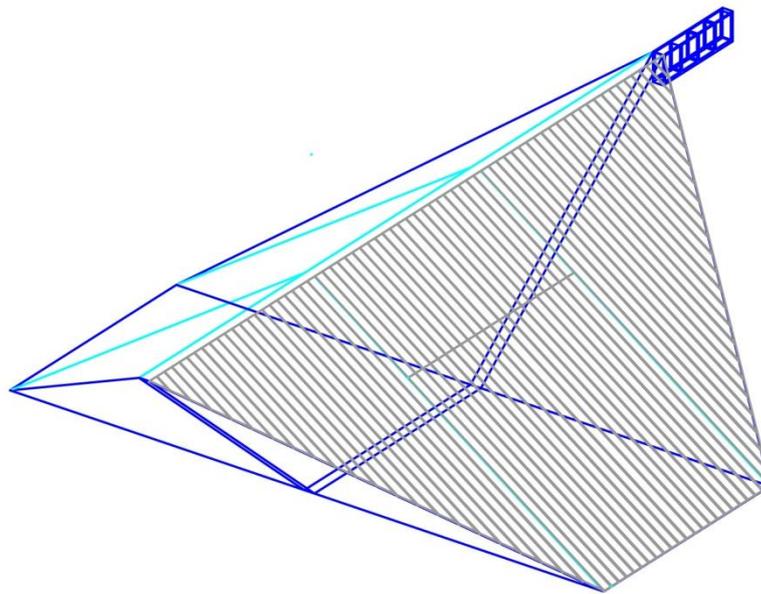


# CHALMERS



## Concrete Face Rock-fill dam compared to Roller Compacted Concrete dam

A case study of dam constructions in Panama

*Master of Science Thesis in the Master's Programme Geo and Water Engineering*

**TOM KARLSSON AND JONAS TALLBERG**

Department of Civil and Environmental Engineering  
*Division of Water Environment Technology*

CHALMERS UNIVERSITY OF TECHNOLOGY  
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Examensarbete / Master of Civil Engineering  
Chalmers tekniska högskola 2011:120

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Cover:

An Autocad figure of a CFRD design for Changuinola II, described in *Chapter 4*.

Department of Civil and Environmental Engineering  
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## ABSTRACT

This report compares two different dam types (Roller Compacted Concrete Dam and Concrete Face Rock-fill Dam) in order to help the decision making when choosing what dam type to build in Panama, where Vattenfall is involved in a joint venture to build a dam called Changuinola II.

First a literature study was conducted to build a basic understanding regarding the two different dam types. Material for this part of the report was mostly gathered through conventional studying of books and reports, but also from conversations and lectures with senior staff members at both Chalmers University of Technology and VPC (Vattenfall Power Consultants).

The case study focuses on how other dam projects have progressed and what we can learn from past experience. The dam projects studied are the construction of Changuinola I, two dams in Rio Esti and the rejected project Changuinola-75. We found that the studying of Changuinola I was of particular interest due to the fact that that project faced the same issues this report is trying to answer.

Finally our findings are applied to the project of Changuinola II. We conclude that Changuinola II should be constructed as an RCCD due to facilities, such as batching plants and logistics centres, from Changuinola I still being present in the area.

Furthermore, a general proposal of which dam type that should be constructed cannot be made due to the fact that external factors affect the choice hugely.

Key words: CFRD, Concrete face rock-fill dam, RCCD, Roller Compacted Concrete dam, dam, construction, Changuinola

Jämförelse mellan två dammtyper, Concrete face rock-fill dam och Roller compacted concrete dam

En fallstudie av dammbyggnader i Panama

Examensarbete inom Geo and Water engineering

TOM KARLSSON OCH JONAS TALLBERG

Institutionen för Bygg- och miljöteknik

Avdelningen för Vatten Miljö Teknik

Chalmers tekniska högskola

## SAMMANFATTNING

Det här examensarbetet fokuserar på att jämföra två olika dammtyper för att underlätta Vattenfalls beslutsprocess då företaget är involverat i ett dammbyggnadsprojekt i Panama. De två dammtyper som jämförs är en så kallad ”Roller Compacted Concrete Dam” (i examensarbetet kallat RCCD) och en ”Concrete Face Rockfill Dam” (i examensarbetet kallat CFRD).

Först utfördes en litteraturstudie för att bygga på den basala kunskapen om de olika dammtyperna. Den största delen av litteraturstudien gjordes med böcker, men även seminarier och samtal samt undervisning av kompetent personal på både Chalmers och Vattenfall Power Consultants förekom.

Examensarbetet fortsätter sedan med en fallstudie av tidigare dammar som har byggts i området där Vattenfall varit en del av projekteringen. De dammar som studerats är Changuinola I, en damm vid floden Rio Esti samt ett förkastat projekt vid namn Chan-75 som aldrig blev byggt. I fallstudien fann vi att Chan-75 var av störst intresse för oss då projektet hanterade samma frågeställningar som examensarbetet försöker besvara.

Slutligen appliceras resultaten av fall- och litteraturstudien på Changuinola II projektet. Rekommendationen är att Changuinola II byggs som en RCCD av anledningar som redovisas i rapporten.

Nyckelord: CFRD, Concrete face rock-fill dam, RCCD, Roller Compacted Concrete dam, dam, construction, Changuinola

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## **Preface**

We would like to thank everyone involved in the making of this report. Steffen Häggström, our supervisor, who has provided us with invaluable input and writing tips. We would also like to thank the staff at VPC (Vattenfall Power Consultants) who were kind enough to let us take part of their work, as well as providing us with extensive material and office space. A special mention also goes out to Anna Gyllenswärd, who has helped us with picture editing and general layout design.

Lastly we send our gratitude to our examiner, Lars Bergdahl, for making this report even possible. This would not have been done without him.

## **Abbreviations**

CFRD	Concrete faced rock-fill dam
MD&A	Malcom Dunstan & associates
RCCD	Roller compacted concrete dam
RMR	Rock mass rating
RQD	Rock quality designation
VPC	Vattenfall power consultant AB





# 1 Introduction

## 1.1 Background

In Sweden very few water power stations have been built during the last decades. Thus there is very little experience of dams built with RCC and CFR technique. Internationally, CFRDs and RCCDs are common and in many parts of the world there is a large potential for new water power stations. The company Vattenfall is currently involved in pre-studies, layout design and project management in several international projects.

Three dams in Panama are some of the international projects that Vattenfall is involved in. The dam types considered are RCCD (Roller Compacted Concrete dams) and CFRD (Concrete Faced Rock-fill Dams).

## 1.2 Purpose

The main purposes of this project is to gather and provide information, knowledge and experiences about RCCD and CFRD dam types, compare them to each other and gain knowledge that can be used in planning of other projects in the future. The increased understanding should include, but not be limited to, how the choice of dam type is done in regard to different parameters such as geotechnical and geological properties as well as accessible material and its difficulties during construction.

## 1.3 Problem

The main problem is to analyze which dam type that is preferable for the construction of Changuinola II.

This report is mainly divided into three parts. Part one is a description of RCCD and CFRD with respect to design and construction. Part two will try to answer the following questions.

- What is the main difference between RCCD and CFRD?
- How do these dams differ in regard to
  - Geological and geotechnical properties of the surrounding area?
  - Maintenance issues?
  - Seismic loads?
  - Economy in regard of construction, materials etc.?

Part three represents the main conclusions and results based on part one and two. The results are then applied to Changuinola II.

## 1.4 Method

Part one is a description of the two dam types, CFRD and RCCD. Knowledge about e.g. design, foundation and construction are gathered is presented.

Part two is based on reports from already finished projects, mainly the Barrigon Dam and Changuinola I in Panama. These projects are used as case studies along with information from other projects worldwide. The original plans for Changuinola I was to construct a CFRD at the same location, in the text referred to Chan-75. Because of far-reaching plans and the amount of background material available, we have chosen to also give a short description of this alternative. Part two also consist of a study of the area for the planned dam Changuinola II.

Finally part three is a study of Changuinola II where part one and two are used as base. Part three will describe and compare the different alternatives possible for Changuinola II with focus on the features stated below. The main focus for the evaluation of Changuinola II is on the two points below.

- Material costs for construction of dam body.
- Time spent, e.g. total construction time, man-hours, need of vehicles etc.

In addition to the topics above, following points are also discussed and evaluated.

- Time and costs used for trial tests, e.g. full scale trial test for RCCD.
- Preparation for construction, e.g. excavation, foundation, diversion and construction of batching and crush plants.
- Comparison of spillway design, space needed, difficulties etc.

These features are chosen from experiences from earlier constructions and in discussion with supervisor.

## 1.5 Delimitation

The delimitations are done in order to obtain the purpose stated above. First the report will concentrate only on the dam design and not take interest in powerhouse or any other surrounding constructions.

Because of the limitation regarding time it has not been possible to consider all costs tied to the construction. The emphasis has therefore been to the most distinguish costs, based on experiences from earlier projects. The powerhouse part of the water power plants is not treated.

The limitation will be as follow:

- Geological and geotechnical challenges
- Maintenance experiences, problems and risks
- Seismic loads
- Economy, pros and cons of the two different dam types
- Development of dam construction for the future

## 2 Dam Types

Chapter 2 in this thesis aims at finding an overall description of the two dam types, Concrete Faced Rock-fill Dams (CFRD) and Roller-Compacted Concrete dams (RCCD). The chapter is divided into two parts, one for each dam type, where each part consists of chapters describing e.g. design, construction, material use and risks.

### 2.1 Concrete Faced Rock-fill dam

Concrete faced rock-fill dams, CFRD, is the term used to describe a certain type of dam that have a dam body of rock-fills or gravel that is compacted in layers. Moreover, there is an anti-seepage system using a concrete face slab. The concrete slab works like an impervious layer while the rock-fill body consists of granular material which has a high permeability and supports the concrete face slab by giving the dam stability. *Figure 2.1* shows an example of a CFRD, the Mohale dam in Lesotho.



*Figure 2.1* Downstream view of the Mohale dam in Lesotho

(Frysinger, 2011)

One of the main characteristics with the CFRD is that the dam type enables usage of local materials from the riverbed and the compulsory excavations in the rock-fill dam body, as opposed to using expensive material from quarries which may have to be transported a long way etc. However, there are some quality requirements on the aggregates which have to be met in order to be able to use them in the dam body. The quality is mainly determined by the local geology and highlights the importance of good geological surroundings in order to exploit all advantages with the dam type.

When designing CFRDs there are certain properties that have to be assessed thoroughly such as zoning of the dam body, filling materials, design of the dam body, various analyses and seepage control to name a few. (Vncold, 2008)

### **2.1.1 History and expansion**

CFRD have a long history and have been in a hidden, but integral, part in human life for centuries. They are basically an evolved embankment dam. According to Yang (1999) the first embankment dam was built around 2000 B.C in Mesopotamia close to Baghdad.

Due to its long history there is a vast knowledge base regarding the building technique, the dam type can be found almost everywhere in the world. Embankment dams, such as the CFRD, are still being built. Current projects span all over the world, from Kárahnjúkar dam in Iceland in the north to Mohale dam in Lesotho, South Africa in the south.

### **2.1.2 Design**

There are many ways to design a CFRD. Recently Vncold (Vietnamese national committee of large dams) produced a paper containing some fundamental design guidelines on behalf of ICOLD (International committee of large dams). In this guideline there is detailed information of how the designing of the dam body should be performed, the quality of materials used, foundation treatments as well as design values for the face slab etc. The guideline summarizes experience from building CFRDs and gives a valuable overview of the potential difficulties of building a dam of this type. (Vncold, 2008)

We have decided to look in to these parameters and shortly describe them.

#### **2.1.2.1 Dam body design**

The dam body design includes several properties of the CFRD are:

- Inclination of the dam slopes
- Crest properties and parapet walls
- Height of the dam
- Settlements of the dam body

A typical cross section for a CFRD project is shown in *figure 2.2*.

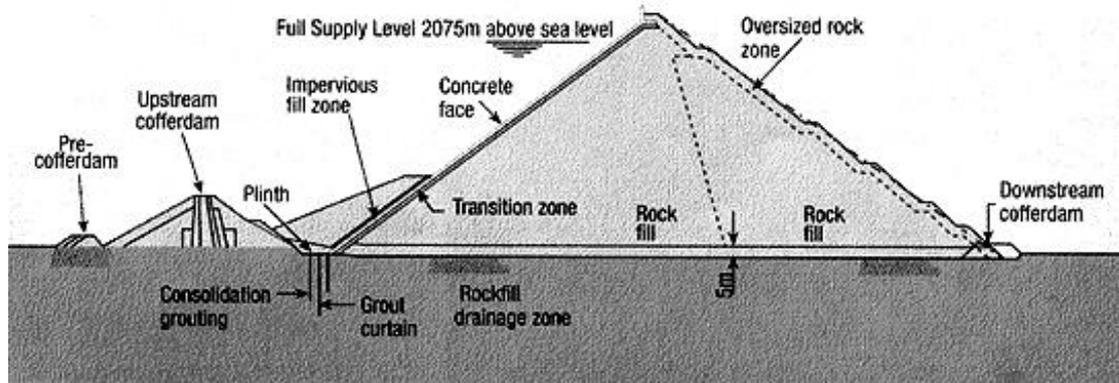


Figure 2.2 CFRD cross section with upstream and downstream cofferdams.

(Novak, et al. 2001)

When designing the dam slopes, consideration to the gravel material has to be taken. If the material is of good enough quality, the up and downstream slopes can be designed at a slope of 1:1.3 – 1:1.4. However, if the material consists of poorer quality rock-fill the slopes should not be steeper than 1:1.5 – 1:1.6. (Vncold, 2008) On the crest of the dam there should be a parapet wall. Furthermore, Vncold (2008) suggests that the crest width should be within the limits of five to eight meters wide. However, the crest width is subject to the regulations of the local authorities as well as the operation of the dam, perhaps there is a need for a road or other operations on the crest.

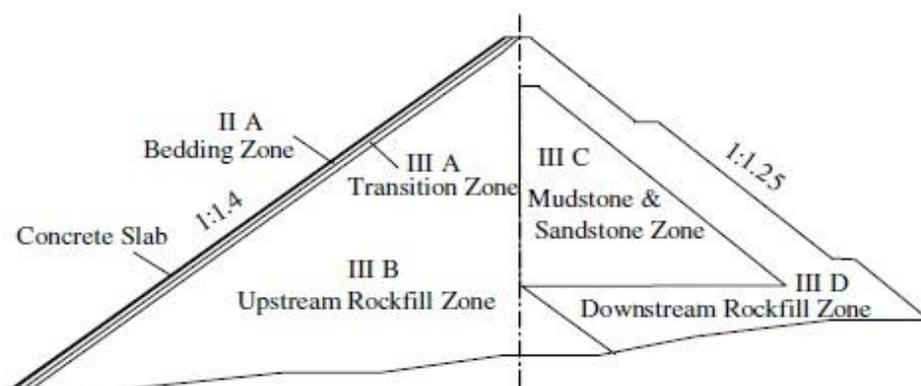
The height of the dam is decided on an economical- as well as on a practical base. The constructor and consultant have to calculate the reservoir volume and the height of the dam in order to produce enough electrical power to ensure good economy within the project. This means that also the variations of the river flow have to be considered. (Vncold, 2008)

It is also important to make an analysis of the dam design to ensure that stresses and deformation is within the set limits. Several models should be made on different parts of the dam body. These include, but are not limited to, the concrete face slab, perimeter joint of the dam body and rock-fill creep etc. When doing the analyses it is important to include dynamic loading to see how the dam responds to seismic loads. (Vncold, 2008)

If the dam is a high CFRD, above 100 meters (Vncold, 2008) it is also necessary to make a stability analysis of the ground. It is also needed if the dam is constructed on weak bedrock or a gravel foundation, if the seismic activity in the area is high, if the dam is constructed with soft rock-fill materials or if there exists difficult terrain conditions at the site.

### 2.1.2.2 Zoning of CFRD

Ensuring good material for the CFRD body is essential to keep it competitive, both from a technical and an economical stand point. The principle is that only a small part of the dam filling materials should be excavated from quarries to make it cost effective. This is enabled by dividing the CFRD body into zones that have different properties, as shown in *figure 2.3*.



*Figure 2.3 Zoning of the CFRD body*

(Zhang, Wang, & Shi, 2004)

The zones can have different relative volumes depending on how large the dam should be, surrounding geology, bank slopes etc. *Figure 2.3* **Error! Reference source not found.** (Zhang, Wang, & Shi, 2004) presents the different zones from the dam “Tianshengqiao 1” in China, where the following properties of the fillings are shown *Table 2.1*.

*Table 2.1 Material properties of zones shown in Figure 1*

(Zhang, Wang, & Shi, 2004)

Material No.	Material description	Maximum particle size [mm]
II A	Processed Limestone	8
III A	Limestone	30
III B	Limestone	80
III C	Sandstone and Mudstone	80
III D	Limestone	160

These values give us an idea of how the materials of the zonings in the CFRD body can look like. The values can vary slightly depending on the rock types of the bedrock and the aggregates available, since the values are determined by material properties.

### 2.1.2.3 Upstream sealing

The concrete face slab functions like an impermeable unit. However, there are both vertical and horizontal joints to allow some deformation on the slab to prevent cracking and thus leakage through the dam body. (Vncold, 2008)

When constructing the face slab thickness, Vncold (2008) suggest that it should be calculated as:

$$T = 0.30 + 0.0035 \times H \quad (2.1)$$

(VnCold, 2008)

Where T is the thickness in meters and H is the height of the dam, also in meters. This suggests that the slab design gets thicker at the base since the variable, H, starts at zero at the top of the dam. The minimum thickness of the slab should therefore be 0.3 m to allow for steel reinforcements and impermeable features.

$$T = 0.40 + 0.0035 \times H \quad (2.2)$$

(Vncold, 2008)



Figure 2.4 Placement of face concrete at Shuibuya dam in China.

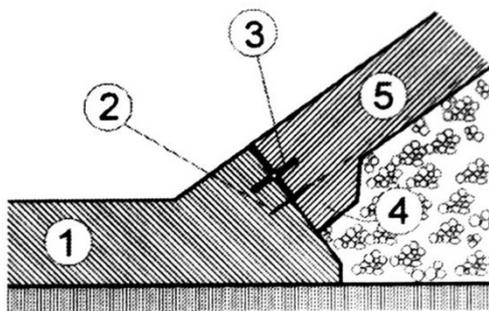
In some cases, the minimum thickness of the slab can be increased to 40 cm for certain high CFRDs in narrow valleys to account for the easy extruded failure zone of central face slabs during these conditions. (Vncold, 2008)

#### 2.1.2.4 Joint seals

Since the face slab is casted in a stage-by-stage construction and not a homogenous slab, there are joints between the different slabs. These can be sealed and reinforced in numerous ways. To prevent seepage through the joints in the face slab it is important to seal them properly. The most important joint to seal, is however the perimeter joint which is between the face slab and the concrete plinth. The plinth connects the slab with the foundation. (Tančev, 2005)

This is usually done by installing sheet plates. In some occasions asphalt is used in between the sheet plates to seal the joints even better. However, the properties of the asphalt need to be carefully chosen so that the asphalt does not become liquefied or to stiff at extreme temperatures. (Reinus, 1968)

Some seals also include copper castings as well as PVC film. The most efficient way to stop seepage through the perimeter joint is to use features of different materials and shape at the same time. Tančev (2005) proposes a design, which he states is the most efficient one, shown in *figure 2.5*.



*Figure 2.5*      *Perimeter joint*

1. *Concrete plinth*
2. *Waterstop of copper sheet*
3. *Rubber waterstop*
4. *Concrete bedding*
5. *Face slab*

(Tančev, 2005)

### 2.1.2.5 Design of concrete plinth

According to Sramoon (2010) the concrete plinth is the most important part of a CFRD. The plinth is needed to connect the foundation with the face slab to form a perimeter joint which functions as a primary water barrier. It is normally constructed with reinforced concrete.

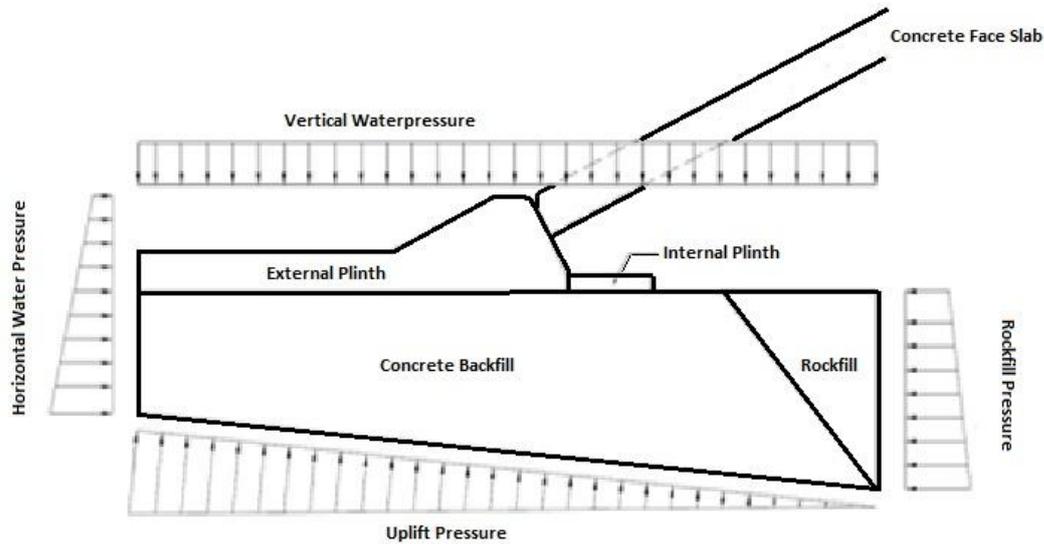


Figure 2.6 Loading conditions for the concrete plinth

(Sramoon, 2010)

The plinth is located on top of the bedrock underneath the dam. It should, according to Vncold (2008), be joined with the bedrock through grouted bolts, a so called grout curtain. In figure 2.6 the loading condition for the concrete plinth is shown to give an overview of how the loads are distributed upstream of the dam.

Table 2.2 Allowable hydraulic gradient of the rock foundation, if weathered (Vncold, 2008)

Rock rot degree	Allowable hydraulic gradient
Fresh weathering	>20
Moderate weathering	10 – 20
Intense weathering	5 – 10
Full weathering	3 – 5

The hydraulic gradient is sometimes called Darcy's slope, which means it is the difference between the head in two points divided with the distance along the flow. The allowable hydraulic gradient shows the ability of the bedrock to withstand leakage at an acceptable level, which is described in *table 2.2*. (Vncold, 2008)

#### **2.1.2.6 Spillway and bottom outlet**

There are several alternatives when designing the spillway and bottom outlet. For the spillway there are two main types, spillway passing through the dam body and spillway outside the dam body.

For embankment dam-types, such as the CFRD, the option of spillway outside the dam body is most suitable. However, the spillway outside the dam body can also be used in some concrete dams. Using crest spillways (spillways trough the dam body) may increase the safety regarding flooding and overflow but that design is not favorable for embankment dams. (Tançev, 2005)

The spillway is also designed on behalf of the maximum flood occurrence which has to be calculated so that the spillway can evacuate sufficient amounts of water. There are three main spillway designs when considering the choice of a spillway outside the dam body. These three options are:

- Ogee spillway (*figure 2.7*)
- Side-channel spillway (*figure 2.8*)
- Shaft spillway (*figure 2.9*)

The Ogee spillway (*figure 2.7*), is a spillway structure that is placed near the dam body. It consists of a constructed pathway with banks that leads the water past the dam to the riverbed downstream. In the case of soil foundation, the ogee must be prepared with facing and lining to counteract erosion etc. (Tançev, 2005)

The side-channel spillway and channel (*figure 2.8*) is very similar to the frontal spillway. However, the water intake is located to the side of the dam.

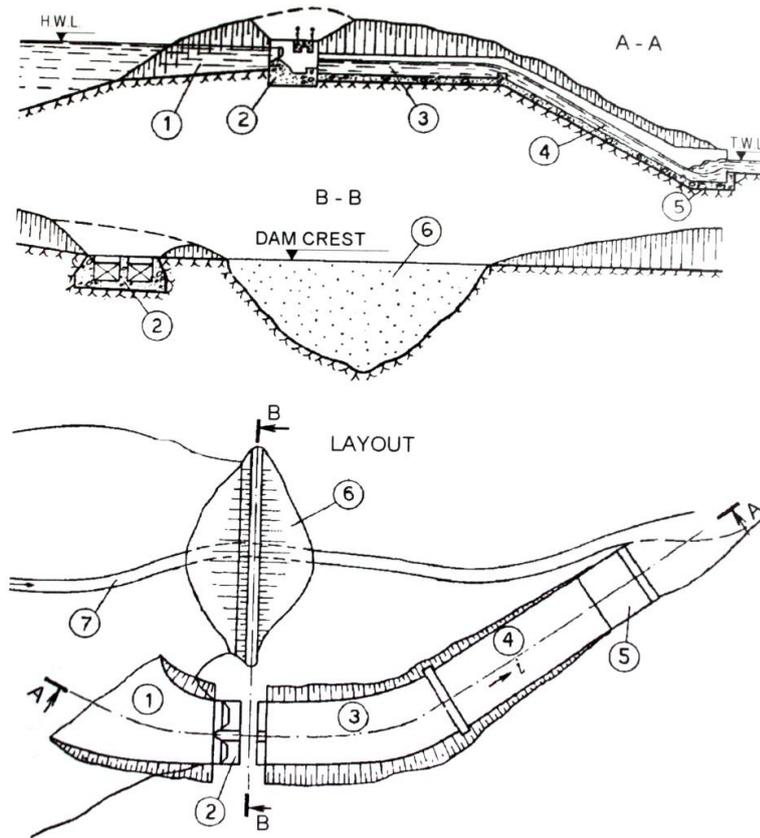


Figure 2.7 Ogee spillway

(Tančev, 2005)

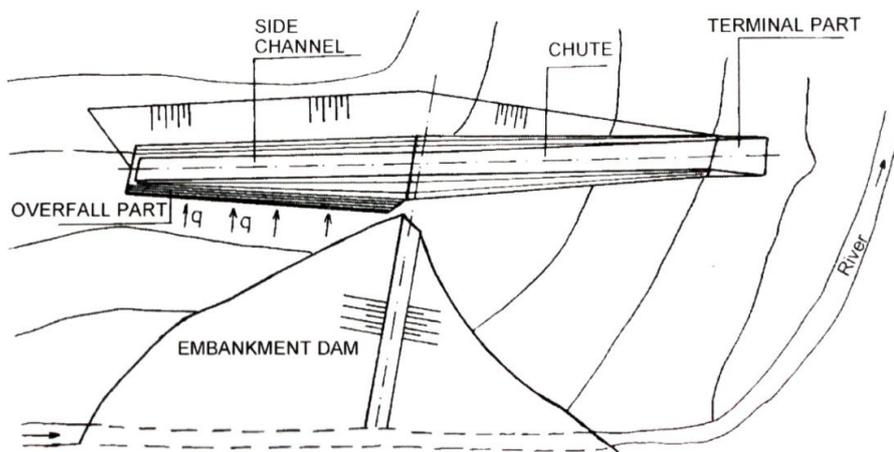
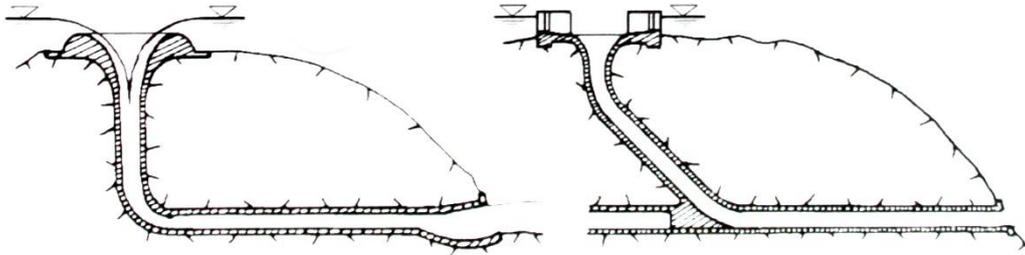


Figure 2.8 Side-channel spillway

(Tančev, 2005)

Finally, the shaft spillway (*figure 2.9*) can be constructed in various ways. Normally it consists of a vertical shaft or, in some cases, an inclined shaft. The shaft spillway is very useful when evacuating large amount of water from high reservoirs. (Tançev, 2005)



*Figure 2.9*                      *Shaft spillway*

(Tançev, 2005)

The bottom outlet should not be confused with spillways and is used for leading the water to the powerhouse and generating power. The design is depending on the surrounding topography and geological conditions. If the geological conditions are satisfactory and the topography allows for it, then the most common and easy way to construct a bottom outlet is through a tunnel on the side of the embankment dam. (Tançev, 2005)

There is also an option for constructing an outlet tunnel through the embankment dam body; however this option is more difficult and is generally not done for CFRDs. The tunnel option also allows large quantities of water which is necessary when building a dam within a reasonably large river due to the high design flows. Somewhere along the outlet the powerhouse is built, so the tunnel option also gives some freedom as to where the powerhouse should be placed. (Tançev, 2005)

The other option for an outlet is pipes or a gallery below the embankment dam, but due to the very size and quantities of water within the Changuinola project this is unfeasible. (Tançev, 2005)

### **2.1.3 Material**

According to Vncold (2008), the filling criterion varies for each subzone of the CFRD body and is based upon engineering experience. The zones of the dam can be seen in *figure 2.3*. The subzones consist of materials with different properties such as porosity, permeability and dry density.

Table 2.3 Filling criteria for dam materials

(Vncold, 2008)

Material / Zone	Porosity [%]	Relative density
Cushion material	15 – 18	
Transition zone	18 – 20	
Main rock-fill zone	19 – 21	
Downstream rock-fill zone	<22	
Gravel material zone		0.80 – 0.85

Also, the dry density can be derived from the porosity show in *Table 2.3* if the rock density is known. It is important that the mean dry density standard deviation of the materials and zones are not bigger than  $100 \text{ kg/m}^3$  to ensure that the stability of the dam is sufficient; the materials need to be homogenous. (Vncold, 2008)

#### 2.1.4 Foundation

The main purpose of constructing a good foundation for the dam is to lessen the deformation caused by the weight of the dam as well as increasing the compressive strength of the bedrock, prevent seepage under the dam and erosion that may follow seepage. According to Tanêev (2005), the requirements of any dam foundation concern deformability, stability and water impermeability.

To prevent the seepage a grout curtain is used. The curtain grout should be well into the bedrock depending on hydro geological conditions and lower the pressure lines under the dam which originates from seeping water. (Renius 1968)

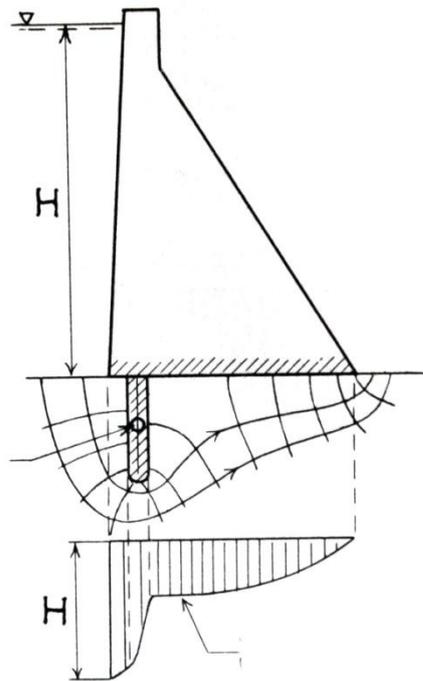


Figure 2.10 Grout curtain

(Reinius, 1968)

The curtain grout works as an impermeable shield and consist of cement, or other compounds such as silica sol, which is pumped into the bedrock under large pressures and thus sealing cracks and stabilizing the rock. (Renius, 1968)

Embankment dams, such as CFRDs, are normally built on rock. However, foundations of gravel can also be feasible due to embankment dams transmitting relatively low stresses to the foundation. If the dam is being constructed on gravel, the curtain grout becomes even more important due to the fact that gravel is much more permeable than bedrock, depending on the bedrocks RMR and RQD values. (Novak, et al. 2001)

RMR and RQD are two methods of classifying the quality of a rock by comparing, amongst other variables, discontinuities of the rock mass.

### 2.1.5 Construction

The construction process of a CFRD is fairly straight forward. However, an equilibrium rise of both upstream and downstream zones is required. When building the CFRD, the backfilling level should also be slightly higher due to different settlement behavior compared to the materials used for the front filling. (Vncold, 2008)

The pre-settlement of the dam body also needs consideration. It is inevitable that settlements will occur within the dam body and if the face slab is constructed too early, the risk of cracking and failure of the slab is increased. Vncold (2008)

recommends the presetting period to be 5 to 7 months and the settlement should not be higher than 5-6 millimeters per month depending on how large (heavy) the dam is.

The materials used for construction could be taken directly out of the riverbed or from a nearby crushing plant. If conditions allow, the gathering of materials is preferably done from the riverbed. (Vncold, 2008)

During construction a water diversion may be used by retaining water using a temporary cross section or allowing a flood passing cross the dam during extreme flows. If a temporary cross section is used, then the requirements of the temporary cross section should be met regarding sliding stability, seepage and high water conditions. (Vncold, 2008)

### 2.1.5.1 Diversion

During the construction on a dam site, it is vital to divert the river flow in order to progress with the dam building. Especially during construction of CFRDs since the gravel material can be easily flushed away or cause damage to an unfinished dam body. Both Tančev (2005) and Novak (et al. 2001) suggest some diversion options which help to facilitate the erection of the dam, shown in *figure 2.11*. During normal conditions, cofferdams are used to create a good working space for the erection but also to shelter the construction site from potential flash floods etc.

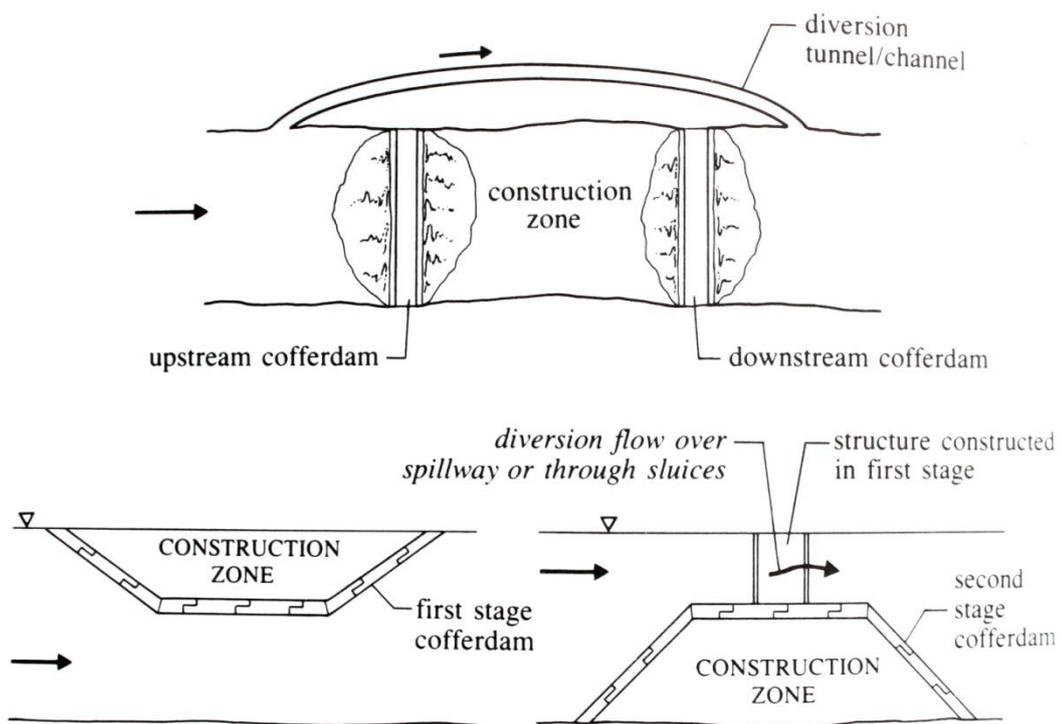


Figure 2.11 Example of diversion solution during the constructing period

(Novak, et al. 2001)

There are two different types of cofferdam designs that preferably can be used when building in a riverbed and there is a stream of water that needs to be controlled, which is the same method used for the CFRD alternative. (Tančev, 2005 and Novak, et al. 2001)

### **2.1.6 Risks and experiences**

Safety monitoring of CFRDs should be based upon local regulations in the actual country that the CFRD is being built in. However, there are some properties that should always be monitored for high CFRDs according to Vncold (2008). Systematic measurements and observations should be made before, during and after construction. The idea is that observations made during the early phase of construction should guide the construction resulting in a more optimized design in the later phase.

Vncold (2008) has designed a list with properties that should always be monitored at high CFRDs. In this report we have focused on the risks that we consider are the most hazardous for dams of concrete faced rock-fill type:

- Overtopping
- Frost and ice
- Seismic loads

In the following chapters we will try to explain what the risks mean and how to avoid them.

#### **2.1.6.1 Overtopping**

The biggest risk for a CFRD is the case of overtopping. According to (Chinnarasri, 2001) overtopping has caused CFRDs to fail numerous times in the past. When water starts overtopping the dam, an erosion process starts which will reduce the dams general stability and could cause the dam to fail.

To counter this risk, an internal warning system could be installed to indicate too large flows in the river or too high levels in the reservoir. If this warning system indicates dangerously high levels of water (usually the CFRD is designed for the 10,000 year flood), the outflow through the spillways can be increased and the risk of overtopping is then reduced although a decrease in energy output is also likely. (Chinnarasri, 2001)

#### **2.1.6.2 Frost and ice**

When building dams in colder climates, the temperature change during the year can have some unwanted effects. For CFRDs this affects the concrete face plate. Temperature contractions and expansions can cause cracking leading to leakage and erosion of the concrete. The main dam body of the CFRD is, however, fairly resistant to frost problems due to the permeable nature of the rock-fill. (Reinius, 1968)

### 2.1.6.3 Seismic

There are some cases of CFRD's being subjected to seismic forces and earthquakes. This can be potentially disastrous for the dam. It can fail or be heavily damaged due to the dynamic forces of the earthquake. (Reinus, 1968)



Figure 2.12 *Damage along the vertical joints at Zipingpu dam in China due to earthquake.*

(Wieland & Houqun, 2009)

In China this has been observed and studied by the Chinese committee of large dams (Zeping, 2009) by firsthand experience. In 2008 an earthquake stroked the CFRD in Zipingpu, measuring 8 on the Richter scale. This is the strongest earthquake a CFRD has ever been subjected to. Even as this was a very intense earthquake the dam did not fail, although the damages on the dam were substantial and the reservoir was filled with water.

The main crest of the dam settled almost 800 mm, with more settlements higher up in the dam body than in the lower parts, which should be expected. The face slab suffered heavy cracking as well as the parapet wall on the dam crest. The cracking of the face slab resulted in somewhat higher leakage. Also the seepage under the dam increased after the quake. (Zeping, 2009)

The downstream slope was also affected by the earthquake. However, the deformation of the downstream slope was not critical due to the two-part design of the slope where the lower part of the slope had a lower inclination than the higher part. (Zeping, 2009)

After all, the water stopping effect of the dam was not fundamentally compromised which concludes that CFRDs are very resilient to dynamic loads such as earthquakes. (Zeping, 2009)

## 2.2 Roller-Compacted Concrete Dams

Roller-compacted concrete dams have many similarities with conventional gravity concrete dams. The dam is built to a certain height and depth where it can resist the expected forces from the water by its weight. But instead of using rock-fill or earth-fill as a CFRD, RCCD consist of concrete which is spread in thin layers and compacted by vibrator rollers. *Figure 2.13* shows the downstream view of the Mujib dam in Jordan.



*Figure 2.13* Downstream view of the Mujib dam in Jordan.

(Lahmeyer International, 2011)

### 2.2.1 History and expansion

The method is originally from the 1970s Canada where the forest industry needed a durable paving material that could carry heavy loads. (Portland Cement Association, 2010) According to Tančev (2005), the method was applied to concrete dams already in the 1970s. The first dams were constructed in the early 1980s and PCA (Portland Cement Association) refers to the first RCCD built in the United States, the Willow Creek dam in Oregon that was finished in 1982. By 2000 more than 200 large RCCDs have been constructed and RCCD is quickly becoming one of the leading dam types in the world. (Tančev, 2005)

### 2.2.2 Design

There is no set way to design an RCCD since RCC simply stand for the construction method and material. The most common design is however the theoretical triangular cross-section which functions as a gravity dam. *Figure 2.14* shows a typical cross-section for a RCCD.

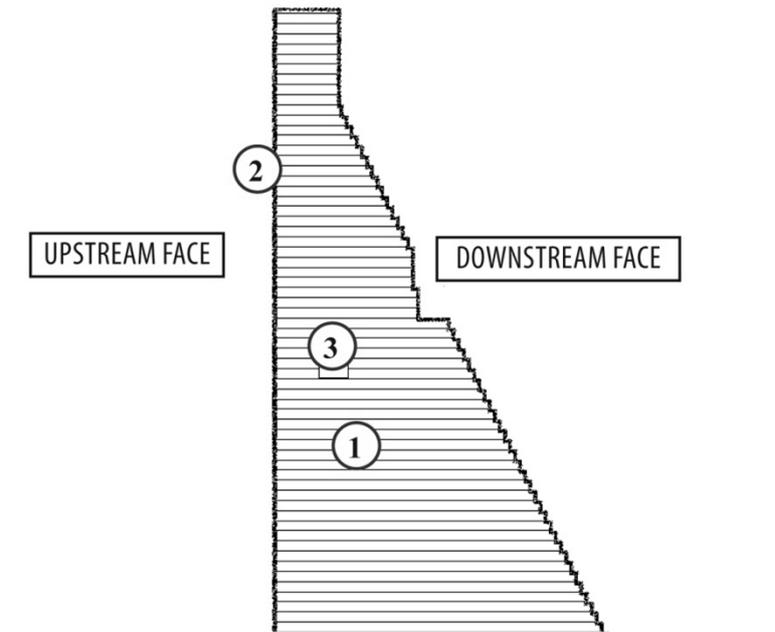


Figure 2.14 Typical cross-section for a RCCD

1. RCC-layers, approximately 300 mm
2. Sealed upstream face
3. Drainage gallery

(Tançev, 2005)

The cross-section can as well consist of a shaped crest, sloped upstream face, drainage gallery and toothed foundation. (Tançev, 2005)

### 2.2.2.1 Dam body design

The dam body can be built after various designs where arch dams and buttress dams are most common together with the gravity dam. The choice of design is normally done after considering the general environment and topography as well as geological conditions for the foundation. The dam body of Changuinola I is designed as an arch dam.

Arch dams are a type of construction method that is built to transfer the water loading from the dam itself, to the banks. Conventional gravity dams, on the other hand, transfer the loading from the upstream face to the foundation.

There are two basic construction designs for arch dams, shown in *figure 2.15*, one with constant external radius along the height of the dam and the other with constant central angle. (Tançev, 2005)

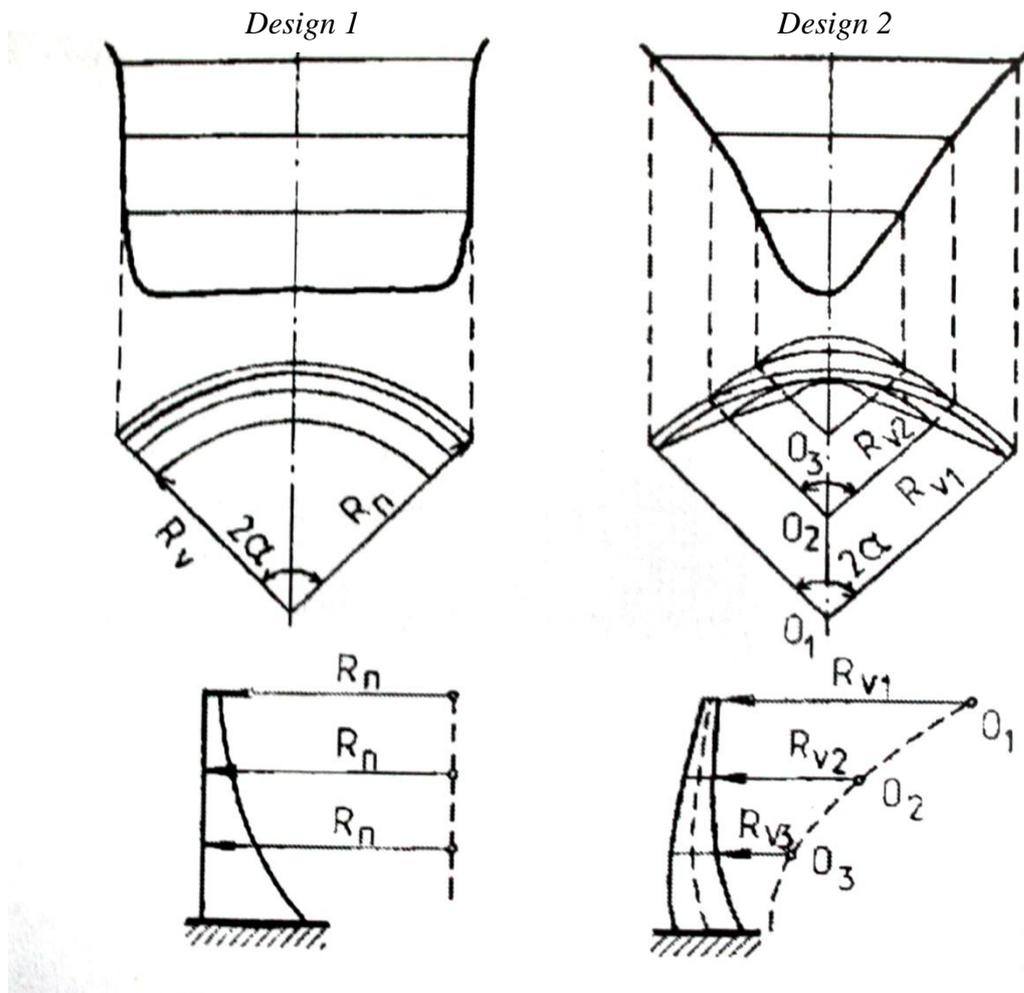


Figure 2.15 Arch dam design for RCCD

(Tančev, 2005)

### 2.2.2.2 Upstream sealing

It is important to provide the RCCD with sufficient resistance to water leakage (seepage). To achieve low permeability in the dam body Tančev (2005) suggests a number of measures that can be used for an upstream face treatment, some of them shown in figure 2.16.

- Geo-membrane
- Asphalt layer
- Higher content of binder in the cement in the vicinity of the upstream face

Both the geo-membrane and the asphalt layer are applied behind prefabricated concrete panels that are placed as the upstream face of the dam.

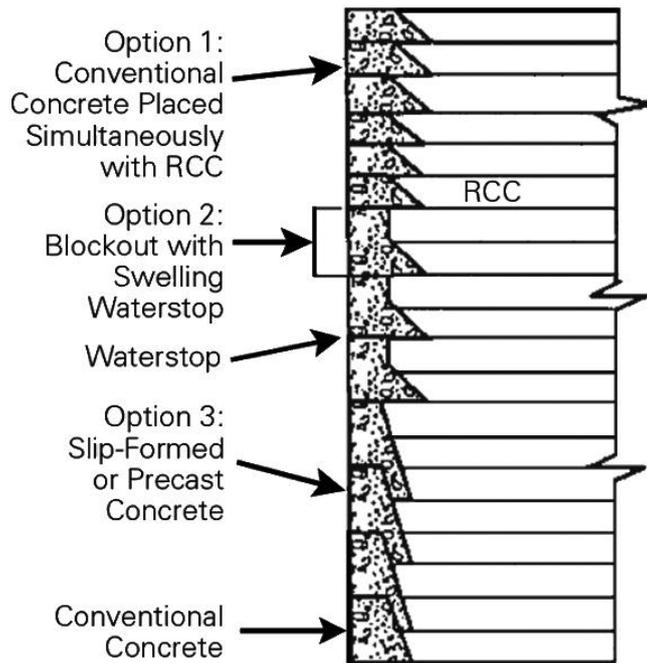


Figure 2.16 Different alternatives for upstream sealing of RCCD using conventional concrete.

(Schrader, 2011)

The method of using a concrete with higher binder close to the upstream face will decrease the permeability by creating a water tight barrier of concrete with higher quality. However, according to some theories in RCCD building the most permeable part of the dam is the joints between the RCC layers and if these joints can be sealed properly then the upstream face treatment might be unnecessary. (Tančev, 2005)

### 2.2.2.3 Spillway and bottom outlet

Commonly, crest spillways are used for concrete dams. This means that excessive flood water will be evacuated through the dam, using an overflow section. The overflow section might be placed in the middle of the dam, or more to the sides, shown in figure 2.17. (Tančev, 2005)

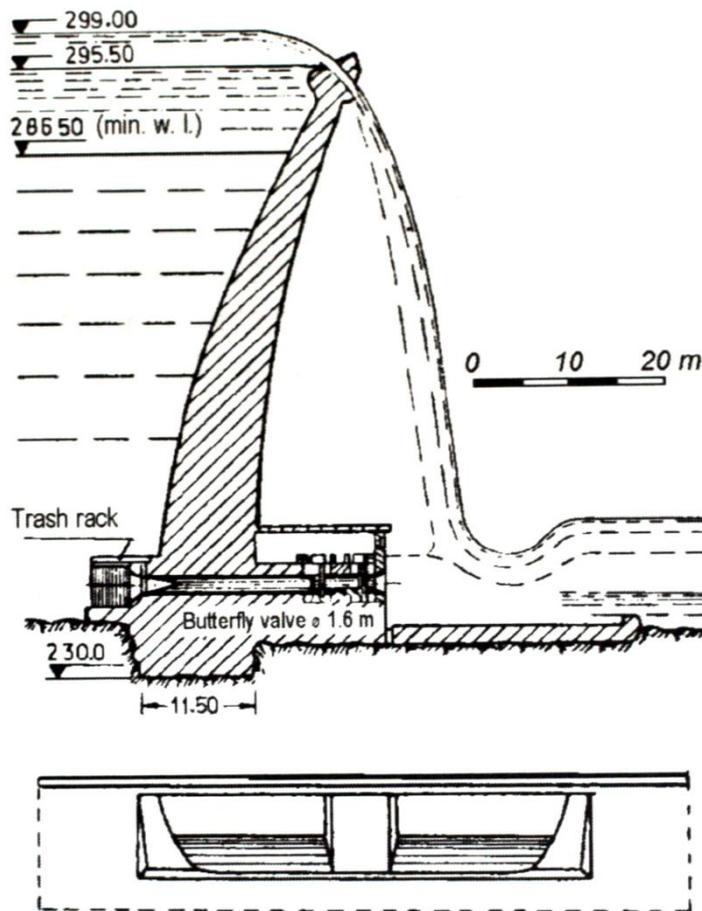


Figure 2.17 Example of overflow section

(Tançev, 2005)

Usually, the bottom outlets consist of steel pipes embedded in concrete. It is recommended to have at least two pipes, which gives more control of the outflow and reservoir levels. They are fairly straightforward to construct when dealing with concrete dams. The main purpose is to regulate the flow or provide water to the turbines in the power house. (Tançev, 2005)

### 2.2.3 Material

There are several options when it comes to the choice of material used for the RCC. Novak (et al. 2001) mentions that research, mainly done in Japan and America, has resulted in two distinct approaches in RCCD construction which demands different materials.

The two approaches will result in different dam profiles as shown in *figure 2.18*.

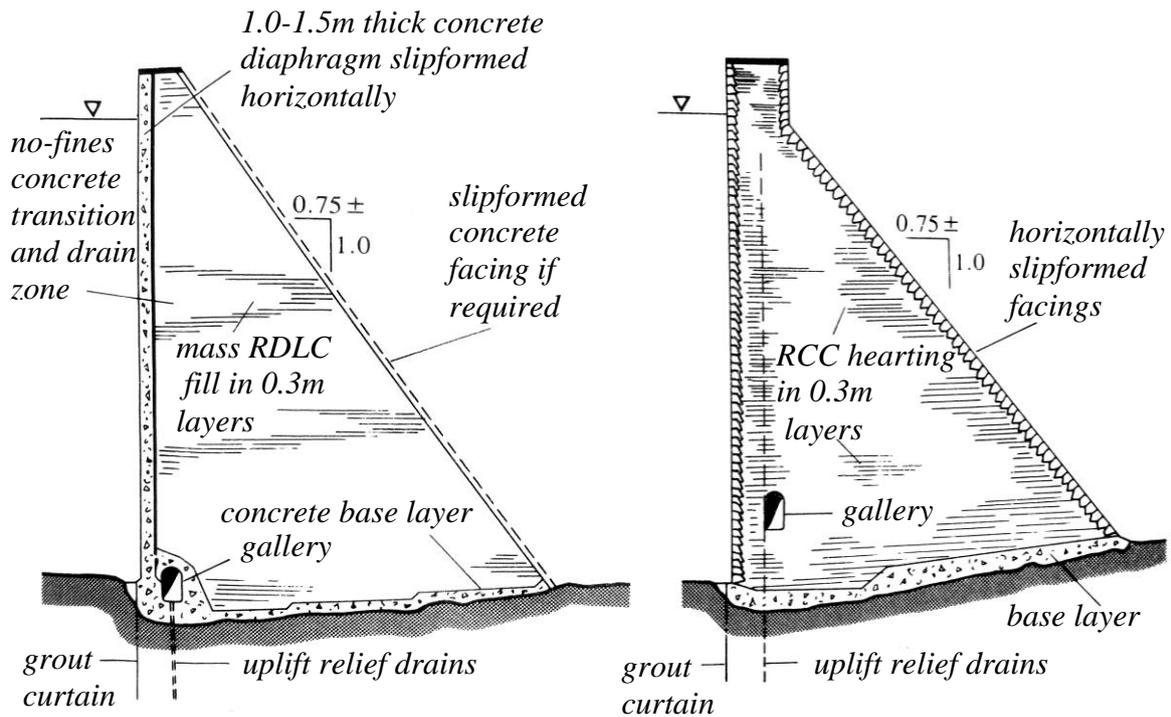


Figure 2.18

(Moffat p.133)

The main difference between the two is the upstream sealing and the possibility of constructing an optimized profile.

Since the RCCD constructed with a dry concrete that is more permeable than the option of a high-paste compacted concrete, it needs a high-quality concrete membrane for water stopping upstream. When using the dry concrete, it also becomes easier to develop an optimum profile during construction.

Table 2.4 Composites of the different concrete mixes

(Novak, Moffat, Nalluri, & Narayanan, 2001)

Characteristics	Lean RCC (RDLC)	High Paste RCC
Cement + PFA (kg/m <sup>3</sup> )	100-125	> 150
PFA / (Cement + PFA) (%)	0-30	70-80
Water to PFA + Cement ratio	1.0-1.1	0.5-0.6
90-day compressive strength (MN/m <sup>2</sup> )	8-12	20-40

Unit Weight (kN/m <sup>3</sup> )	23	25
Layer thickness (m)	0.3	0.3
Contraction Joints	Sawn	Sawn or formed

One important parameter is that the high-paste concrete, while having a higher compressive strength, is demanding a lot more PFA or cement which can be difficult to attain in desolate places. Specifications of the composites of the different concrete mixes mentioned by Novak (et al. 2001) are shown in *table 2.4*.

### 2.2.4 Foundation

Foundation of the RCCD is very similar to the foundation described for CFRD. The main difference is that the RCCD is much heavier and thus demand better bedrock allowing the stresses produced by the dam weight and the water pressure. If the bedrock is heavily foliated, the foundation may suffer from shearing failure according to Tančev (2005). Before and during construction of RCCD, the geological conditions at the site must be investigated much more thoroughly than for example during a CFRD project.

The foundation treatment of the Changuinola I dam has been covered in Design Criteria Memorandum. Several tests of rock masses and geotechnical conditions have been performed.

Due to the weight of the dam and the bedrock's general condition, an excavation foundation treatment has been carried out. The decision was made due to rather poor RMR-values and the joint conditions.

In addition to the excavation, a grout curtain should also be constructed so the foundation meets the set value of seepage of 2-3 Lugeon. (Changuinola Civil Works Joint Venture, 2009)

### 2.2.5 Construction

The big advantage with RCCDs is the rapid and simple construction. According to PCA the construction method gained their popularity "because it proved to be less expensive than conventional methods of dam construction, including rock-fill and earth-fill construction." The main reason to the smaller expense is assigned the shorter construction time (Portland Cement Association, 2011). Also in *Hydraulic structures* (Novak, et al. 2001) the shortened construction time is mentioned as one of the advantages.



Figure 2.19 Example of layer thickness for a RCCD.

(Schrader, 2011)

The conventional RCCD is constructed in 300 mm layers, as shown in *figure 2.19*. Concrete is spread from abutment to abutment by conveyor belts and dumpers before it is compacted with vibratory rollers. The technique has been evolved in the later years and the ambition to optimize the design has generated improvements of different details in the construction. Some stated below.

- The construction of RCCDs as arch dams.
- The use of an impermeable upstream barrier. GEVR (Grouted Enriched Vibratable RCC).

It is also important to have a full-scale testing of the RCC to ensure the quality of the construction and that the concrete is impermeable enough, but also that the construction equipment is functional as well as the education of the workers on how an RCCD project is performed. Another reason to have a full-scale test is that the test will yield important data on:

- Aggregate mix
- Concrete mix
- Fresh concrete behavior
- Hardened concrete behavior
- In-situ behavior, such as compaction of the concrete and foundation

### 2.2.5.1 Diversion

The sensitivity of the cofferdam structure is not as high for an RCCD as for a CFRD. This means that less time and money can be spent on building smaller provisory cofferdams since RCCDs can be erected much faster than a CFRD. Thus the cofferdams only have to be designed for seasonal high flows and not more than annual high flows. Furthermore, the RCCD structure is not as sensitive to erosion as the CFRD, so overtopping of the cofferdams is not necessarily catastrophic. (US Army Corps of Engineers, 1995)

### 2.2.6 Risks and experiences

The risks of building dams with RCC technique are, nowadays, well documented. The experiences of these types of dams have increased and so has the know-how. We have chosen to look closer at some risks we feel could have a real impact on the average RCCD project. The common denominator for these risks are that they are all well documented and their behavior is well understood and described in the leading books of hydraulic structures.

The risk factors are described below:

- Internal uplift
- Sliding stability
- Internal stresses and loads
- Seismic loads
- Frost and ice

Other considerations when building RCCDs also have to be made, such as bedrock and foundation stability. However, those aspects are already described earlier in the report.

#### 2.2.6.1 Internal uplift

The internal uplift is caused by seepage through the concrete in the dam body. This pressure is due to water penetrating along the upstream face as well as water seeping through the foundation. This uplift pressure has caused several dam failures. The seepage through the dam causes not only uplift pressures, but also chemical erosion problems like hydratisation etc. (Renius, 1968)

To solve the problem of internal uplift in the dam body a drainage system can be built in the dam. Normally this is done by constructing a drainage gallery inside the dam which reduces the pressure downstream in the dam body. A drainage gallery in the foundation reduces the water pressure and stabilizes the dam (*figure 2.20*). This measure can also be used to improve the water permeability of the dam. (Renius, 1968)

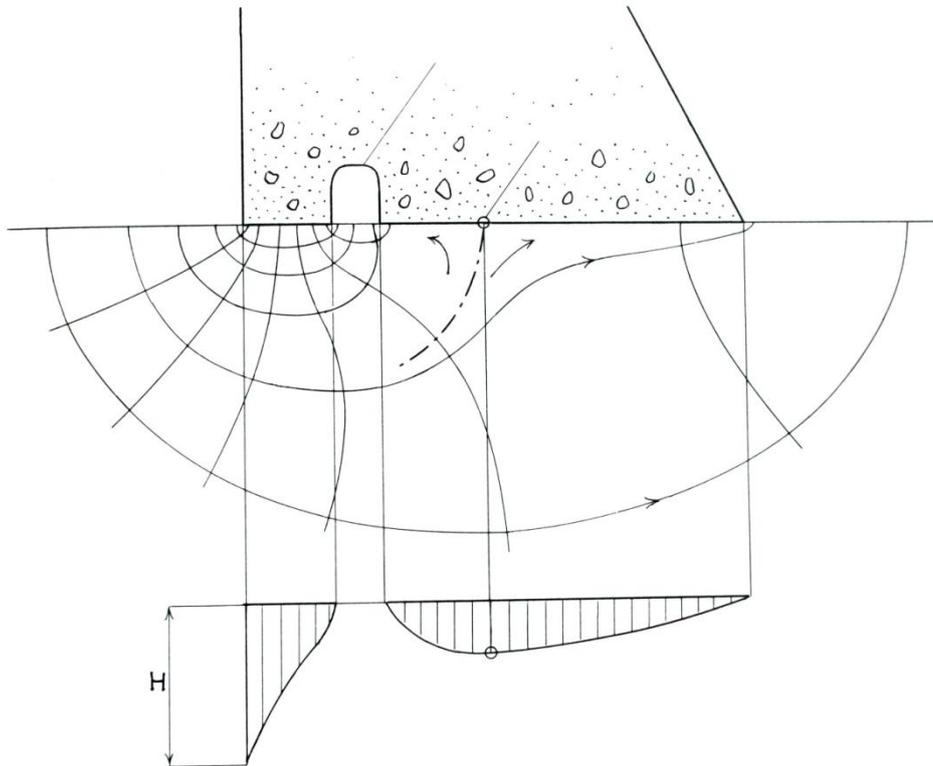


Figure 2.20 Drainage gallery in foundation

(Reinus, 1968)

### 2.2.6.2 Sliding stability

There are three ways the stability of a gravity dam could be jeopardized according to Tanêev (2005). These three includes sliding across the foundation or cracks in the foundation, overturning of the dam losing contact with the foundation and failure of the concrete in the dams body or failure of the rock mass in the foundation.

The only real risk of those three scenarios is, according to Tanêev (2005), the dam sliding across the foundation. One remediation of the risk is usually to form the foundation with several “teeth” to stabilize the joint created with the foundation (see figure 2.21). This however, is only really effective if the rock mass under the foundation is considered good enough.

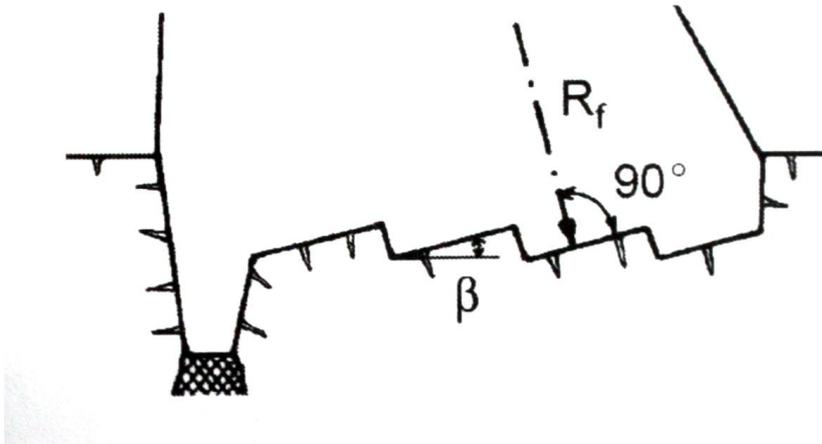


Figure 2.21 Teeth to prevent sliding

(Tançev, 2005)

Another way to eliminate this risk is to lower the uplift pressure under the dam. This can be done by having a grout curtain under the dam front which will decrease the pressure; if the grout curtain also is complemented with a drainage gallery the uplift pressure will be greatly reduced. (Häggström, 2011 and Tançev, 2005)

### 2.2.6.3 Internal stress and loads

The internal loads within the dam can be minimized by designing the profile of the RCCD, especially for large dams. It is important that the stress levels give no tension under any loading condition (full reservoir, empty reservoir etc.)

However, since multi-stage profiles are somewhat more expensive to build single-stage design is often used today. Single-stage means that the downstream slope is uniform while the multi-stage design consists of multiple angles of the slope, shown in *figure 2.22*. (Novak, et al. 2001)

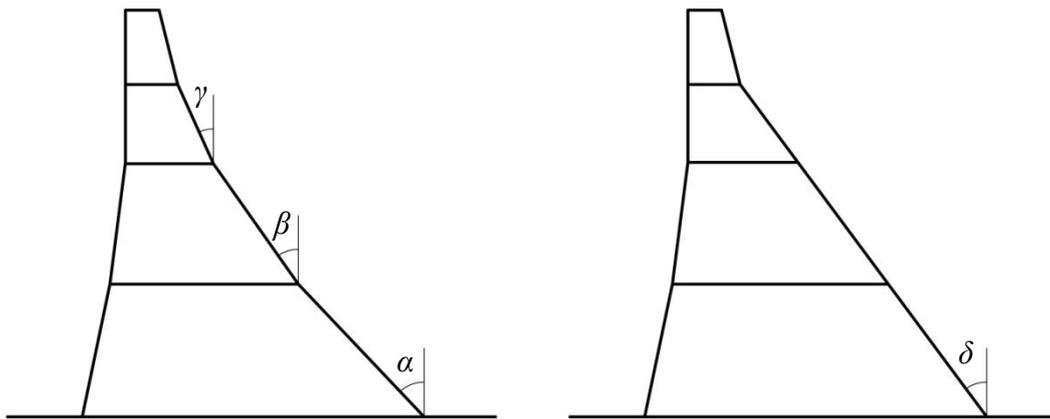


Figure 2.22 Downstream with multiple angles and with uniform slope.

#### 2.2.6.4 Seismic

Earthquakes and dynamic loads affect the RCCD type quite differently than the CFRD type. Since concrete dams are considered as compact elastic structures it is important that the dynamic loads does not give tension stresses that can induce cracking in the dam body. However, this risk is reduced in the design phase where such loadings are considered.

There is also the risk of structural resonance when the dam is being subjected to dynamic loads. Structural resonance is more likely to occur in high (large) dams rather in smaller dams. This is because the natural frequency of the dam is normally estimated as:

$$f_n = 600 \times \frac{T}{H^2} \quad (2.4)$$

(Novak, et al. 2001)

Where H is the height of the dam and T is the length of the dam base. This basically means that dams 20 meters high have a natural frequency of 15-25 Hz, depending on base thickness. Dams 50 meters high have a corresponding frequency of 6-9 Hz. Normally, major seismic shocks are in the range of 1-10 Hz. (Tançev, 2005)

Resonance in the entire structure is very unlikely but due to inertia forces created by the dynamic loading, local high-stress areas in the dam can produce cracking. These effects can be reduced by designing the cross-section of the dam in such ways that the vulnerable areas are satisfactory. (Novak, et al. 2001)

#### **2.2.6.5 Frost and ice**

When it comes to RCCDs, the frost can have some effect on the uplift pressure. If frost is being allowed on the bedrock downstream of the dam, or on the face of the downstream side, the uplift pressure will become greater thus increasing the risk of sliding. However, the risk of overturning the RCCD will remain at a constant low level. (Reinius, 1968)

### 3 Case Studies

When planning a dam project there is an infinite amount of choices to make. The most urgent choices may be where to place the dam and what kind of dam construction should be used. In this report the two different dam constructions CFRD and RCCD are compared and evaluated. This is done on the basis of four case studies:

- The Rio Esti Hydropower Project (Rio Esti) in the Chiriquí province is the oldest project of the case studies and was finished already in 2003. The entire hydropower project consists of two dams where the Barrigon dam, evaluated in this report, is constructed as a CFRD.
- Changuinola I in the Bocas del Toro province is the first part of the Changuinola hydropower project. The dam is under construction when this report is written (2010). Changuinola I was first planned as a CFRD (see the study of Chan-75 below.) but plans were changed in an early stage and the dam is now built with RCC-technique instead.
- Chan-75 is used as a case study though it was never built. The dam was planned as a CFRD, slightly lower than Changuinola I but higher than Rio Esti. Chan-75 was replaced with the Changuinola I project.
- Changuinola II is the second part of the Changuinola hydropower project and it is under investigation (2010). Changuinola II is planned to be built as a RCCD. It is very likely that the investments in Changuinola I can be useful in this project, making an RCCD design more favorable than a CFRD. Though, the aim of this evaluation is to give an overall and objective description of the two dam types and therefore those advantages are excluded in the first run.

The four cases appear in text in the same order as listed above. The description is based on the same aspects as the dam type description in *Chapter 2*. The aim of the project is not to do any economical study of which alternative is preferable. It is impossible to ignore that money is the main driving force when the choices are made.

This comparison between the case studies is used for evaluation of the Changuinola II project since the case study for this dam is slightly different due to the fact that Changuinola II is yet to be built.



*Figure 3.1 Overview of Panama*

The two case studies have the same layout in order to simplify the comparison. Below follows a short description of the properties described for each case.

- **Site**  
This part describes the conditions for the different sites, which includes for example geology, topography, flows and climate.
- **Design**  
This part aims to give a summary of the chosen dam design, CFRD or RCCD and how the spillway is designed.
- **Construction**  
This part will describe the construction procedure.
- **Challenges and Experiences**  
This part describes what challenges that have appeared during design and construction. It will also describe what experiences can be helpful in the future.

### **3.1 Rio Esti**

The Rio Esti hydroelectric power project was completed during 2003. It is located 400 km west of Panama City in the province of Chiriquí. The plant partially regulates the daily flow during dry season and it is projected to have a total installed capacity of 120 MW. Besides canals, tunnels and powerhouses Rio Esti consists of two dams, Chiriquí Dam and Barrigon Dam. (Swedpower, 2001)

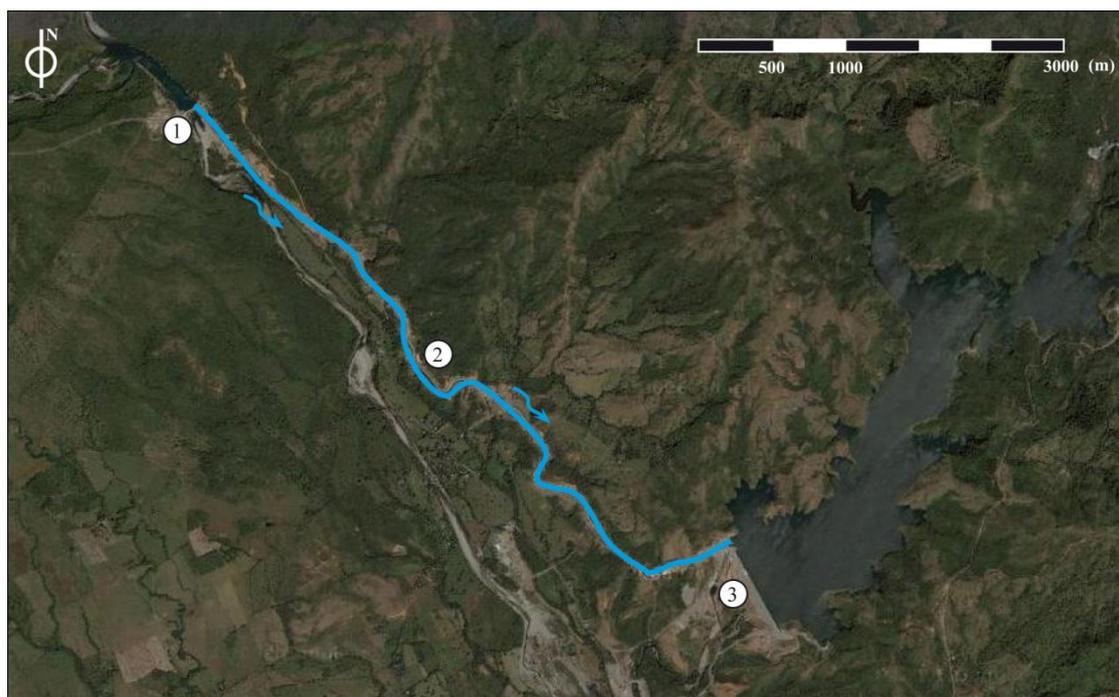


Figure 3.2 Overview of Rio Esti Hydroelectric power project

1. Chiriquí dam
2. The canal
3. Barrigon dam

The Chiriquí Dam is a 213 meters long concrete dam located in the Chiriquí River. It is 35.5 meters high and has an ungated spillway with a discharge capacity of 3,390 m<sup>3</sup>/s. The Barrigon dam, which is the one of interest for this project, is a 60 meters high CFRD which has a chute spillway with a discharge capacity of 1077 m<sup>3</sup>/s. (Swedpower, 2001)

The main part of the inflow to the Barrigon reservoir is water conveyed from Chiriquí through a canal. The canal is a 6.1 km long concrete channel with intake in the Chiriquí reservoir and outlet in the Barrigon reservoir. The position of the canal is shown in *figure 3.2* and *figure 3.3*. (Swedpower, 2001)

The Barrigon Dam is a storage reservoir and has a total volume of about 47 million m<sup>3</sup>. The operation elevation is between +218.0 and +222.0 m.a.s.l and gives an active volume of 10 million m<sup>3</sup>. (Swedpower, 2001)

### 3.1.1 Site description

The dam site is mainly formed of volcanic sedimentary rocks and some more recent fluvial deposits. The deposits, which can be found on the right side of the valley, can be up to 35 meters deep according to the performed geological survey. The survey also concludes that the bedrock can be considered to be of good quality. (AES Panama, 2001:1)

### 3.1.2 Design

The design for Barrigon Dam is distinct described in “*Estí Hydroelectric Project: Owner’s technical requirements, part B - Design requirements and criteria*” (AES Panama, 2001:1) and this part will contain the most important features and assumptions. First it is a description of the criteria determining what properties are used for design. Followed by summary of the design details used for Barrigon Dam.



Figure 3.3 Overview of the Barrigon dam area

1. The canal
2. Spillway chute
3. Bottom outlet
4. Dam body
5. Headrace tunnel

#### 3.1.2.1 Design criteria

The criteria used for design are partly dependent on the special features of this dam site, described in *chapter 3.1.1*, and partly dependent on international criteria and standards for this certain dam type. Requirements and criteria are stated in “*Owner’s Technical requirements, Part B – Design Requirements and Criteria*” (Swedpower, 2001). Some of the essential design features are summarized below.

**Dam crest elevation** – To prevent overtopping it is important to choose a correct height of freeboard. At the Estí project this is reached by using two criteria:

1. The elevation during the 10,000-year flood with an additional minimum of 0.5 m.

2. Maximal normal operating elevation plus wave run-up and an additional minimum of 0.5 m.

**Seepage** – The maximum seepage loss for Barrigon dam is 200 liters/ second.

**Spillway** – Spillways are designed with capacity to discharge the 10,000-year flood.

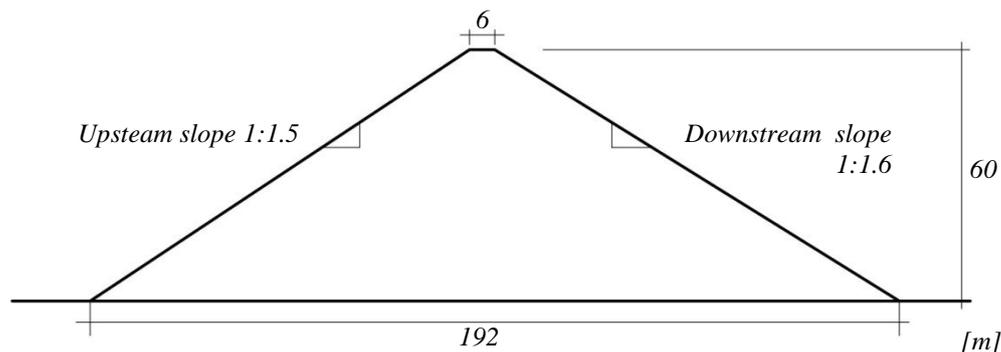
### 3.1.2.2 Dam body design

As mentioned above Barrigon Dam is built as a Concrete Faced Gravel Dam / Rock-fill Dam with a crest length of 675 meters. *Figure 3.4* shows a cross section of the dam construction with numbers referring to the text below. (Swedpower, 2001)

1. The height of the dam is 60 meters with a freeboard of 2.75 meters.
2. Upstream slope is 1:1.5.
3. As face a 300 mm reinforced concrete face is used.
4. Downstream slope is set to 1:1.6.
5. The crest width is 6 meters
6. The dam body is split into several zones of different granular materials.

### 3.1.2.3 Spillway design

An important detail not shown in *figure 3.4* is the spillway solution. The spillway is designed with three radial gates to control the floods. All gates are 6 meters wide, 10.5 meters high and have a still level at +211.0 m.a.s.l. The gates starts to operate when reservoir level reaches +222.0 meters, maximal operation level, and then operate in stages until they are fully open when reservoir rises to +222.5 m.a.s.l. (AES Panama, 2001:1)



*Figure 3.4* Cross section of the Barrigon dam

During construction the river will be diverted through the bottom outlet culvert, later used for emergency drawdown. The culvert is dimensioned to control the 1 in 50 year flood flow ( $464 \text{ m}^3/\text{s}$ ). (AES Panama, 2001:1)

### 3.1.3 Construction

The construction of the Barrigon CFRD is executed in several steps, diversion culverts, excavation, concreting of plinth, reinforcement, fill of zones in different stages, face construction, parapet wall and finally the filling procedure. (AES Panama, 2001:2)

Diversion of the river is mandatory preparative work in order to proceed with dam excavation and concreting of the plinth. An excavation for the diversion culvert is done along the proposed alignment. When a suitable rock foundation is confirmed, the excavation is widened enough to accommodate the diversion culverts. When excavation is done the culvert can be concreted. (AES Panama, 2001:2)

Before the river is diverted the excavation works are confined to the abutment areas. When the diversion culverts are in use and the flood plain is dewatered, excavation works can be extended to the whole foundation area. Concreting of the plinth at the upstream toe is performed and when the concrete has gained sufficient strength, grouting is started from the plinth surface. (AES Panama, 2001:2)

## 3.2 Changuinola I

Changuinola River lies in the far north-west of Panama in Bocas del Toro province. In 2003 the first water rights concession was issued to Hydroteribe S.A. Though, the company had not enough financial resources and in 2004 the water rights were sold to AES Changuinola S.A. AES is one of the largest power companies in the world and has a capacity to serve 100 million people worldwide. In Panama AES is the largest energy producer with a capacity of more than 480MW. (AES Changuinola, 2008)

The project head is between the elevation +320 and +55 m.a.s.l and the original plan was to split the range into three power stations, according to *figure 3.5*. Problems with the projects economical feasibility made it desirable to raise the reservoir level for Chan-75 to +165 m.a.s.l, which inundated the power station for Chan-140.

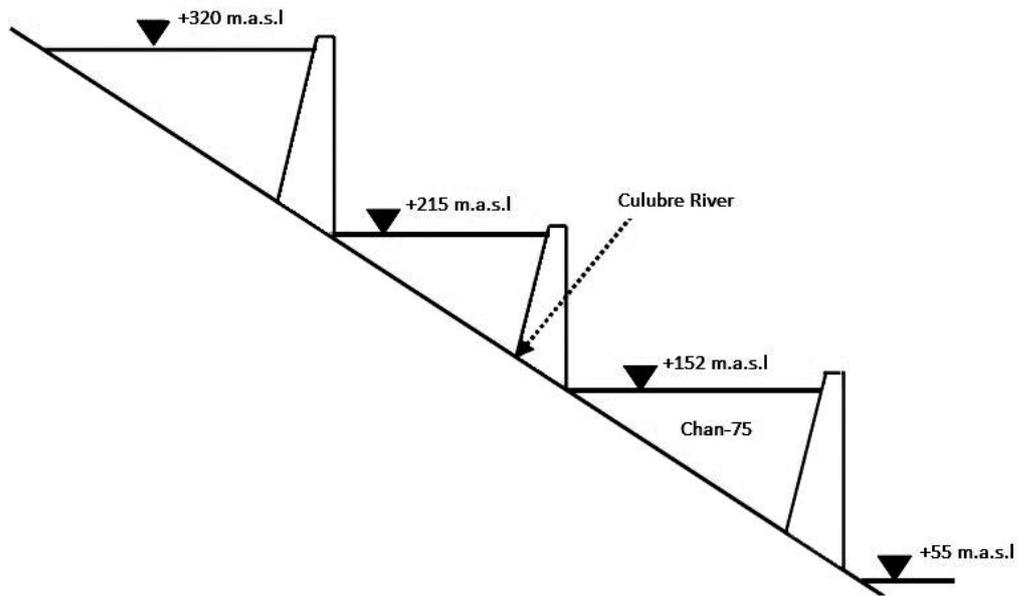


Figure 3.5 Original plans for Changuinola Hydropower project

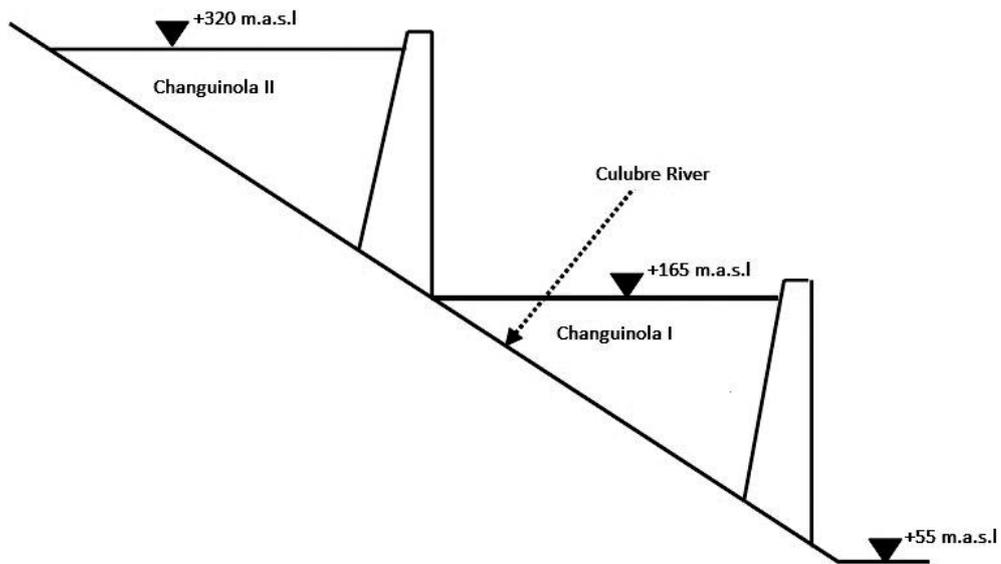


Figure 3.6 Final solution for Changuinola Hydropower project

The layout was now changed and the three projects were reduced to two large projects, Changuinola I and II (figure 3.6). Changuinola I has the same position as the former Chan-75 and Changuinola II is placed about two kilometers upstream the

confluence of Changuinola and Culubre rivers. (Changuinola Civil Works Joint Venture, 2009)

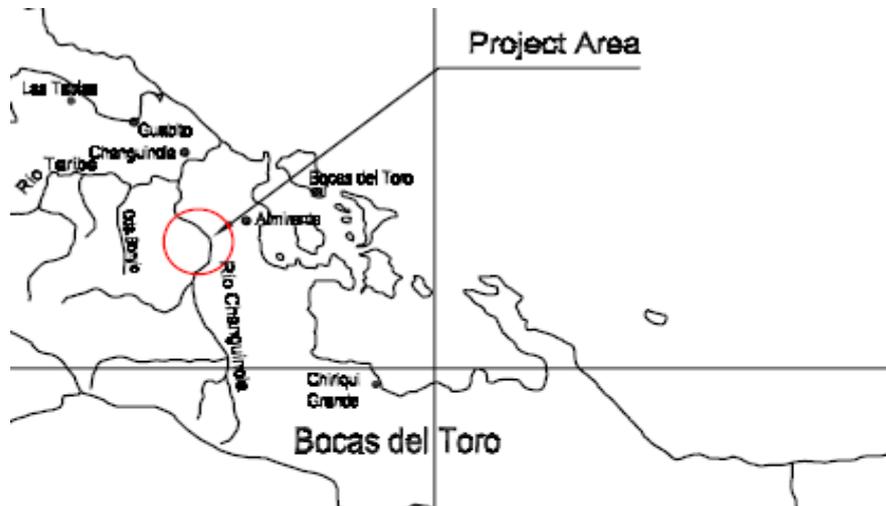


Figure 3.7 The Changuinola project area

The construction of Changuinola I started in October 2007 and the commercial operation is estimated to start early 2011. To a large extent Changuinola I is the same project as the original Chan-75. The main differences are stated below:

1. The reservoir elevation is raised from +152 to +165 m.a.s.l.
2. The dam type is changed from a CFRD to an RCCD.

According to AES Changuinola the project will have an annual average generation of almost 1,050 GWh and the approximate investment will be 563 million dollars. (AES Changuinola, 2008)

As mentioned above the first plan was to build a CFRD but after some reconsideration it was decided to build a RCCD instead. According to Frostberg (2010) the decision was made due to the following four arguments:

1. Economy
2. Construction time
3. Problems with space for spillway and diversion for CFRD
4. Expensive diversion tunnels under the dam for CFRD

### 3.2.1 Site description

The area at Changuinola River is characterized by very steep slopes and a deep river valley. The geology and geotechnical properties of the area are, however, poorly studied which means that the case study will lack certain valuable information regarding said properties. (Monaghan, 2007)

### 3.2.2 Design

Figure 8 shows an overview of the Changuinola I dam construction.

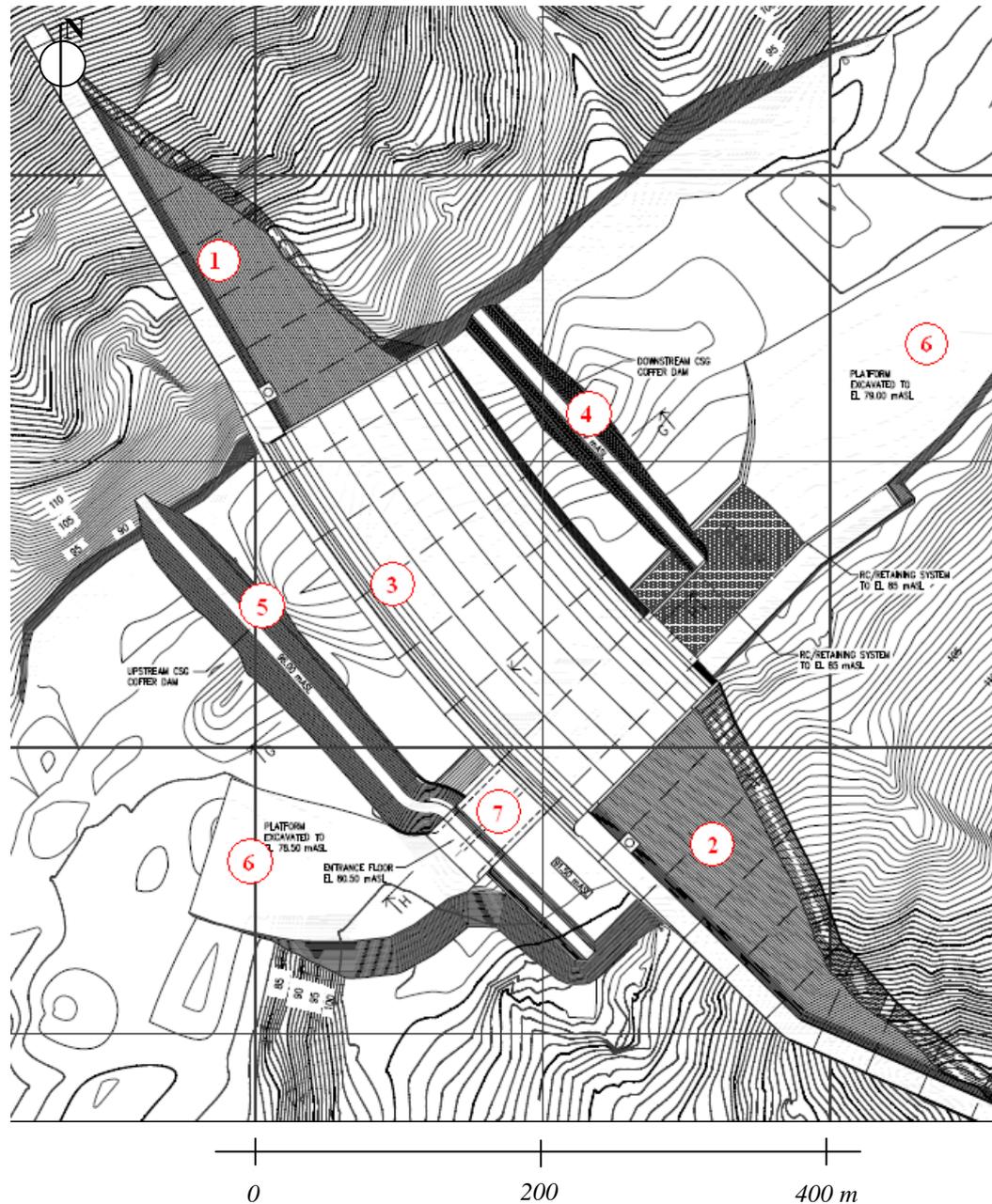


Figure 3.8 Plan layout for Changuinola I during construction.

1. Left abutment
2. Right abutment
3. Dam body with uncontrolled ogee spillway
4. Downstream cofferdam
5. Upstream cofferdam
6. Diversion inlet and outlet
7. Diversion channel

(Changuinola Civil Works Joint Venture, 2009)

### **3.2.3 Construction**

#### **3.2.3.1 Material**

The total volume of concrete needed for Changuinola I is approximately 900,000 m<sup>3</sup> (Changuinola Civil Works Joint Venture, 2009). Therefore, the access to cement, fly ash and aggregate is essential during construction. The concrete is produced in a batching and mixing plant located just downstream the construction site. From there the concrete is delivered to the dam wall by a conveyor belt.

Besides the importance of efficient production and transportation of the RCC, the access of raw material is essential to an effective construction. Following chapter will describe the producing of RCCD at Changuinola I.

#### **3.2.3.2 RCCD**

The production of an RCCD can roughly be divided into four parts.

1. Raw material supply. Such as cement, aggregate, fly ash and water.
2. Batching and mixing
3. Transportation to construction site.
4. Construction, spreading and vibrating.

To achieve efficiency during the construction period it is essential to take all these parts in consideration and make them work smoothly.

It is preferable to have a nearby access to raw material as it is of such a great importance for the construction. It can be assumed that water supply won't be any problem at this site and will therefore not be treated here.

Cement and fly ash have to fulfill certain criteria and also have a steadiness in their quality. This is necessary in order to warrant an even and high quality of the concrete in an RCCD. At the Changuinola I project it was not possible to meet these demands in Panama and therefore both cement and fly ash were shipped from Florida, United States. (Frostberg, 2010)

According to Frostberg (2010) the choice of shipping material from Florida lead to significant problems with logistics. The cement and the fly ash were shipped to the harbor in Amirante. Those continuous transportations were made on a ship specially modified for this purpose. Cement and fly ash were pumped into two silos and from there loaded on trucks for transportation to the batching plant at Changuinola.

### **3.2.4 Challenges and experiences**

The challenges for the Changuinola I project are similar to the standard dam construction problematics; materials and location.

The construction of a RCCD at Changuinola I have not been totally fuss-free. According to Gunnar Frostberg problems have occur as an affect of underestimations of the complexity of the problems and too optimistic approach concerning the time aspect. Frostberg (2010) is pointing at:

- Access to cement and fly ash of sufficient quality. In the case for Changuinola I this has been bought from Florida, US, leading to extended logistic problems compared to a local solution in Panama.
- Numerous tests are required in order to get a proper aggregate mix. Plants for aggregate crushing has to be built if they don not already exist.
- Numerous tests have been done in order to find a proper formula for the RCC.
- It has also been an extensive work finding rock that satisfies the necessary criteria.
- It is not possible to cast RCC when it rains more than 0.2 – 0.5 mm per hour, which can occur in this region.
- Capacity for batching plant and conveyor belts also limits the construction speed.

According to Frostberg (2010) it takes about 12 months to cast RCC for a dam with similar size as Changuinola I. But it is important to be well prepared for those parameters mentioned above.

### **3.3 Chan-75**

The Changuinola Hydropower Project has been re-projected several times and there have been a lot of different design alternatives. Because of the access to material about earlier plans and the near relation to Changuinola II, it will follow a summary of the plans for Changuinola I as a CFRD. These plans are most often referred to as Chan-75.

Chan-75 is the original plans for the Changuinola River. The dam was designed as a CFRD and the preliminary design was evaluated in 2005 and it is located at the same place as Changuinola I. This chapter is based on the background material from these plans and it is noteworthy that this dam has not been built. For further information about the project see *chapter 3.2*.

#### **3.3.1 Site description**

Site description is the same as for Changuinola I, *chapter 3.2.1*.

#### **3.3.2 Design**

As mentioned above Chan-75 is a stage in developing the Changuinola Hydropower Project. This chapter describes the first stages of the plans for building a CFRD. This is not a constructed project. The plans are therefore less detailed than for the other projects. However the plans are of interest because of the similarities with Changuinola II. *Figure 3.9* shows the final design proposal.

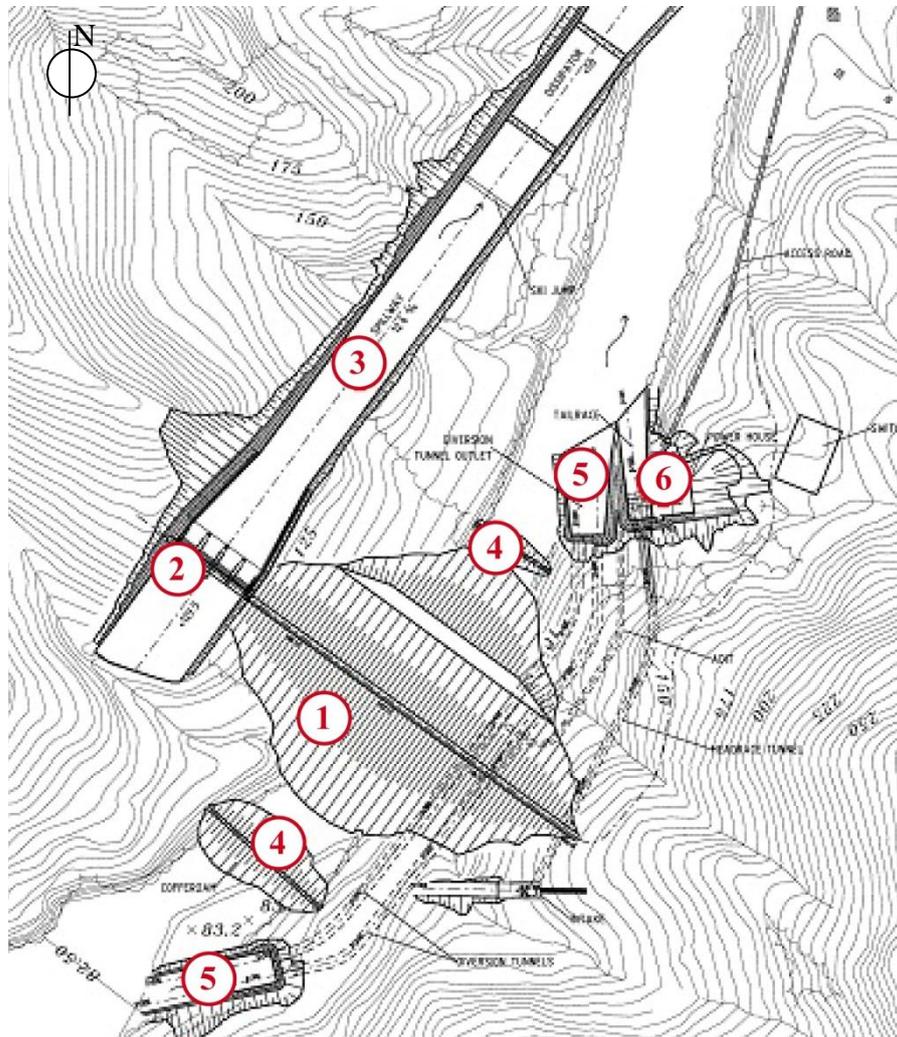


Figure 3.9 Plan layout for Chan-75 during construction.

1. CFRD body
2. Spillway gates
3. Spillway Chute
4. Cofferdams, for construction only
5. Diversion inlet/outlet, for construction only
6. Power house

(Gavilan group, 2005)

### 3.3.2.1 Dam body design

Figure 3.10 shows the dam body in profile.

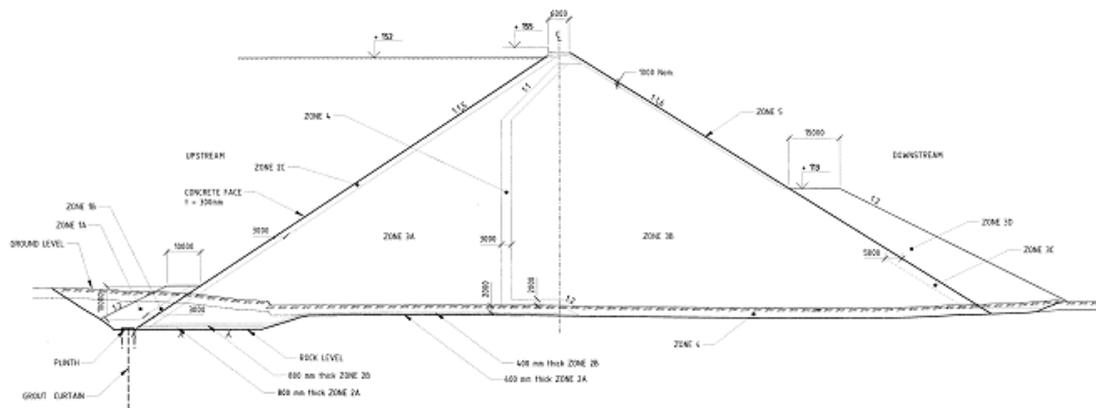


Figure 3.10 Profile view of Chan-75

(Gavilan group, 2005)

### 3.3.2.2 Spillway design

The spillway for Chan-75 was preliminary designed as shown in figure 3.11. It consists of four gated spillways, each 16 meters wide and 18.5 meters high. This design would give a possible discharge capacity of 10,000 m<sup>3</sup>/s.

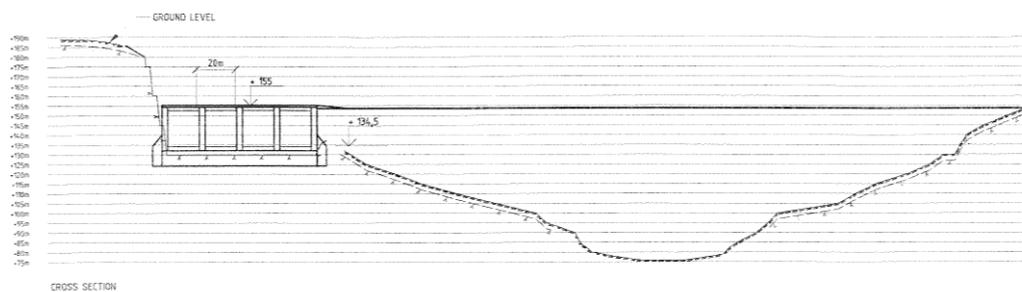


Figure 3.11 Spillway design for Chan-75

(Gavilan group, 2005)

## 4 Changuinola II

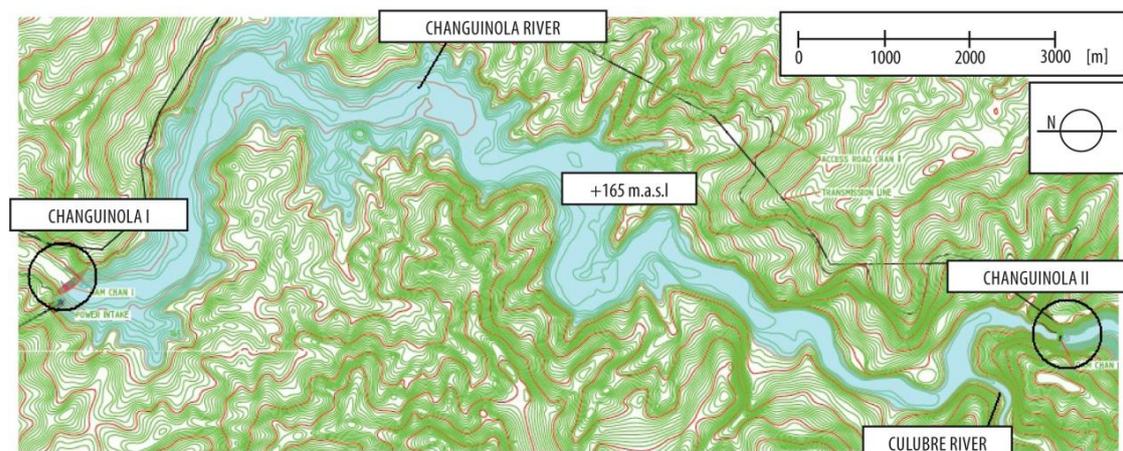
Changuinola II will be located a few km upstream the site for Changuinola I in the Changuinola River. Further location details are described in *chapter 3.2* and in *figure 4.1*. The head elevation for Changuinola I is +165 m.a.s.l and it is reached somewhere around two kilometers upstream the confluence of Changuinola and Culubre rivers.

In November 2007 a preliminary conceptual design was developed for how the remaining head between +320 and +165 m.a.s.l shall be used. “*The preliminary conceptual design*” (Frostberg, 2007) is developed by VPC with assistance of MD&A. This report is used as a base in this part, which will describe the alternatives developed and evaluated by VPC and MD&A. There will also be a description of a possible location and construction of a CFRD.

The “*The preliminary conceptual design*” (Frostberg, 2007) considers three different dam sites, A, B and C, and for all sites a RCCD is suggested.

In the report from 2007 (Frostberg, 2007) the biggest focus is on site C which according to VPC would be the optimal site for a RCCD. A further development of this assumption is done below in *chapter 4.1.1*.

As mentioned above this chapter will both describe a RCCD and CFRD alternative for the constructing of Changuinola II. The RCCD alternative, developed by VPC and MD&A, is described in *chapter 4.2*. The CFRD alternative, developed based on descriptions in this report, is developed in *chapter 4.3*.



*Figure 4.1* Overview of Changuinola River and the two dam sites, Changuinola I and Changuinola II.

(Frostberg, 2007)

The preliminary conceptual design does only consider the alternative to constructing a RCCD but according to Gunnar Frostberg (2010) it would be possible to build a CFRD at for example site B. In order to compare the CFRD with RCCD it is necessary to create a CFRD option. This chapter both gives a summary of the RCCD alternatives in the Preliminary Conceptual Design and compares those with possible CFRD alternatives.

## 4.1 Site description

Beneath three different site alternatives are described, A, B and C. The alternatives are developed by VPC in assistance with MD&A and they are described in “*The preliminary conceptual design*” (Frostberg, 2007) and the following text is based on this report. All three sites are located upstream of the confluence between Changuinola and Culubre rivers, within a span of 1 km, and *figure 4.1* shows the sites intermutual location.

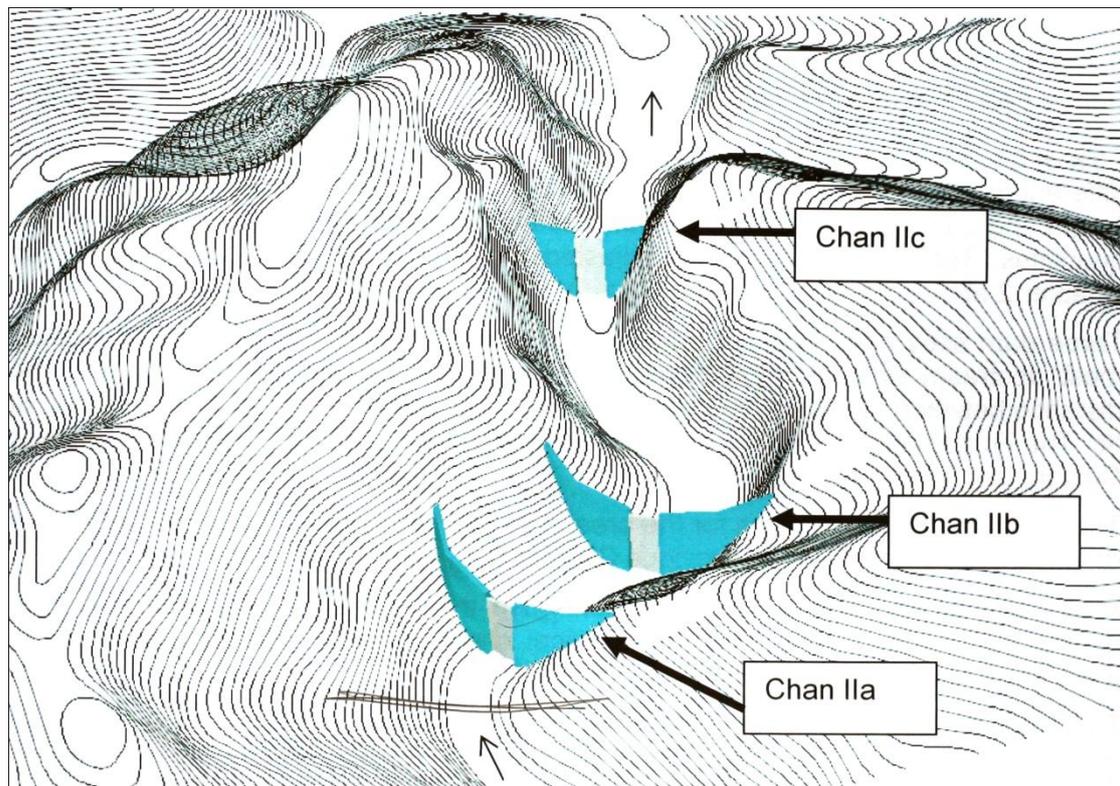


Figure 4.2 Locations for dam sites A, B and C

(Changuinola Civil Works Joint Venture, 2007)

As mentioned above the emphasis for the conceptual design (Frostberg, 2007) is in site C, which is considered to be the most favorable site for a RCCD. In the report this choice is based on the amount of concrete needed for the construction, shown in *table 4.1*. Assumptions and calculations for this decision are done on basis of satellite maps. The obvious focus on site C in the “*The preliminary conceptual design*” (Frostberg, 2007) will also reflect on this chapter.

Table 2.4 Concrete amount for the dam sites

(Frostberg, 2007)

Dam site	RCC [Mm <sup>3</sup> ]	GEVR [m <sup>3</sup> ]	Convotional concrete, [m <sup>3</sup> ]
Site A	1.46	67,760	22,810
Site B	1.84	79,220	22,810
Site C	1.15	47,620	22,810

The considerable smaller amount of concrete needed and the narrow floodway for site C makes it more favorable for an RCCD. But according to Gunnar Frostberg (2007) it would be possible to build a CFRD at site A which is not as narrow and have not as steep embankments as site C. Therefore there is more room for diversion tunnels and spillway.

- Site A, the most upstream site, located about 2 km upstream of the confluence with Culubre River. (*figure 4.2*)
- Site B, located around 1600 meters from the confluence between Changuinola River and Culubre River. As shown on the map (*figure 4.2*) the site is located just upstream of a bend on the river.
- Site C, the most downstream site located about 1 km upstream of the confluence with Culubre River. As mentioned above this is the most favorable site for constructing an RCCD.

#### 4.1.1 Site C

According to Frostberg (2007), the valley slopes on both sides are very steep, typically 65 degrees over 200 meters, which indicates very good rock mass quality. RMR was estimated to around 73 while RQD was thought to be 24. Site C is also thought to be particularly in favor for an arch dam type.

At the time for this report, no thorough geotechnical survey has been done which leaves an uncertainty regarding some of the dam features such as the spillway construction etc. This will have to be resolved before initiating the final design stage.

## 4.2 RCCD alternative

The plans in “*The preliminary conceptual design*” (Frostberg, 2007) are to construct an RCCD at site C. This construction will be described in the following text

## 4.2.1 Design

The dam body design is proposed as an arch dam due to the site conditions with steep slopes and relatively narrow valley.

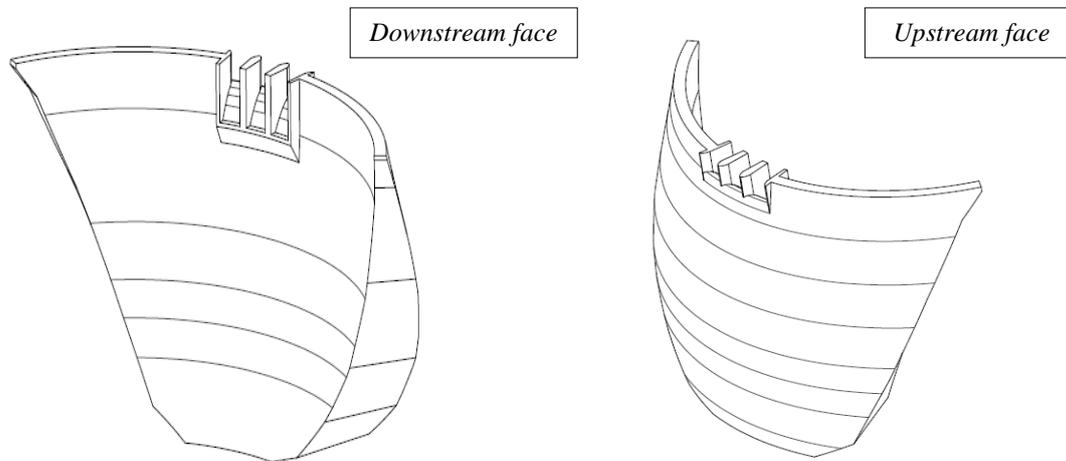


Figure 4.3 Preliminary dam body design for Changuinola II. Downstream face to the left and upstream face to the right.

(Changuinola Civil Works Joint Venture, 2007)

### 4.2.1.1 Dam body

As previously stated, the report also suggests that a volume of about 1.1 million cubic meters of RCC is needed to build the dam at site C. The concrete will be distributed in the preliminary design as in figure 4.4.

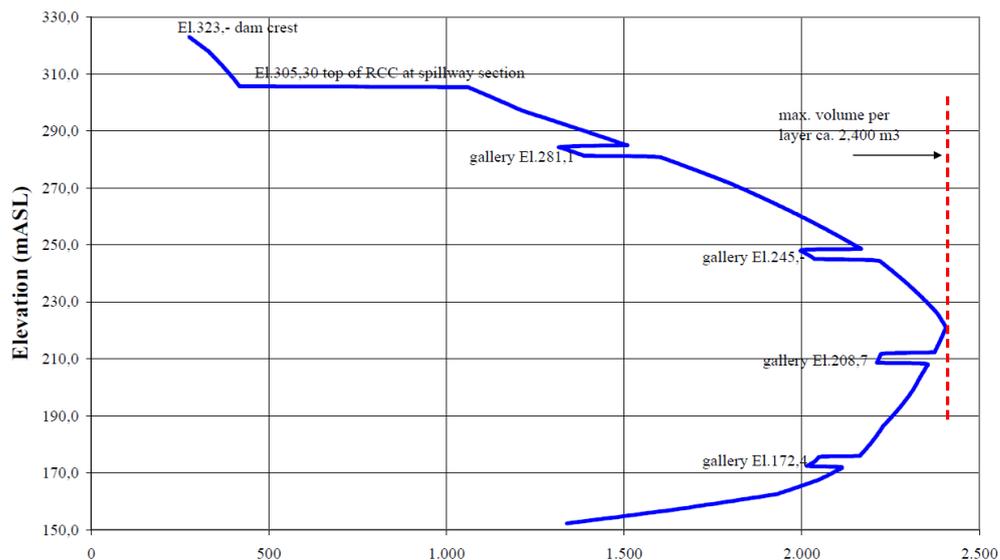


Figure 4.4 Volume of the 300 mm thick RCC layers derived from the preliminary design of the arch dam at Changuinola 2, Site C, for a total volume of 1.1 Mm<sup>3</sup>.

(Changuinola Civil Works Joint Venture, 2007)

#### 4.2.1.2 Sealing

GEVR (Grout Enriched Vibratable RCC) will be used for sealing the interface between the rock abutments and the dam body. It will also be cast against the formwork. All this together will ensure a good bonding with the rock and an excellent surface finish for the dam face. (Frostberg, 2007)

#### 4.2.1.3 Spillway design

The preliminary report presents the 10,000 year flood at  $6,554 \text{ m}^3/\text{s}$  and the proposed highest water level as 323 meters above sea level. To handle these design criteria a type of splitter spillway is recommended by the preliminary report, which is shown in *figure 4.5*.



*Figure 4.5* Recommended spillway design. Example of splitter spillway at Katse dam in Lesotho.

(Geolocation.ws, 2006)

## 4.2.2 Construction

### 4.2.2.1 Diversion

As diversion solution the preliminary conceptual design describes two solutions.

- Diversion culverts along left or right abutment. (Preferred option)
- Diversion tunnels in the left abutment. (Second option)

For the RCCD alternative the preferred option is used and therefore the only option described in this chapter. The second option is assumed to be used for a CFRD and therefore described in *chapter 4.2.1*.

The preferred option entails two reinforced concrete culverts, 9 times 9 meters, and two cofferdams, one 20 meters high upstream and one 6 meters high downstream. This corresponds to at least a four year recurrence interval flood and has clear similarities with the diversion solution for Changuinola I. It is also noted in the Preliminary conceptual design that it would be possible to enhance the diversion capacity with higher cofferdams. (Frostberg, 2007)

### 4.2.3 Material

It is important to secure the availability of material of adequate standard. This is for all components in the concrete mixtures, e.g. aggregate, cement, water and fly ash.

As aggregate supply the Frostberg (2007) suggest two solutions. The first alternative is to extract the aggregate needed from alluvial deposits in the Changuinola River, between the Changuinola I and Changuinola II. The second alternative is to investigate a limestone quarry on the right abutment above the area.

Regarding cement and fly ash it is not possible to find a solution in “*The preliminary conceptual design*” (Frostberg, 2007). However, according to Gunnar Frostberg (2010) the solution for Changuinola I was used because of the absence of adequate material in the local area. Therefore it seems likely to assume that the same solution will be used for Changuinola II, which imply transportation by ship from Florida to Almirante harbor and further on by trucks to plants at dam site.

“*The preliminary conceptual design*” (Frostberg, 2007) also have conducted a pre-study of how the entire dam site, with all installations, concrete and aggregate production areas, as well as access roads might look like. Consideration has been taken to where deposits of aggregates actually exist where the location of access roads and concrete production areas is more of a logistical problem. *Figure 4.6* show an overview of the project area.

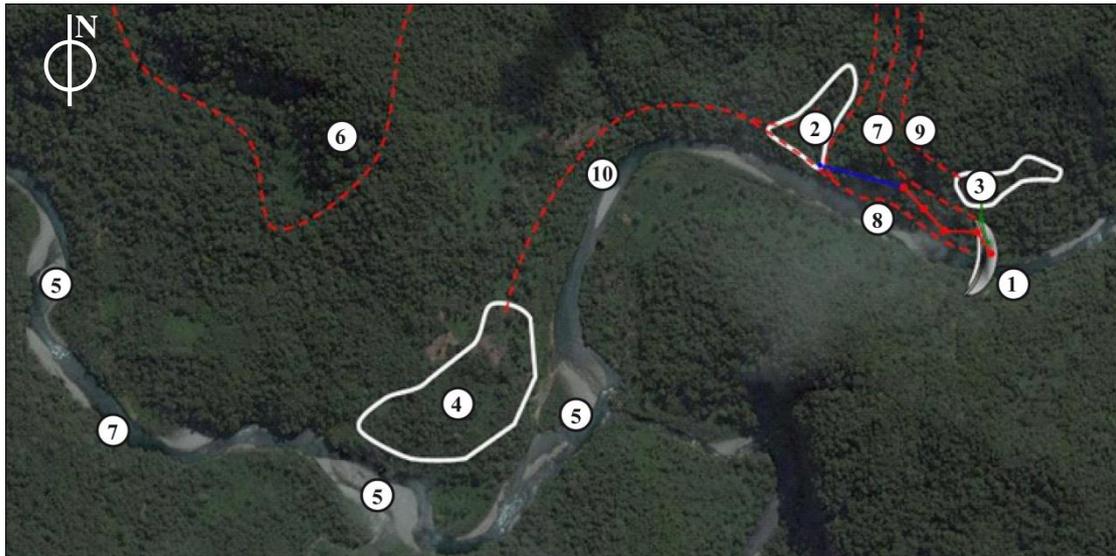


Figure 4.6

1. Dam site
2. Concrete production area (option 1)
3. Concrete production area (option 2)
4. Aggregates production area
5. Alluvium deposit
- 6-10. Access roads

(Changuinola Civil Works Joint Venture, 2007)

## 4.3 CFRD alternative

Because the preliminary design only consists of RCCD alternatives it is necessary to construct a CFRD alternative in order to make the comparison possible. In this part previous chapters are used as base for the development of a CFRD alternative for Changuinola II.

### 4.3.1 Dam design

It is likely to assume that the dam design will have a lot of similarities with the design for Rio Esti and the planned CFRD alternative for Changuinola I (Chan-75), see the CFRD section in figure XX. Therefore these two projects can be used as model for the design in of a CFRD alternative for Changuinola II. It is important that the design fits the certain properties at the Changuinola II dam site.

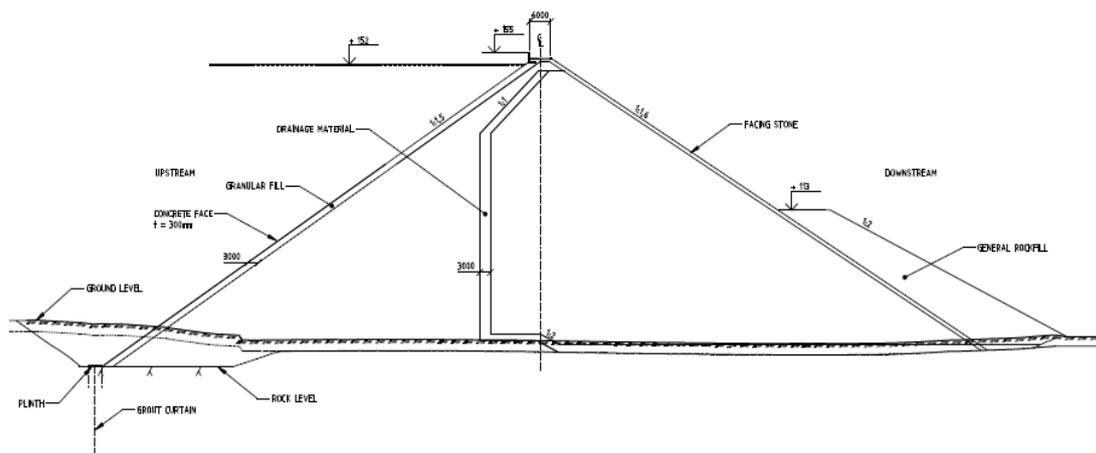


Figure 4.7 Section for Chan-75

(Gavilan group, 2005)

The most important difference when designing a CFRD for Changuinola II is the height. The RCCD planned are distinctly higher than the CFRDs described in previous chapter and this will obviously influence the design. Improvements are done in accordance to *chapter 1*.

### 4.3.2 Dam body

The geometry of the dam body can be split into five parts, whereof four are shown in section (*figure 4.7*).

1. Height
2. Upstream slope
3. Downstream slope
4. Crest width

And the fifth geometrical property is the crest length. If these properties can be determined it will be possible to calculate the amount of rock-fill needed. Below follows a description of how the determination is done.

The height is depending on head elevation chosen for the project. For Changuinola II the head elevation is investigated in "*The preliminary conceptual design*" (Frostberg, 2007) and set to +320 m.a.s.l. In combination with head elevation for Changuinola I, +165 m.a.s.l, a dam height of at least 155 meters are needed.

Upstream and downstream slopes are chosen in accordance to *chapter 2*. Natural gravel fill results in less steep slopes instead of what could be applied if rock-fill were used. However, the access of alluvial material is hopefully very good and therefore it will not be a problem.

According to *chapter 2* the crest should have width between 6 and 8 meters. Due to size of the dam an 8 meter wide crest is used.

### 4.3.3 Sealing design

As described in *chapter 2.1.2* and also in the case studies, sealing is done by using a concrete face slab. Sealing to the foundation is done in combination with a concrete plinth. In *chapter 2.1.2*, thickness of the concrete slab is calculated as:

$$T_i = T_{\min} + X \times H_i \quad (4.1)$$

(Vncold, 2008)

Where  $T_{\min}$  is 0.3 meters in the regular case and increased to 0.4 meters for high dams.  $H_i$  is the distance from the top and  $T_i$  is the thickness at certain height. As described in *chapter 2.1.2* the increased thickness is used to avoid ruptures in the upper part of the slab, due to compaction in the dam body. Using the given dam height of 155 meters a horizontal joint should be placed approximately 50 meters from top for the central slabs. *Equation 4.2* gives the thickness at the top of the lower stage slab.

$$T_{50} = 0.30 + 0.0035 \times 50 = 0.475 \quad (4.2)$$

(Vncold, 2008)

Next step is to correlate the thickness at the top of the lower stage with bottom of the upper stage.

$$T_{50} = 0.40 + X \times 50 \quad (4.3)$$

(Vncold, 2008)

$$X = \frac{T_{50} - 0.40}{50} = 0.0015 \quad (4.4)$$

(Vncold, 2008)

Gives the following thickness for the upper stage ( $H_i < 50m$ ):

$$T_i = 0.40 + 0.0015 \times H_i \quad (4.5)$$

(Vncold, 2008)

And for the lower stage:

$$T_i = 0.30 + 0.0035 \times H_i \quad (4.6)$$

(Vncold, 2008)

The increased thickness are only used for the central slabs were the stresses are highest. For lower slabs closer to the abutments, *equation 4.1* is used for the whole height.

#### 4.3.4 Spillway design

What spillway design to use depends on the design flood. The RCCD alternative for Changuinola II are dimensioned to control the 10,000 year flood, which corresponds to 6,554 m<sup>3</sup>/s. Comparing to Rio Esti and Chan-75, it is likely to assume that the 10,000-year flood is enough also for the design of a CFRD alternative.

Spillway for Chan-75 was preliminary designed for a capacity of 10,000 m<sup>3</sup>/s, which will clearly be enough. *Figure 3.11* shows the spillway design for Chan-75. As shown it is four gated spillways, each 16 meters wide and 18.5 meters high.

#### 4.3.5 Diversion

For diversion design it is assumed that the second option in *chapter 3.7.2.1*, diversion tunnels in the left abutment, can be used. This alternative is originally designed for the RCCD alternative and described in the "*The preliminary conceptual design*" (Frostberg, 2007). However, it is in this report assumed that it is a suitable solution for diversion design when constructing a CFRD. Assumptions are done in comparison with diversion design for Rio Esti and Chan-75.

The diversion for the CFRD is designed with two circular tunnels in the left abutment, each with a diameter of 9.9 meters. The lengths of the tunnels are 413 and 366 meters.

Table 4.2 Lining options

(Frostberg, 2007)

Type of lining	Lining thickness [mm]	Cross-sectional area [m <sup>2</sup> ]	Velocity <sup>1</sup> [m/s]	Velocity <sup>2</sup> [m/s]
Shotcrete	150	72.4	12.4	16.2
Concrete	500	62.2	14.5	18.9

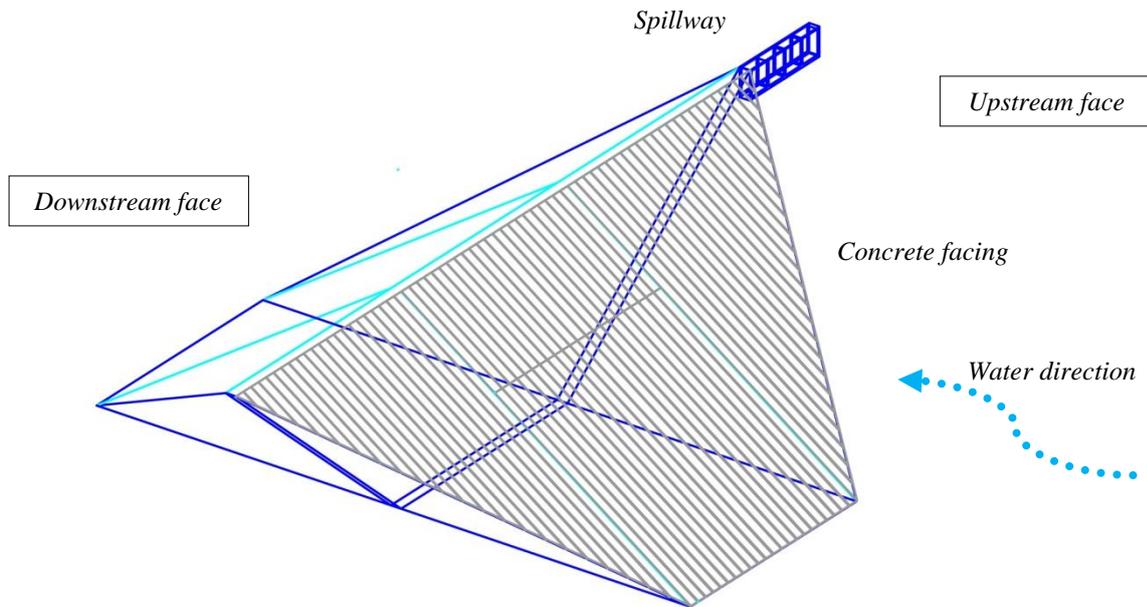
<sup>1</sup>) 4-year design flood (1728 m<sup>3</sup>/s)

<sup>2</sup>) 10-year design flood (2350 m<sup>3</sup>/s)

There are two alternatives of lining for the tunnels (*table 4.2*), 150 mm shotcrete or 500 mm concrete. Diversion should be designed for a four year recurrence interval flood (1798 m<sup>3</sup>/s), which corresponds to a velocity of about 12 m/s for the shotcrete lining and about 14.5 m/s for the concrete lining. The shotcrete velocity is on the limit for the shotcrete lining and at the same time the concrete velocity is well below limit. This makes it likely to assume that concrete lining should be used. As bonus the diversion solution would even manage the ten year recurrence interval flood of 2350 m<sup>3</sup>/s, which corresponds to a velocity of about 18.9 m/s.

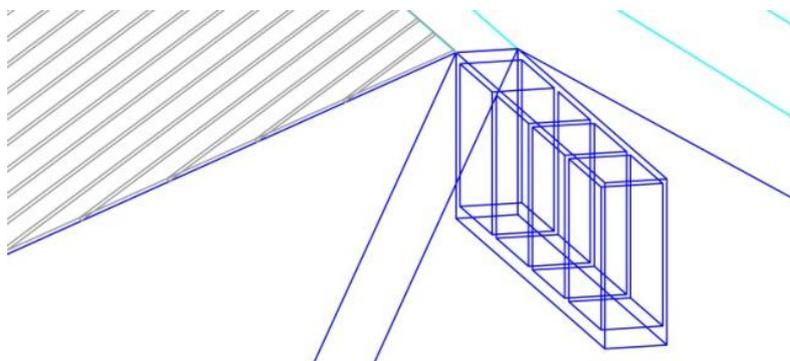
### 4.3.6 Final design

This part is a brief description of what a CFRD alternative could look like for Changuinola II. The figures made in AutoCAD are simplified models of a possible dam construction. *Figure 4.8* show perspective view of the CFRD with attached spillway construction. The total volume of the dam body is more than 11 million m<sup>3</sup>.



*Figure 4.8* Perspective of final design proposal

*Figure 4.9* shows the spillway solution. It is the same spillway design used for Chan-75 and it has a capacity of 10,000 m<sup>3</sup>/s. This is more than enough considering the design flow for Changuinola II is 6,554m<sup>3</sup>/s.



*Figure 4.9* Four gated spillway

Figure 4.10 shows upstream view with the concrete face.

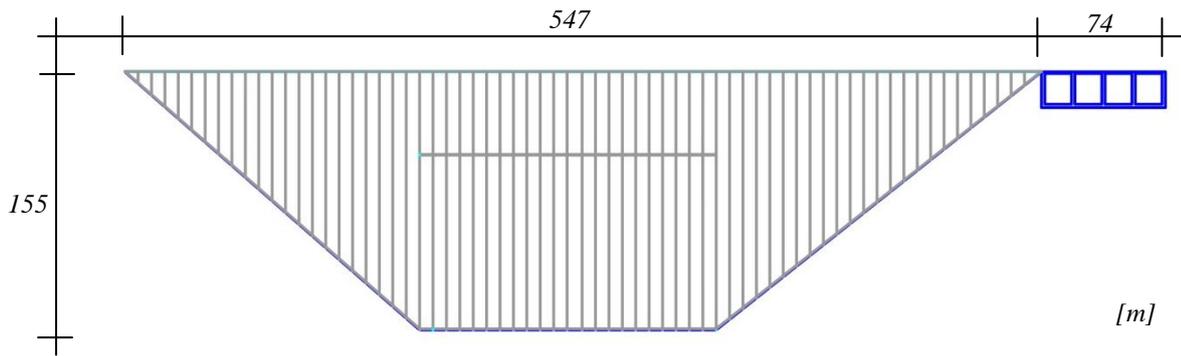


Figure 4.10 Upstream face

## 5 Conclusion and Discussion

In this chapter we will firstly present our conclusions from the project. We will then continue by a discussion regarding the conclusion and then finish the report by a brief discussion about problems encountered during the process of writing this report and some recommendations for further work.

### 5.1 Conclusion

- Changuinola II dam should be constructed as a RCCD due to reasons discussed in the discussion chapter.
- The choice of dam type is a very complex issue which is influenced by multiple variables such as topography, access of material, geological surroundings and economy.
- Dam features, such as spillways, outlets and power houses all depend on the dam type, economy and surrounding topography which means no general conclusion can be made regarding this.
- Both RCCD and CFRD are viable options when choosing a dam type.

### 5.2 Discussion

As we demonstrate in the conclusion part, we recommend Changuinola II being built as a RCCD. Constructing Changuinola II as a RCCD will make it possible to achieve some benefits from Changuinola I, e.g. as follows.

- The logistics for transport of concrete and fly ash with ship from Florida, through the port in Almirante, to Changuinola River are in a large extent already built.
- It is possible to re-use some of the crushing- and batching plants from Changuinola I.
- Trial tests for the concrete may not have to be done in the same extent.
- The knowledge about constructing RCCD is big, both concerning construction management but also among construction workers.

We believe these bullet points out-weigh the choice of a CFRD. If Changuinola I would not have been built and excluded from the case study, this project might have come to a different conclusion.

#### 5.2.1 Problems

In writing this report there has been some encountered issues.

- Material for the case studies have been rather difficult to assess, mostly regarding economic information but also time consumption for the construction and project planning stage as well as the amount of material used.
- The scope proved to be very extensive which resulted in cutting down certain features that were meant to be included.

### **5.2.2 Further work**

We propose that further studies regarding a cost study should be done, where key numbers for the different dam types are evaluated. Such key numbers could include, but are not limited to, man hour per m<sup>3</sup>, hours needed for project planning, cost per kWh.

Furthermore a study could be done evaluating concrete choices for the RCCD, since the material is a significant cost to the project as well as the possibility to place the concrete in desired pace and in a correct profile.

Also, a study concentrating on dam features and general recommendations for dam features could be done in order to help the project planning process and choices.

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