Snorrason (1982 and 1984)

They established methods relating the peak outflow to the dam height and stored water in the reservoir. These relations were found using the results of eight simulated dam failures analyzed using DAMBRK and HEC-1. Therefore these equations were developed using simulation ([28]; [3], p.151).

MacDonald and Langridge-Monopolis (1984)

They did a best-fit analysis and boundary curves on 42 failed earth dams in order to determine peak outflow ([29]; [16], p.17). The developed equation is ([21], p.94):

$Q_p=1.175(V_wH_w)^{0.41}$

Where V_w=the total quantity of stored water at failure

And H_w=the hydraulic height of water directly at the reservoir before breach, measured from the bottom of the final breach.

This formula will exaggerate the peak flow for embankment dams.

They have also tried to establish similar relations on non-earthen dams, but this attempt did not succeed because the standard deviation of the data was large ([16], p.17).

Costa (1985)

This method is mainly based on regression analysis. It Applies for both embankment and concrete dams, because the 31 cases studied to develop this method were a mix of both embankment and concrete dams ([36]; [16], p.17; [21], p.94).

The peak outflow is given by ([21]):

$Q_p=0.763(V_wH_w)^{0.42}$

But this formula overestimates the peak outflow for the embankment dams because a concrete dam will have bigger breach than a similar embankment dam having the same volume ([21], p.94).

Froehlich (1995)

The equation is found by running a multiple linear regression on 22 dams where discharge data were available ([16], p.17&[21], p.94). This equation is given by ([21], p.94):

$Q_p=0.607V_w^{0.295}H_w^{1.24}$

This equation gives a good agreement with the measured computed peak flows over the entire range.

Physical Models

In the last 3 decades, many mathematical models for simulating dam breach have been established ([3], p.151).

In what follows, a discussion about each model is presented.

Cristofano (1965)

The model is based on the following assumptions ([37]; [3], p.151-153; [16], p.17):

-The breach is a trapezoid having a fixed bottom width

-The breach side slopes depend on the angle of repose of the material

-The breach channel bottom slope is equal to the internal angle of friction of the material.

This model relates the breach peak outflow rate to the rate of erosion of the breach channel using an equation that account for the soil shear strength and the force of the flowing water.

Then main flaw of this method is that the model performance is heavily based on an empirical coefficient.

Harris and Wagner HW model (1967)

The main assumptions of this method are ([38]; [3], p.153-157;[16], p.18):

- Whenever overtopping takes place, the erosion occurs and continues till reaching the dam's bottom
- The use of the Schoklitsch transport sediment method for erosion
- The shape of the breach is parabolic.

Brown and Rogers or BRDAM (1977, 1981)

They created BRDAM breach model. This model was mainly based on Harris and Wagner's work that is applicable to breaches caused by overtopping and piping ([3], p.157).

Ponce and Tsivoglou (1981)

The "foundations" of this model are ([39]; [3], p.161-164):

- The use of the Peter-Meyer Müller sediment transport equations
- The use of the one-dimensional differential equations of unsteady state and sediment conservation.
- The use of a Manning's *n* for depicting flow resistance
- The breach width is related to the flow through the breach.
- The model accounts for reservoir storage depletion if the upstream boundary conditions were set.

The main flaw in this method is that the differential equations of unsteady state and sediment transport are solved in a very complex way and prone to problems of numerical instability.

DAMBRK (Fread, 1977)

DAMBRK simulates the breach in a way that the breach is initiated at the top of the dam and expands uniformly downward and outward to reach ultimate breach dimensions for a time specified by the user ([40]; [16], p.18). This model simulates 1-the shape of the breach 2-brech outflow 3-flood routing in the downstream valley ([3], p.169).

BREACH (Fread 1988)

The BREACH model is used to simulate more accurately the breaches caused by piping and overtopping. The model is based on ([41]; [3], p.176-186):

- The new version of Meyer-Peter and Müller sediment transport equation that is performed by Smart (1984) for steep channels.
- The orifice or weir equations are used in order to predict the peak outflow.
- The assumption that the flow is in a quasi-steady state
- The Manning's n is determined from the Stickler's equation
- The model accounts for tailwater depth.
- The shape of the breach may depend on the slope stability of the breach side slopes,
- The upper portion of the dam is analyzed by shear and sliding
- It simulates the breach assuming having an overtopping or piping.

This model is the most widely known and used.

The FLOW SIM 1 and FLOW SIM 2

The main aim of this program is the flood routing downstream the dam, but it can also conduct breach formation. The main assumptions used in this model are ([16], p.19):

- The possible breach's morphologies are: triangle, rectangle and trapezoid.
- The use of the Schoklitsch sediment transport method.

.

Breach Erosion of Embankment Dams or BEED (Singh and Scarlatos (1985))

This model simulates breach expansion, flood routing, and sediment routing and routes the flows of water and sediments through the downstream waterway ([42]; [3], p.186). The model uses the Einstein-Brown and Bagnold sediment transport method. The use of these equations requires a lot of assumptions that are far beyond the original ranges allowed for them ([3], p.189).

Results

To conclude, the majority of the physical methods are based on different erosion and sediment transport formulas that in turn assume different flow conditions. Moreover most those models use an averaged Manning's n. The adequacy of those models does not perfectly agree with the observed and tested dams' failures.

Comparison of the Empirical Methods

The several case studies on which different models or methods have been based vary enormously in the available data and characteristics (width, height, size, type of dam, mode of failure and so on).

Comparison of Predicted Breach Parameters

The depth, width and side slopes angle of the breach are the most documented breach parameters in the case studies. This is not true for the time of failure, which is not very much recorded ([16], p.32).

In brief, the depth of breach can be well estimated because, in most cases, this parameter cannot be very different from the dam's height. Whereas, the inaccuracy is mostly found in estimating breach width and time of failure.

Breach Width

For most of the 84 analyzed case studies in figure 28, it seems that, most of the time; the breach width is between 2 to 5 times the dam's heights ([16], p.33).

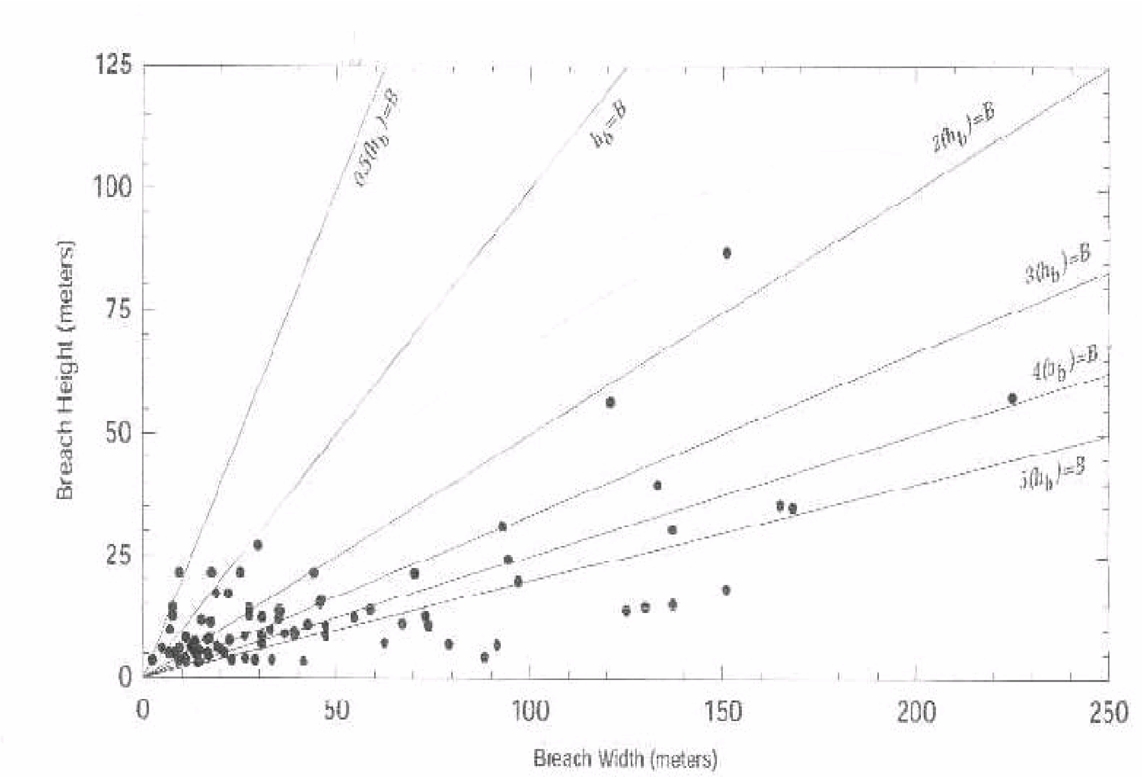

Figure 27. Observed height vs. width of breach ([16], p.33)

Figure 29 represents the predicted and observed breach width for the relations proposed by Reclamation (1988), Von Thun and Gillette (1990), and Froehlich (1995) applied on about 75 dams ([16], p.33).

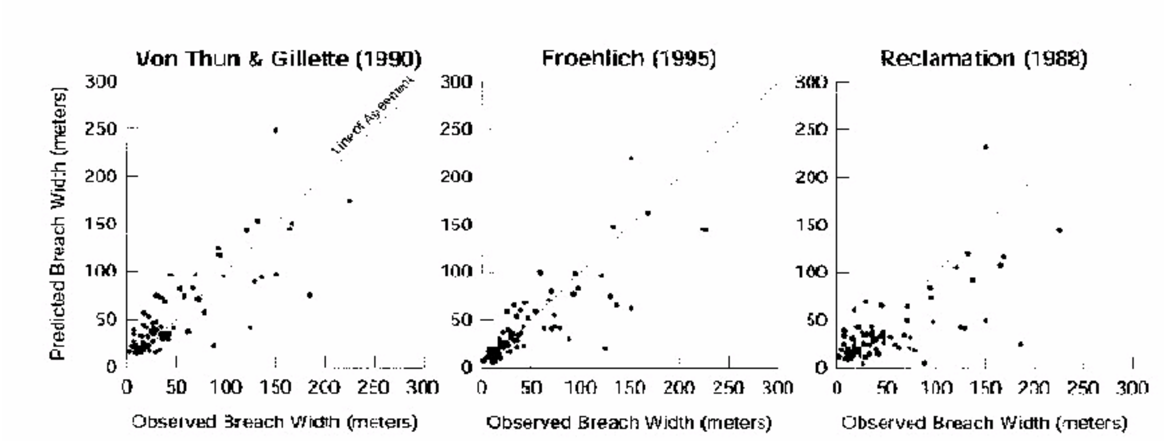

Figure 28. Breach width, predicted vs. observed ([16], p.34)

From observation, it is clear that the Froehlich's relation gives the best fit especially for breach widths less than 50 meters.

Time of Failure

MacDonald and Langridge-Monopolis (1984) predicted the lower envelope of the time of failure as a function of the volume of material eroded from the embankment during the breach. Figure 30 shows the observed volume of material eroded vs. the predicted ones for 60 breached dams ([16], p.35).

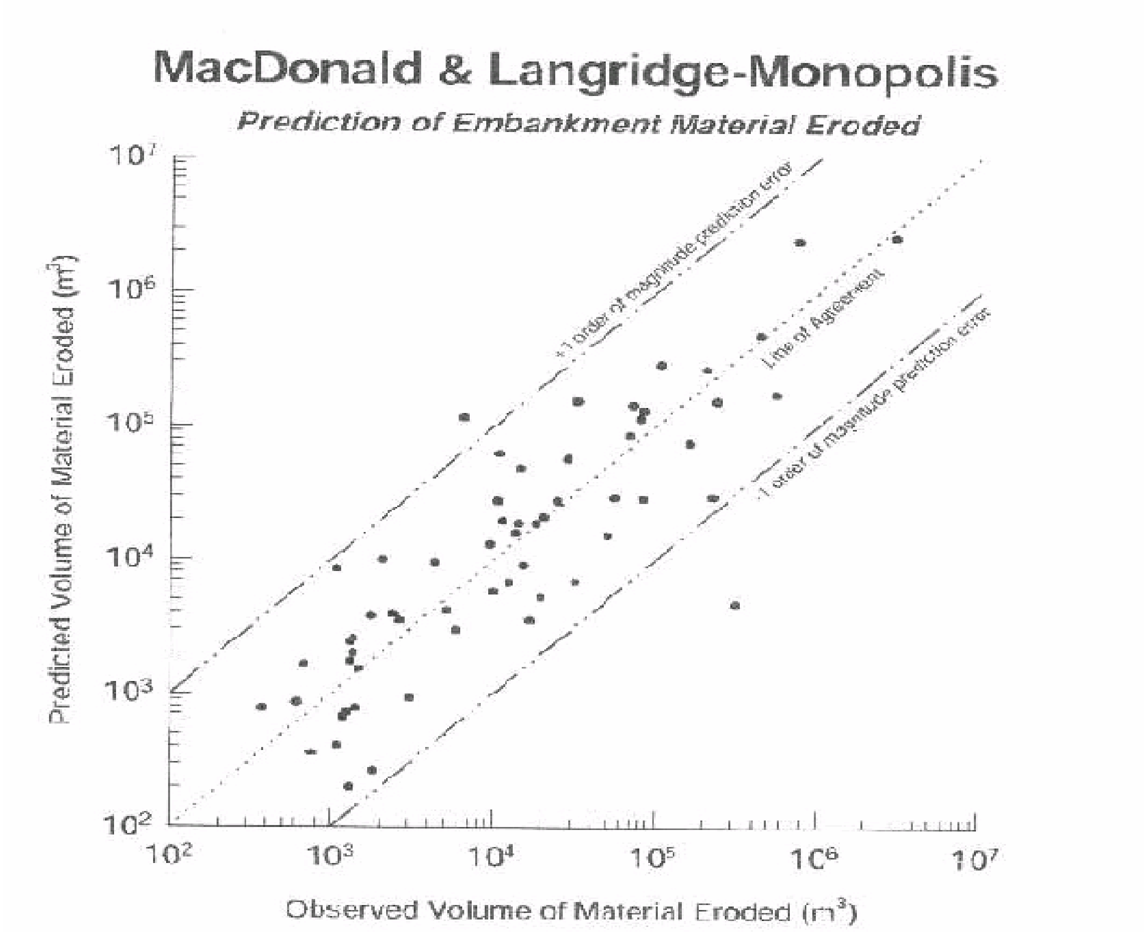

Figure 29.Predicted vs. observed volume of eroded materials using MacDonald and Langridge-Monopolis method ([16], p.35)

The flaws of this method are:

-The use of the breach formation factor that directly affects the estimated erosion volume

-A lot of assumptions are used in order to calculate this factor.

Using the Von Thun and Gillette, MacDonald and Langridge-Monopolis, Froehlich and Reclamation relations, a plot of the time of failure is given in figure 31.

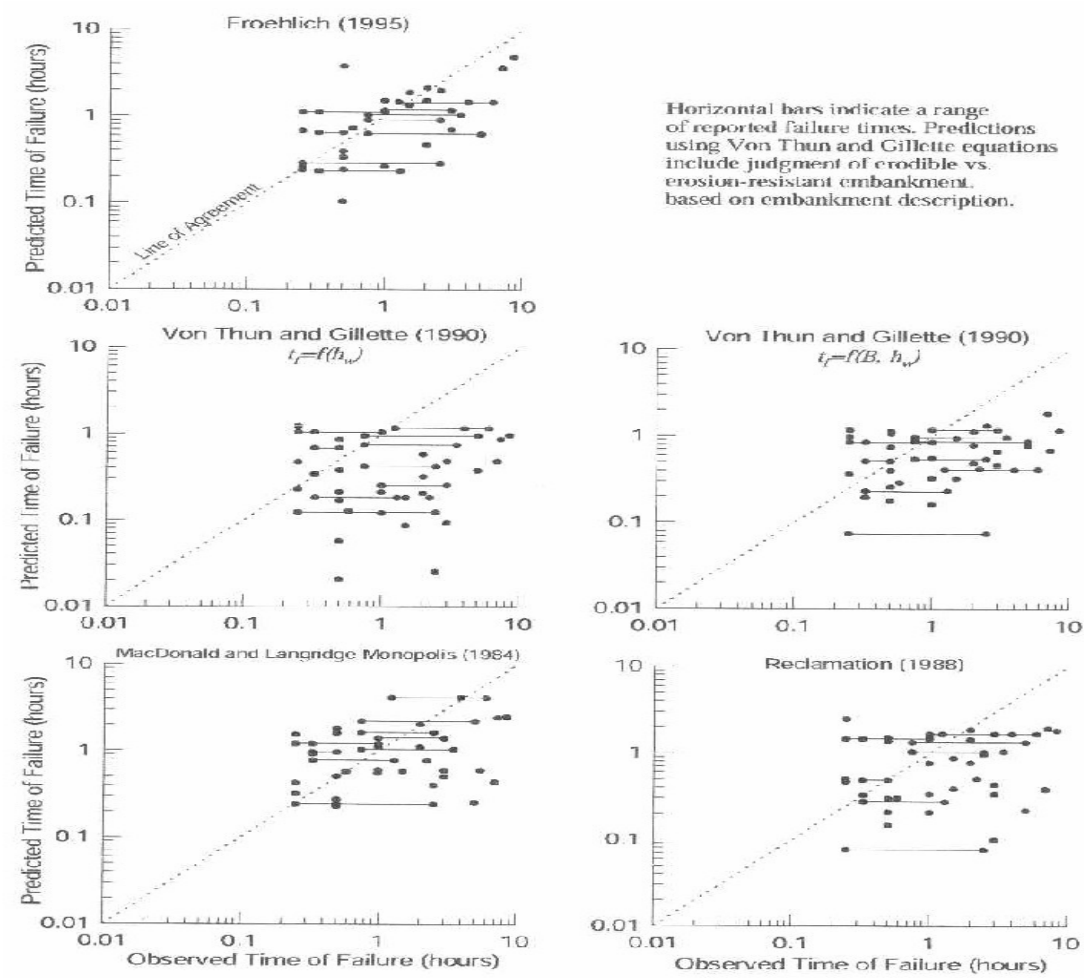

Figure 30.Predicted vs. observed time of failures using different methods ([16], p.36)

A conclusion that can be drawn is that no method offers a good prediction for the time of failure.

Comparison of Predicted Peak Flows

Figures 32 through 34 compare the peak outflow calculated from different relations with other parameters. It should be noted that the development of each of these equations has used different number of databases ([16], p.37).

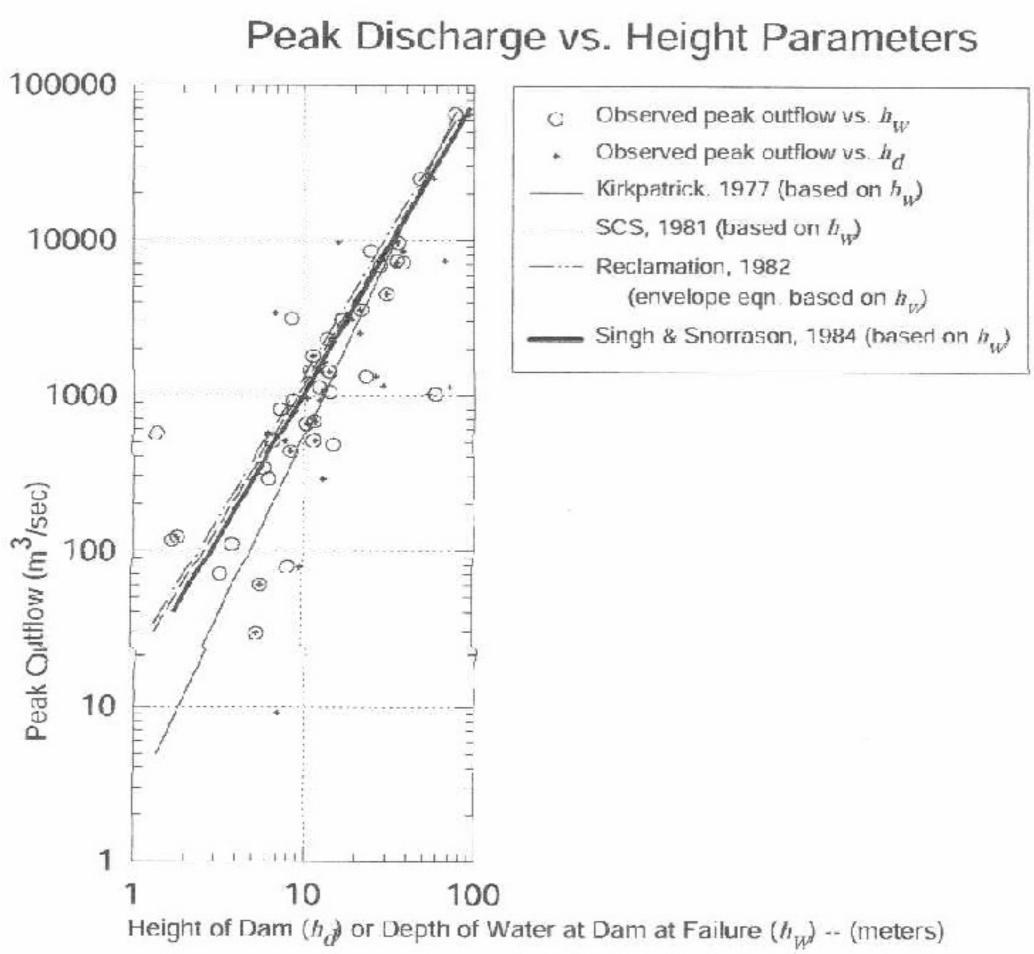

Figure 31.Peak outflow vs. different heights ([16], p.37)

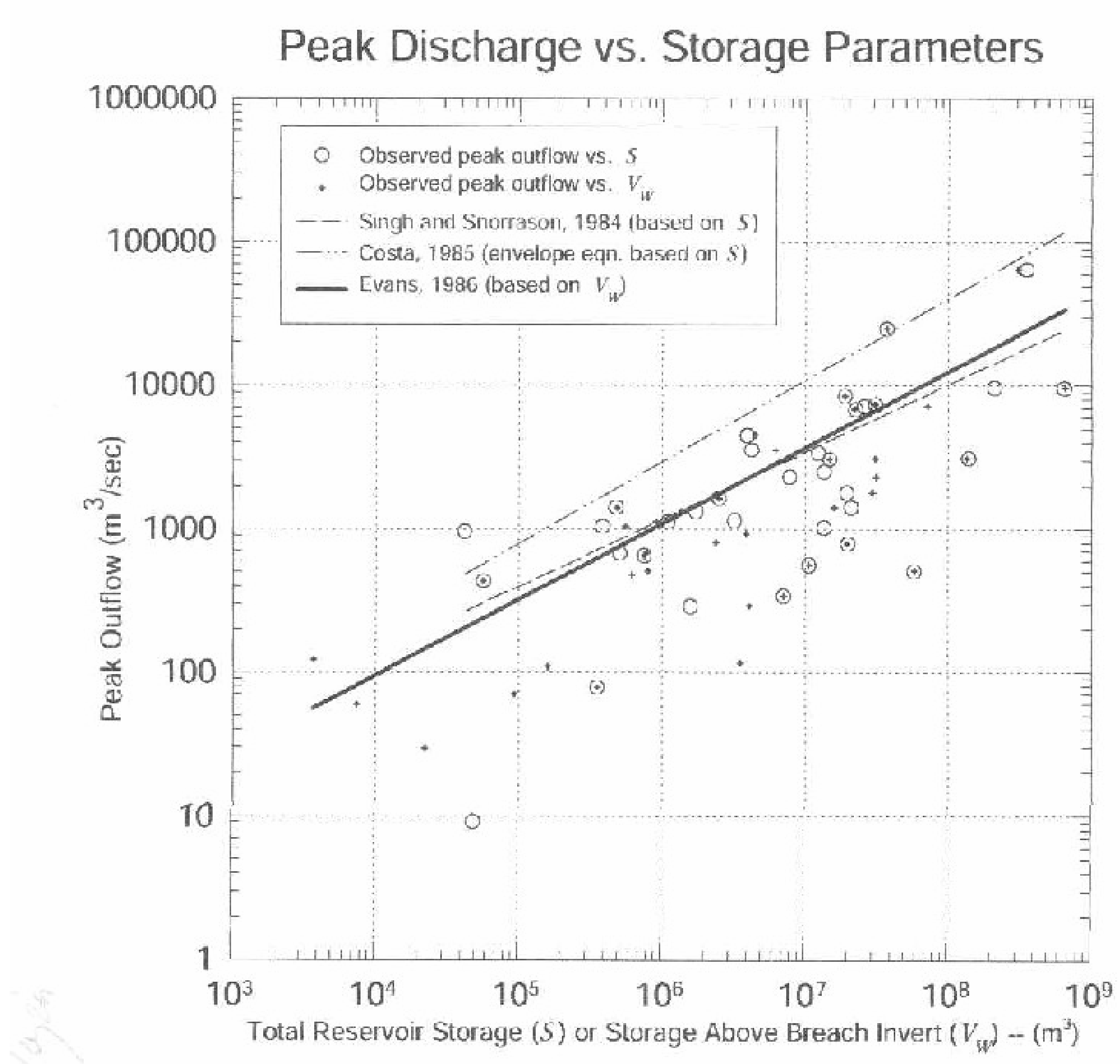

Figure 32.Peak outflow vs. different storages ([16], p.38)

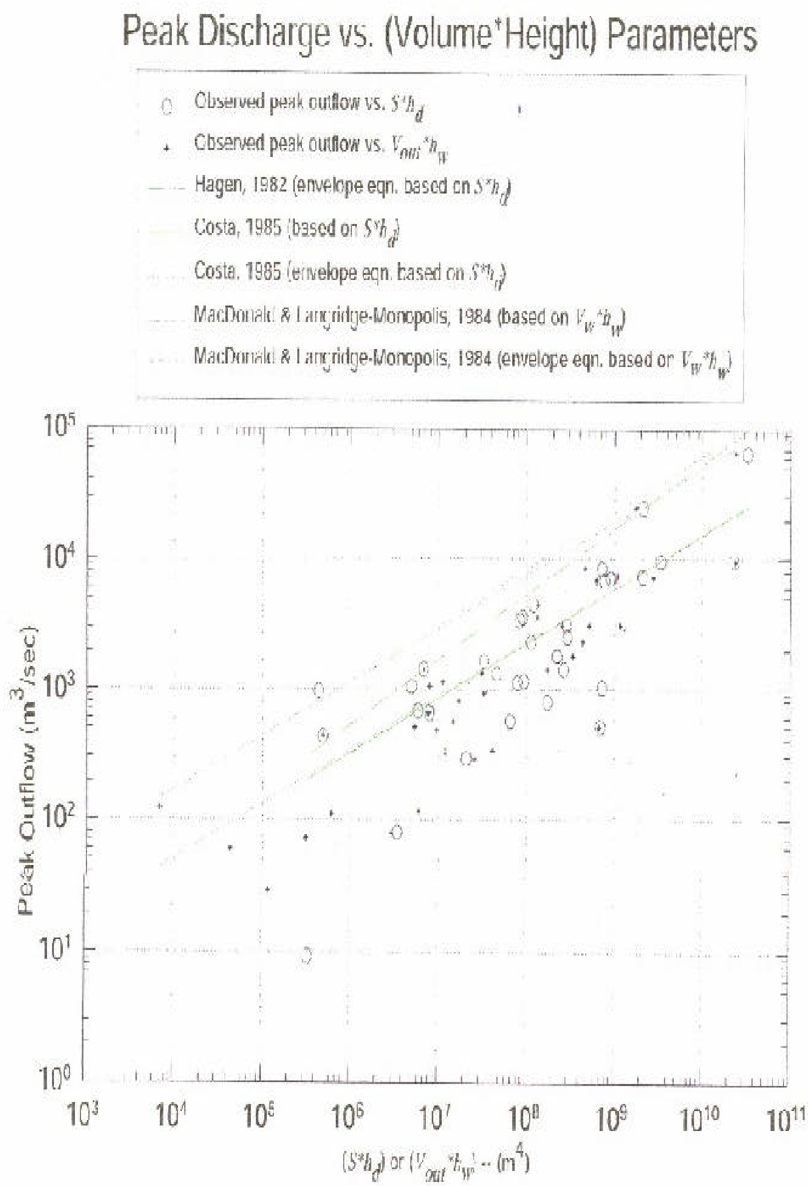

Figure 33.Peak outflow vs. different (height*volume) ([16], p.39)

It seems that most of these formulas do not offer a good estimation for the peak outflow at failure.

The peak outflow relation developed by Froehlich is applied on 32 dams, among which 22 have been used to develop this relation (see fig 35). Most of the data fit well this estimation ([21], p.94-95).

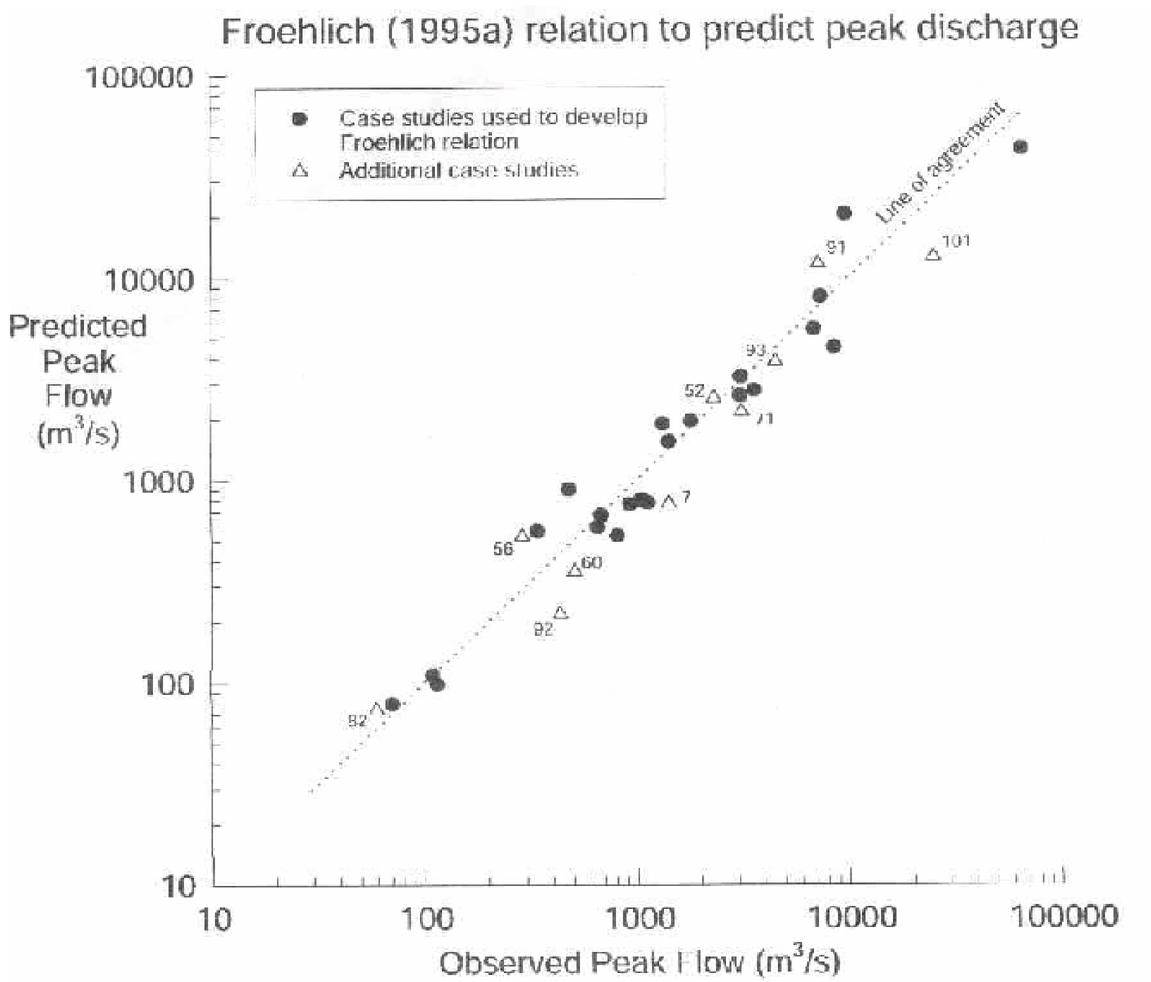

Figure 34.Predicted vs. observed peak outflow using Froehlich's equation ([16], p.37)

Chapter 3
Application of Empirical Breach Formulas

Introduction

This chapter focuses on the application of some of the methods discussed in chapter 2. Specifically, the methods are applied to analyze the failure of the Timberlake dam. This dam is an embankment dam that failed due to overtopping on June 22, 1995 at about 10:20 PM. After the failure of the dam, the geological survey assessed the peak flow at 50000cfs (1416 m^3/sec) .The purpose of this chapter is to calculate the peak flows for various failure configurations and durations and compare the obtained values with the USGS estimate.

The characteristics of this failed dam are given in table 9.

Table 9.Some breach parameters

Location	Campbell county 1 mi upstream of New London, VA
Failure time	10:20 PM on June 22 1995
Rainfall	8.5 -10 in 24 hours
Dam type	Earthen dam
Year of construction	1926
Inflow	Buffalo Creek feeds it
Height of dam	33 ft or 10.058m
Crest length of the dam	600 ft or 182.88 m
Storage volume (without freeboard)	1449 ac-ft
Total surface area (no freeboard)	93 ac
Drainage area	4.36 sq.mi
Normal capacity (with freeboard)	990 ac.ft
Breach width B	90-180 ft

Basic Variables

First, we determine certain variables that are of importance.

Table 10 shows these parameters.

Table 10. Basic variables definition

Variable	Definition
$d_{overtop}$	Height of overtopped water above the dam
h_d or h_b	Distance of from the breach to the bottom of the final breach to the top of the embankment
H_w or h_w	Hydraulic height of water directly at the reservoir before breach, measured from the bottom of the final breach
H	Height of the dam
V_w	Total quantity of stored water at failure
V_a	Volume of water that was above the dam's crest at the moment of failure
W, B	Average width of the embankment from the bottom of the final breach to the top of the embankment

$d_{overtop}$ =height of overtopped water above the dam ,it is assumed that it varied between 1 to 5 foot or $\boxed{0.36m \quad d_{ovtopp} \quad 1.58\ m}$

Moreover, h_w or H_w=the hydraulic height of water directly at the reservoir before breach, measured from the bottom of the final breach=h_{dam}+$d_{overtopp}$ so:

h_w=H_w=34-38 ft=10.36-11.5824 m ,or $\boxed{10.36\ m<h_w \text{ or } H_w< 11.5824\ m}$

We will assume that the shape of the storage reservoir is rectangular, so any 1 ft increase in water height behind the reservoir will be accompanied by an increase in storage equal to the total surface area. In other words, when overtopping occurs, for each additional 1 ft in height, the storage will increase by 93ac-ft.

The total quantity of stored water at failure V_w is equal to the maximum storage of dam if there is no freeboard (1449 ac-ft) adding to it the volume of water that was above the dam's crest at the moment of failure.

In this case, V_w=1449+V_a

Where V_a=volume of water above the crest of the dam=(approximated total surface area)*(the overtopping height $d_{overtop}$)= 93*1 to 93*5 ac-ft

So 1542 ac-ft<V_w<1914 ac-ft

$\Rightarrow$ $\boxed{1902028.99\ m^3 < V_w < 2360884.23\ m^3}$

Now define B or W as being the average width of the embankment from the bottom of the final breach to the top of the embankment.

But it is reported that the breach width was between 90 and 180 ft.

So if we assume that the bottom width was 90 ft and the top one was 180 ft, we can say that the average is 135 ft, but this is not for sure true because the shape of the breach was not 100% trapezoidal, so for determining the average width it is better to have a range. Assume that, on average, this range varies between 100 ft and 160 ft or 30.48m and 48.768 m (in order to account for a variety of shapes).

$\Rightarrow$ $\boxed{30.48\ m < W \text{ or } B < 48.768\ m}$

Now we define h_d or h_b as the distance from the bottom of the final breach to the top of the embankment .In this case the height of the failure is equal to the height of the dam.

$\Rightarrow$ $\boxed{h_d = h_b = 10.058m.}$

In summary, the values of the basic variables are given in table 11.

Table 11.Values of the basic variables

Variable	Value
$d_{overtop}$	1-5 ft or 036-1.58 m
h_d or h_b	10.058 m
H_w or h_w	34-38 ft or 10.36-11.5824 m
H	10.058 m
V_w	1902028.99-2360884.23 m^3
V_a	93 -93*5 ac-ft
W, B	30.48-48.768 m

Now, let us apply the main empirical relations that we have on hand for this case.

Prediction of Peak Outflows

Several methods for predicting peak outflows have been discussed in chapter 2.

Using the range of values for H_w or (h_w) V_w and W or (B), we obtain the lower and upper bounds for the dam breach peak flow Q_p.

The SCS Method

The SCS equations are given by:

$10.36m<H_w<11.5824$

$Q_P=16.6H_W{}^{1.85}$ for $H_w>31.4$ m (1)

And $Q_P = 0.000421\left(\frac{V_W H_W}{WH}\right)^{1.35}$ for $H_w<31.4$m (2)

Using the range $10.36<H_w<11.5824$ m for the Timberlake dam, we have:

$Q_P=16.6(10.36)^{1.85}=1254.65m^3/s$ (Lower bound)

$Q_P=16.6(11.58)^{1.85}=1541.59m^3/s$ (Upper bound)

Now, for $H_w<31.4$ m

$$Q_P = 0.000421\left(\frac{1902028.99*10.36}{48.768*10.058}\right)^{1.35} = 691.17m^3/s \text{ (Lower bound)}$$

$$Q_P = 0.000421\left(\frac{2360884.23*11.5824}{30.48*10.058}\right)^{1.35} = 1894m^3/s \text{ (Upper bound)}$$

But the flow calculated in (2) should not exceed the value given by (1) and not less than

$Q_p = 1.77H_W{}^{2.5}$

$Q_p = 1.77*(10.36)^{2.5} = 611.466m^3/s$ (Lower bound)

$Q_p = 1.77*(11.5824)^{2.5} = 808.108m^3/s$ (Upper bound)

$\Rightarrow$ $\boxed{691.17 \text{ m}^3/\text{s} < Q_P < 1541.59 \text{ m}^3/\text{s}}$

MacDonald and Langridge-Monopolis Method

The peak outflow is given by:

$$Q_P = 1.175(V_W H_W)^{0.41}$$

In this case:

$$Q_P = 1.175(1902028.99 * 10.36)^{0.41} = 1150.33 m^3/s \text{ (Lower bound)}$$

$$Q_P = 1.175(2360884.23 * 11.5824)^{0.41} = 1315.69 m^3/s \text{ (Upper bound)}$$

$\Rightarrow$ $\boxed{1150.33 \text{ m}^3/\text{s} < Q_P < 1315.69 \text{ m}^3/\text{s}}$

Costa Method

The peak outflow is given by:

$$Q_P = 0.763(V_W H_W)^{0.42}$$

$$Q_P = 0.763(1902028.99 * 10.36)^{0.42} = 883.58 m^3/s \text{ (Lower bound)}$$

$$Q_P = 0.763(2360884.23 * 11.5824)^{0.42} = 1013.93 m^3/s \text{ (Upper bound)}$$

$\Rightarrow$ $\boxed{883.58 \text{ m}^3/\text{s} < Q_P < 1013.93 \text{ m}^3/\text{s}}$

Froehlich (1995) Method

This flow equation is given by:

$$Q_P = 0.607 V_W^{0.295} H_W^{1.24}$$

$$Q_P = 0.607 * 1902028.99^{0.295} * 10.36^{1.24} = 784.51 m^3/s \text{ (Lower bound)}$$

$$Q_P = 0.607 * 2360884.23^{0.295} * 11.5824^{1.24} = 960.18 m^3/s \text{ (Upper bound)}$$

$\Rightarrow$ $\boxed{784.51\ m^3/s < Q_P < 960.18\ m^3/s}$

This equation gives a good agreement with the measured peak flows over the entire range.

Estimation of Breach Width B or W

We will estimate the breach width B or W using several methods.

Johnson and Illes Method

The range of B is given by:

$0.5h_d \quad B \quad 3h_d$ for earthfill dams, using the value of h_d=10.058 m (see table 3), we have:

$\boxed{5.029\ m \quad B \quad 30.174\ m}$

Singh and Snorrason Method

They found that the breach width range is:

$2h_d \quad B \quad 5h_d$

For the same h_d value of 10.058 m (see table 3), we have:

$\boxed{20.116\ m \quad B \quad 50.29\ m}$

FERC Method

Proposed that:

Usually $2h_d < B < 4h_d$

Therefore: 20.116 m$<B<$40.232 m

But B can range $h_d<B<5h_d$

$\Rightarrow \boxed{10.058\ m<B<50.29\ m}$

Froelich (1987,1995) Methods

In his paper of 1987,Froelich suggested the following:

$B^*_{avg}=0.47K_0(S^*)^{0.25}$

Where B^*_{avg} = the nondimensional average width and S*=dimensionless storage=(S/h_b^3)

h_b=height of breach=10.058m

S=storage at failure =1902028.99-2360884.23 m^3

$$S^* = \left(\frac{1902028.99}{10.058^3}\right) = 1869.31 \text{(Lower bound)}$$

$$S^* = \left(\frac{2360884.23}{10.058^3}\right) = 2320.276 \text{ (Upper bound)}$$

K_0=1.4 because there is overtopping

$B^*_{avg}=0.47*1.4*(1869.31)^{0.25}=4.326$ (Lower bound)

$B^*_{avg}=0.47*1.4*(2320.276)^{0.25}=4.566$ (Upper bound)

But $B^*_{avg}=B_{avg}/h_b$

So $B_{avg}= B^*_{avg}*h_b$

$B_{avg}=4.326*10.058=43.51m$ (Lower bound)

$B_{avg}=4.566*10.058=45.93m$ (Upper bound)

$\Rightarrow \boxed{43.51m< B_{avg}<45.93m}$

In the 1995's paper, the new equations became:

$B=0.1803K_0V_w^{0.32}\, h_b^{0.19}$

Where K_0=constant=1.4 because there is overtopping

$$B=0.1803*1.4*(1902028.99)^{0.32}(10.058)^{0.19}=39.98 \text{ m (Lower bound)}$$

$$B=0.1803*1.4*(2360884.23)^{0.32}(10.058)^{0.19}=42.85 \text{ m (Upper bound)}$$

Therefore, $\boxed{39.98 \text{ m} < B < 42.85 \text{ m}}$

Reclamation Method

Suggested that:

$B = 3h_w$

$$B=3*10.36=31.08 \text{ m (Lower bound)}$$

$$B=3*11.58=34.74 \text{ m (Upper bound)}$$

$\Rightarrow \boxed{31.08 \text{ m} < B < 34.74 \text{ m}}$

Von Thun and Gillette Method

They suggested that:

$B_{avg}=2.5h_w+C_b$

In the Timber lake case, C_b=18.3 because $V_w=1.902028*10^6-2.360884*10^6 \text{ m}^3$

$$B_{avg}=2.5*10.36+18.3=44.2 \text{ m (Lower bound)}$$

$$B_{avg}=2.5*11.5824+18.3=47.256 \text{ m (Upper bound)}$$

$\Rightarrow \boxed{44.2 \text{ m} < B_{avg} < 47.256 \text{ m}}$

Estimation of Overtopping Depth $d_{overtop}$

The overtopping depth has been suggested by only one method: Singh and Snorrason.

Singh and Snorrason Method

They suggested that the overtopping depth range as:

$\boxed{0.15\text{ m} \quad d_{ovtopp} \quad 0.61\text{ m}}$

Which violates our assumption that 0.36 m d_{ovtopp} 1.58 m for the Timberlake dam.

Estimation of Volume of Eroded Materials V_{er}

Only Macdonald and Langridge Monopolis have established a method to estimate the volume of eroded materials.

MacDonald and Langridge-Monopolis Method

They estimated the quantity of eroded embankment materials V_{er} (m^3) for earth dam.

$V_{er} = 0.0261(V_{out}*h_w)^{0.769}$

Where V_{out}=volume of water discharged through the breach which in this case, is equal to V_w because the entire dam has failed, so all the stored water passed by the breach.

h_w=hydraulic depth of water at dam at failure above breach bottom =10.36 m-11.5824 m

In this case:

$V_{er} = 0.0261(1902028.99*10.36)^{0.769}$=10621 m^3 (Lower bound)

$V_{er} = 0.0261(2360884.23*11.5824)^{0.769}$=13664.57 m^3 (Upper bound)

Estimation of Failure Time t_f

Several researchers have estimated the failure time. The most important one are given below.

Singh and Snorrason Method

They offered the range for the failure time as:

$\boxed{0.25 \text{ hr} \quad t_f \quad 1.0 \text{ hr}}$

We can't comment on the t_f because there is no actual recorded exact duration for the time of failure of the Timber Lake dam.

MacDonald and Langridge-Monopolis Method

They suggested that:

$t_f = 0.0179(V_{er})^{0.364}$

$t_f = 0.0179(10621)^{0.364}$=0.52 hr (Lower bound)

$t_f = 0.0179(13664.57)^{0.364}$=0.57 hr (Upper bound)

$\Rightarrow \boxed{0.52 \text{ hr} < t_f < 0.57 \text{ hr}}$

FERC Method

They suggested that:

$\boxed{0.1 < t_f < 0.5 \text{ hours} \quad \text{(non-engineered, poorly compacted)}}$

But no exact t_f value is available.

Froehlich (1987,1995) Methods

In the 1987 paper, he suggested the following:

$t_f^* = 0.79(S^*)^{0.47}$

t_f^*=dimensionless breach formation time $t_f/(gh_b)^{0.5}$

$t_f^* = 0.79(1869.31)^{0.47}$=272.469 (Lower bound)

$t_f^* = 0.79(2320.276)^{0.47}$=301.599 (Upper bound)

So $t_f=t_f^*(gh_b)^{0.5}$

$t_f=272.469(9.81*10.058)^{0.5}$=2706.496s (Lower bound)

$t_f=301.5\backslash 99(9.81*10.058)^{0.5}$=2995.85s (Upper bound)

Therefore $\boxed{2706.496\text{ s}<t_f<2995.85\text{ s or }0.7518\text{ hr}<t_f<0.832\text{ hr}}$

In 1995,the new equations became:

Also:

$t_f=0.00254V_w^{0.53}\ h_b^{(-0.9)}$

$t_f=0.00254(1902028.99)^{0.53}\ (10.058)^{(-0.9)}$=0.67 hr (Lower bound)

$t_f=0.00254(2360884.23)^{0.53}\ (10.058)^{(-0.9)}$=0.76 hr (Upper bound)

Therefore $\boxed{0.67\text{ hr}<t_f<0.76\text{ hr}}$

Reclamation Method

Suggested that:

t_f(hours) = 0.011B

t_f=0.011*31.08=0.34 hr (Lower bound)

t_f=0.011*34.74=0.38 hr (Upper bound)

$\Rightarrow \boxed{0.34\text{ hr}<t_f<0.38\text{ hr}}$

Von Thun and Gillette Method

They suggested that t_f calculation depends on the materials ‘resistance for erosion:

$t_f = 0.020h_w + 0.25$ (erosion resistant)

$t_f = 0.020*10.36 + 0.25$=0.4572 hr (Lower bound)

t_f = 0.020*11.5824 + 0.25=0.4816 hr (Upper bound)

$\Rightarrow$ $\boxed{\text{0.4572 hr} < t_f < \text{0.4816 hr if the dam is erosion resistant}}$

Now:

$t_f = 0.015h_w$ for easily erodible materials

t_f = 0.015*10.36=0.1554 hr (lower bound)

t_f = 0.015*11.5824=0.1737 hr (upper bound)

$\Rightarrow$ $\boxed{\text{0.1554 hr} < t_f < \text{0.1737 hr if the dam is easily erodible}}$

Moreover, they have suggested other equations that estimate the time of failure using the average lateral erosion rate (the ratio of the final breach width to breach formation time) and depth of water above the breach invert. They conclude that there is a better estimation using these equations than the first ones that they developed. These new equations are ([16], p.16):

$$t_f = \frac{B_{avg}}{4h_W} \quad \text{(Erosion resistant)}$$

In this case:

$$t_f = \frac{44.2}{4*11.5824} = 0.954 \text{ hr (Lower bound)}$$

$$t_f = \frac{47.256}{4*10.36} = 1.14 \text{ hr (Upper bound)}$$

$\Rightarrow$ $\boxed{\text{For not easily eroded materials, is estimated to be: 0.954 hr} < t_f < \text{1.14 hr}}$

$$t_f = \frac{B_{avg}}{(4h_w + 61)} \quad \text{(Highly erodible)}$$

In this case:

$$t_f = \frac{44.2}{(4*11.5824 + 61)} = 0.4118 \text{ hr (Lower bound)}$$

$$t_f = \frac{47.256}{(4*11.5824+61)} = 0.4402\,\text{hr (Upper bound)}$$

$\Rightarrow$ $\boxed{\text{For highly erodible materials: } 0.411 \text{ hr} < t_f < 0.461 \text{ hr}}$

Estimation of Side Slope Horizontal Factor Z

Most of the methods that suggest the Z factor, they suggest a range for it not more.

FERC Method

FERC suggested for a non-engineered dam 1 <Z< 2

But at most, in this case Z=1.36=((180-90)/2)/33

Froehlich (87,95) Method

He suggested that:

$Z=0.075\ K_c(h^*_w)^{1.57}(W^*_{avg})^{0.73}$

Where h^*_w=dimensionless height of water above breach bottom (h_w/h_b) and

W^*_{avg}=average dimensionless embankment width ($W_{crest}+W_{bottom}$)/($2h_b$)

No core was present so K_c=1

$$h^*_w = \left(\frac{10.36}{10.058}\right) = 1.03\,\text{(Lower bound)}$$

$$h^*_w = \left(\frac{11.5824}{10.058}\right) = 1.151\,\text{(Upper bound)}$$

W_{crest}=600ft=182.88m

W_{bottom}=assume 500 ft=152.4m

So $W^*_{avg} = \left(\frac{182.88 + 152.4}{2 * 10.058}\right) = 16.66$

$Z=0.075*1*(1.03)^{1.57}(16.66)^{0.73}=0.612$ (Lower bound)

$Z=0.075*1*(1.151)^{1.57}(16.66)^{0.73}=1.122$ (Upper bound)

Where Z is the side slope horizontal factor

Therefore: $\boxed{0.612<Z<1.122}$

K_c = constant=0.6 if there is a core or 1.0 if no core is present

In his 1995 paper he suggested that Z= 1.4 if there is overtopping so:

$\boxed{\text{Z=1.4 because there is overtopping}}$

Singh and Scarlatos Method

They found that the top width is 106% to 174% larger than the bottom width

$\boxed{0.09<Z<1.12}$ with an average of 129% and an acceptable standard deviation of 18 %.

In this case, at maximum B_{top}/B_{bottom} =180/90=2 or 200% so it is close to what they suggested

They suggested that the breach side slopes were inclined 40^o to 80^o with the horizontal.

In our case the breach inclination is:

$Tan^{-1}(180-90)/2/33)=53.67^o$ with the (assuming a trapezoid with bottom width 90 ft and top width of 180 ft).

And in the worst case, the breach shape will be a rectangle so with 90^o angle with horizontal.

$\boxed{\Rightarrow \quad \text{The breach side slopes are between } 53.67^o \text{ and } 90^o}$

Von Thun and Gillette Method

In their work, they assumed for such types of dams that the side slopes of breach are 1H: 1V or 45^o so $\boxed{Z=1.}$

Whereas, in our case the smallest observed angle can be 53.67^o.

Summary of Results

Table 12 summarizes the results.

Table 12.Summary of results

Parameter	Method	Equation	Value
Peak Flow Q_p	SCS	$Q_p=16.6h_w^{1.85}$ for H_w>31,4 m $Q_p=0.000421(V_wH_w/WH)^{1.35}$ for H_w<31.4 m	1254.65-1541.59 m^3/s 691.17-1894 m^3/s
	Macdonald and Langridge - Monopolis	$Q_p=1.175(V_wH_w)^{0.41}$	1150.33-1315.69 m^3/s
	Costa	$Q_p=0.763(V_wH_w)^{0.42}$	883.58-1013.93 m^3/s
	Froelich	$Q_p=0.607V_w^{0.295}H_w^{1.24}$	784.51-960.18 m^3/s
Breach Width B or W	Johnson and ILLes	$0.5h_d \quad B \quad 3h_d$	5.029-30.174 m
	Singh and Snorrason	$2h_d \quad B \quad 5h_d$	20.116-50.291 m
	FERC	$h_d \quad B \quad 5h_d$	10.058-50.291 m
	Froehlich 1987 (B_{avg})	$B^*avg=0,47K_0(S^*)^{0.25}$ and $B_{avg}=B^*_{avg}{}^*h_b$	43.51-45.93 m
	Froehlich 1995	$B=0.1803K_0V_w^{0.32}h_b^{0.19}$	39.98-42.85 m
	Reclamation	$B=3h_w$	31.08-34.74m
	Von Thun and Gillette (B_{avg})	$B_{avg}=2.5h_w+C_b$	44.2-47.256 m
Time of Failure t_f	Singh and Snorrason	Given	0.25-1 hr
	Macdonald and Langridge - Monopolis	$t_f=0.0179(V_{er})^{0.364}$	0.52-0.57 hr
	FERC	Given	0.1-0.5 hr
	Froehlich 1987	$t_f^*=0.79\ (S^*)^{0.47}$	0.75-0.83 hr
	Froehlich 1995	$t_f=0.00254V_w0.53h_b^{(-0.9)}$	0.67-0.76 hr
	Reclamation	$t_f=0.011B$	0.34-0.38 hr

	Von Thun and Gillette (f(h_w)erosion resistant)	$t_f=0.02h_w+0.25$	0.46-0.48 hr
	Von Thun and Gillette (f (h_w) easily erodible)	$t_f=0.015h_w$	0.15-0.17 hr
	Von Thun and Gillette (f (avg lateral erosion rate), erosion resistant)	$t_f=B_{avg}/4h_w$	0.954-1.14 hr
	Von thun and Gillette (f (avg lateral erosion rate), easily erodible)	$t_f=B_{avg}/(4h_w+61)$	0.41-0.46 hr
Z factor	FERC	Given	1-2
	Singh and Scarlatos	Given	0.09-1.12
	Froelich (87)	$Z=0.075\ K_c(h^*_w)^{1.57}(W^*_{avg})^{0.73}$	0.62-1.122
	Froelich (95)	Given	1.4
	Von Thun and Gillette	Given	1
	Mac Donald and Langridge-Monopolis	Given	0.5

Comparison

In what follows a comparison of the different parameters calculated from different methods is held.

Flow

Table 13 shows the estimated values of the peak flows that pass through the breach of the dam.

Table 13.Comparison of peak flows through the breach

Method	Lower bound (m^3/s)	Upper bound (m^3/s)
SCS	691.17	1541.59
Macdonald and Langridge-Monopolis	1150.33	1315.69
Costa	883.58	1013.93
Froehlich (95)	784.51	960.18

For a sudden breach following Chow (1959), discharge per unit width q:

$q= 8(gy_1^3)^{1/2}/27$

With y_1=height of water behind the dam before the break

$q=8(9.81*10.058^3)^{1/2}/27=29.6m^3/sec\text{-}m$

For a breach width of say 40 m, we obtain the total discharge as $Q=29.6*40=1184\ m^3/s$.

This flow is very similar to that estimated by the other methods.

It is clear that the SCS method gives both the lowest and highest limit for the peak flow with the widest range. All the other 3 methods give a relatively small range. In this case, it seems that both Froehlich and Costa's methods are not conservative at all compared to the SCS. It seems that the best method for calculating the peak flow of the Timberlake dam is the SCS because it is the most conservative in the upper bound.

Time of Failure

Table 14 shows the estimated values for the time of failure t_f.

Table 14.Comparison of time of failure t_f

Method	Lower bound (hr)	Upper bound (hr)
Singh and Snorrason	0.25	1
Mcdonald and langridge-Monopolis	0.52	0.57
FERC	0.1	0.5
Froelich (87)	0.7518	0.83
Froelich (95)	0.67	0.76
Reclamation	0.34	0.38

Von Thun And Gillette (erosion resistant)	0.46	0.48
Von Thun and Gillette (f (h_w) easily erodible)	0.15	0.17
Von Thun and Gillette (f (avg lateral erosion rate), erosion resistant)	0.954	1.14
Von Thun and Gillette (f (avg lateral erosion rate), easily erodible)	0.41	0.46

As we realize, most of the methods suggest that the time of failure is less than 1 hour, and is specifically around 30 minutes. The lowest value (0.1 hr) is given by FERC and the highest one (1.14 hr) is given by Von Thun , this last value takes into account the average lateral erosion rate for an erosion resistant structure. That is why in order to be safe and the population downstream be evacuated, the downstream region of the dam, on average, should be evacuated in less than 30 minutes after the breach formation begins taking place.

Breach Width

Table 15 shows the estimated values for the breach width B or W.

Table 15.Comparison of the different breach width B or W

Method	*Lower bound (m)*	*Upper bound (m)*
Johnson and Illes	5.029	30.174
Singh and Snorrason	20,116	50.291
FERC	10.058	50.291
Froehlich (87) (B_{avg})	43.51	45.93
Froehlich (95)	39.98	42.85
Reclamation	31.08	34.74
Von Thun and Gillette (B_{avg})	44.2	47.256

The first 3 methods give a wide range for the general breach width; whereas both Froehlich (95) and Reclamation give a narrow range with an average around 37 m. Froehlich (87) and Von Thun and Gillette methods predict the average width of the dam. Because of the different ranges of the several methods, it seems that FERC is the most appropriate for this case because it gives the widest range.

Z Factor

Table 16 shows the estimated values for the Z horizontal slope factor.

Table 16.comparison of the Z factors

Method	*Lower bound*	*Upper bound*
FERC	1	2
Singh And Scarlatos	0.09	1.12
Froehlich (87)	0.62	1.12
Froehlich (95)	1.4	1.4
Von Thun and Gillette	1	1
MacDonald and Langridge-Monopolis	0.5	0.5

The Z values are around 0 and 2 .It seems that the lower bound of the Singh Scarlatos is the steepest (rectangular shape) and the upper bound of the Froehlich (95) will fit everything in between. MacDonald and Langridge-Monopolis assumed a steep slope.

Comparison of Peak Outflows with Other Determined Parameters

In this part of this chapter, a close look on some analysis between the peak outflow and the other breach parameters is held for the available methods.

NOTE: The range of each method is correct but no values in this range are calculated because we are interested here in only the upper and lower bounds not what occurs in between. Therefore, the plots given in what follows are drawn as straight lines although this is not true because no value is found inside the range.

Q vs. t_f

The only 2 methods that give us both the peak breach outflow and the time of failure are the Macdonald and Langridge-Monopolis and the Froehlich (1995). Table 17 shows these results and figure 36 plots them.

Table 17.Q vs. t_f

	Q		t_f	
Method	**Lower Bound (m^3/s)**	**Upper Bound (m^3/s)**	**Lower Bound (hr)**	**Upper Bound (hr)**
Macdonald and Langridge-Monopolis	1150.33	1315.69	0.52	0.57
Froehlich (95)	784.51	960.18	0.67	0.76

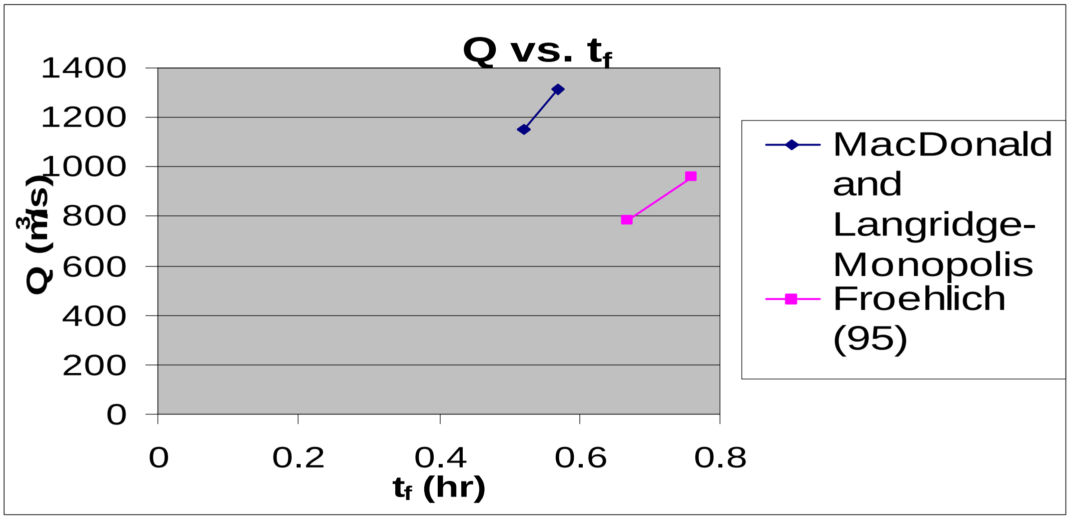

Figure 35.Q vs. t_f

From figure 36,it is clear that with the Macdonald's equation you will have a big outflow with a low time of failure whereas with Froehlich, there is a large time of failure compared to a low flow. These 2 formulas prove that the peak outflow and the time of failure are inversely proportional. In other words: $Q=F(1/t_f)$.

Q vs. B

Only Froehlich's method (1995) gives both the peak breach outflow and the breach width. Table 18 shows these results and figure 37 plots them.

Table 18.Q vs. B

	Q		B	
Method	**Lower Bound (m^3/s)**	**Upper Bound (m^3/s)**	**Lower Bound (m)**	**Upper Bound (m)**
Froehlich (95)	784.51	960.18	39.98	42.85

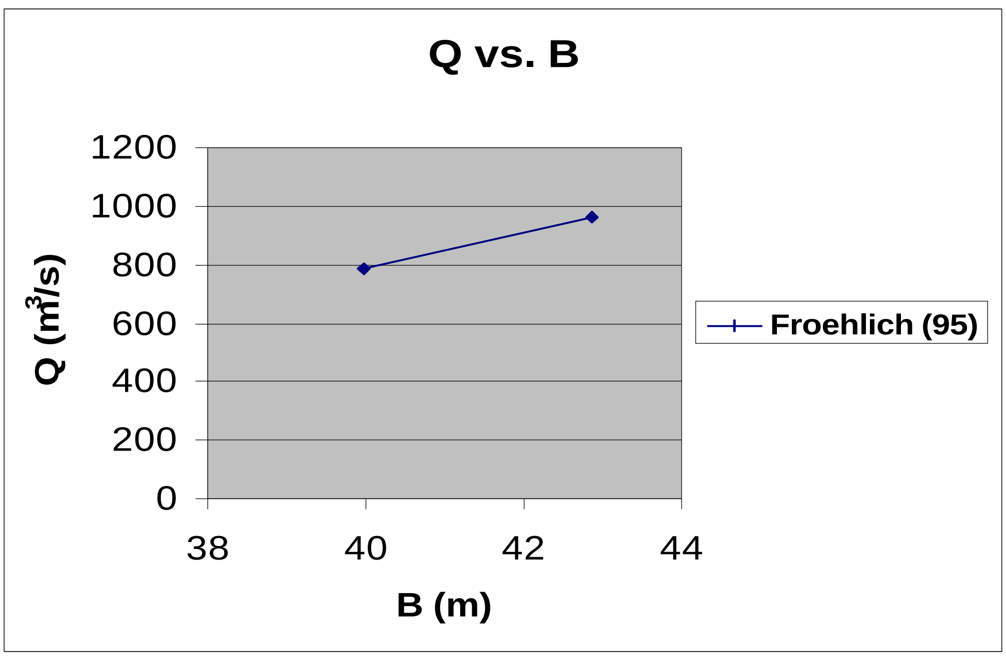

Figure 36.Q vs. B

Figure 37 shows that as you increase the breach width, the peak outflow will increase and vice versa. Therefore it seems that Q and B are directly proportional. In other words: Q=F (B).

Q vs. Z

The only 2 methods that give us both the peak breach outflow and the slope horizontal factor are the Macdonald and Langridge-Monopolis and the Froehlich (95). Table 19 shows these results and figure 38 plots them.

Table 19.Q vs. Z

	Q		Z	
Method	**Lower Bound (m^3/s)**	**Upper Bound (m^3/s)**	**Lower Bound**	**Upper Bound**
Macdonald and Langridge-Monopolis	1150.33	1315.69	1150.33	0.5
Froehlich (95)	784.51	960.18	784.51	1.4

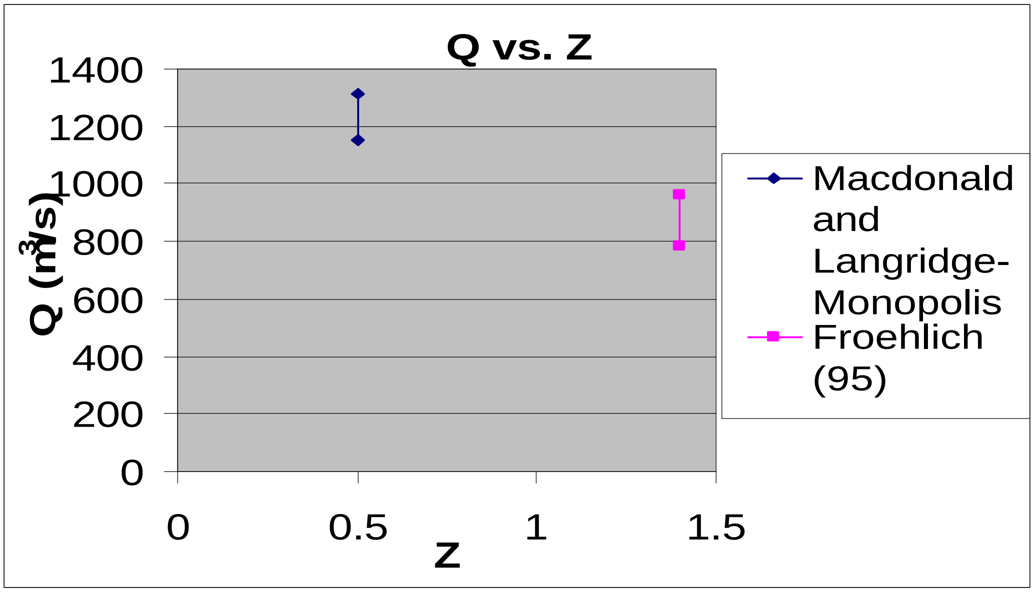

Figure 37.Q vs. Z

Figure 38 shows that as you increase Z the peak outflow Q decreases but also Z for these 2 methods are kept constant. Therefore Q and Z can be inversely proportional but it is not sure, or: Q=F (1/Z).

Conclusion and Recommendations

Most of the empirical methods were based on case studies, therefore the predicted results depend on the different types and numbers of cases studies .For example, if 10 cases studies are used in a model, they will give a certain result, whereas if there are 20 cases, it will be more accurate perhaps or even it can also be missing the point because of the difference of shapes and sizes of breaches and type of materials forming the dam. So what can be suggested, is that each method should be very specific to a certain size and shape of certain type of embankment dams.

From the application of the empirical methods on the Timberlake dam, one can conclude that the longer the time of failure is, the smaller is the peak flow and the same is with the Z factor: as the slope become steeper the peak flow increases. Whereas, the breach width is directly proportional to the peak outflow. But these conclusions cannot be general because the application was only on one specific case study (Timberlake dam). Moreover, the found results are specific for this case study

A good model for a dam breach should apply to the overtopping, piping and internal seepage but the focus will be on overtopping. The model should apply for any type of embankment dams.

Physical models should be used at large scales in order to ignore the issues of the dam's materials properties and hydraulic conditions .The problem with the materials is that they can vary a lot from one place to another, so a wide scale will make a lot of those issues more negligible and conceivable. Moreover, those models should take care of the foundation, headwater and tailwater conditions.

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