EVALUATION OF CONCRETE FACE ROCKFILL ALTERNATIVE FOR DAM TYPE SELECTION: A CASE STUDY ON GÖKÇELER DAM

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Approval of the thesis

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ABSTRACT

EVALUATION OF CONCRETE FACE ROCKFILL ALTERNATIVE FOR DAM TYPE SELECTION: A CASE STUDY ON GÖKÇELER DAM

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In this study a recent dam type, concrete face rockfill dam (CFRD), its design and behaviour is overviewed. The design features of Gökçeler Dam are introduced as a case study. Selection of concrete face rockfill type for Gökçeler Dam Project is discussed together with the other two alternatives, namely earth core rockfill (ECRD) and roller compacted concrete (RCC) dam. Gökçeler Dam type selection as concrete face rockfill dam is also verified by an economic analysis conducted calculating internal rate of return for all alternative types. In cost analysis a currency independent defined unit cost (DUC) is specified to verify the time independent validity of the economic analysis.

Keywords: Gökçeler Dam, Concrete Face Rockfill Dam, Defined Unit Cost, Cost Analysis

ÖZ

ÖN YÜZÜ BETON KAPLI KAYA DOLGU ALTERNATİFİNİN BARAJ TİP SEÇİMİ ÇALIŞMALARINDA DEĞERLENDİRİLMESİ: GÖKÇELER BARAJI ÖRNEK ÇALIŞMASI

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Yüksek Lisans, İnşaat Mühendisliği Bölümü
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Bu çalışma kapsamında yeni gelişen bir baraj tipi olan ön yüzü beton kaplı kaya dolgu baraj (ÖBKD) tipinin tasarım ve davranış özellikleri genel olarak ele alınmıştır. Örnek çalışma olarak Gökçeler Barajı'nın tasarım özellikleri tanıtılmıştır. Gökçeler Barajı Projesi için ön yüzü beton kaplı kaya dolgu tipinin seçilmesi, diğer iki alternatif olan kil çekirdekli kaya dolgu (KÇKD) ve silindirle sıkıştırılmış beton (SSB) dolgu tipleri de dikkate alınarak değerlendirilmiştir. Bütün alternatifler için iç kârlılık oranlarının da hesaplandığı bir ekonomik analiz çalışması gerçekleştirilerek, Gökçeler Barajı için ön yüzü beton kaplı kaya dolgu tipinin seçilmesi tahkik edilmiştir. Yapılan bu ekonomik analiz çalışmasının geçerliliğini zamandan bağımsız olarak koruyabilmesi için para birimlerinden bağımsız olan Tanımlanmış Birim Fiyat (TBF) belirlenmiştir.

Anahtar Kelimeler: Gökçeler Barajı, Ön Yüzü Beton Kaplı Kaya Dolgu Baraj, Tanımlanmış Birim Fiyat, Maliyet Analizi

To My Family,

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LIST OF ABBREVIATIONS AND SYMBOLS

A : Surface area of face slab
CFRD : Concrete face rockfill dam
DUC : Defined Unit Cost
ECRD : Earth core rockfill dam
g : Gravity acceleration
H : Maximum height of dam section
IRR : Internal rate of return
o : Opening normal to the joint
RCC : Roller compacted concrete
s : Settlement normal to the face slab
SF : Shape factor of the river valley
t : Shearing along joint direction
th : Thickness of face slab
α : Angle between face slab and AB plane of plinth

CHAPTER 1

INTRODUCTION

1.1 General

Concrete Face Rockfill Dam, CFRD, is a kind of embankment dam designed with an impervious element of concrete face slab constructed on the upstream face of underlying rockfill body in order to achieve watertightness (Kleiner, 2005-a).

Concrete Face Rockfill Dam, CFRD, originated from gold mining region of Sierra Nevada in California by 1850's. Gold miners developed the construction of dumped rockfill dams in order to provide the water required for cooling their drilling equipment. In the early times, these dumped rockfill dams had been waterproofed by wooden upstream facings which were by time switched with concrete facing (ICOLD, 1989-a).

Originating from wooden face dumped rockfill dam, in the recent 50 years CFRD became a frequently used type of dam by invent of vibratory roller compactors which was one of the drastic improvements of the construction technology. Initiative type selection studies of dam projects include concrete face rockfill dam alternative as the recent trend.

Dam projects are multifunctional phenomena and most of the projects are involved with power generation or irrigation for which cost-benefit analysis are vital for feasibility. Construction cost of the dam body constitutes the majority of overall project cost, thus selection of dam type adopted for the project is very important. For type selection, performance characteristics and construction requirements of dams must be studied in detail.

1.2 Scope of the Study

The main concern of the present study is introducing concrete face rockfill dams as an alternative for dam type selection studies by discussing design and performance characteristics of this type of dams and conducting economic analysis supported in the light of this information.

A case study of Gökçeler Dam is carried out in order to observe the difference among various dam type alternatives from many aspects, such as practibility, performance, economy and feasibility.

In Chapter 2, design characteristics of concrete rockfill type of dams developed upto now, are introduced. Since the design of this specific type of dam is mainly dependent on the experience of precedent, former design features, their improvement by time and the reasons forcing these improvements are investigated. Results of these improvements and recent design features are discussed.

In Chapter 3, performance case studies of significant precedent CFRDs leading very important design improvements are discussed. General performance tendency, deficiency of expected operation and construction behaviours are stated. Efficiency and cost of executed remedial measures are also discussed in this chapter.

Chapter 4 is reserved for case study. The design features of Antalya – Gökçeler Dam and general characteristics of dam site.

In Chapter 5, economic analysis are performed using currency independent defined unit cost (DUC). Principal construction works required to be performed on a dam site are selected and labeled by code numbers specific to Gökçeler Dam. Unit prices for each of these principal construction works are determined by analysing the costs of sub-stages. Hence basis for cost analysis is established.

Three alternative project formulations are taken into account and construction cost of each facility taking place within the formulations are calculated individually using formerly determined unit prices of specific construction works. Total investment cost of the alternatives are calculated considering the work schedule durations, expropriation costs, contingency cost and interest value of the cumulative costs. Benefits are assumed to be constant for all alternatives and internal rate of return (IRR) values are calculated in order to select the dam type.

Finally in Chapter 6, conclusions of the performed study are stated and recommendations for further studies are declared.

CHAPTER 2

CHARACTERISTICS OF CONCRETE FACE ROCKFILL DAMS

2.1 History

The first dumped rockfill dam with a concrete face slab, Chatworth Park in California constructed in 1895, was the first of the American CFRD series followed by 84 m high Dix River in Kentucky and 100 m high Salt Springs Dam which had been in service since 1931 in California (ICOLD, 1989-a).

The development period of rockfill can be divided into three main stages such as; early, transition and modern periods and CFRDs evolved from traditional design of early period to present design of modern period (Kleiner, 2005-b).

The early period started with the gold miners in 1895 and wooden or concrete face dumped rockfill dams were commonly constructed until 1940's. Operating dams were suffering significant leakage problems caused by the unbearable amount of joint movements resulting from high compressibility of dumped rockfill. During the construction and the first impoundment, dumped rockfill which was underlying and supporting the face slab, were compacted and settled under gravity and reservoir loading, guiding the face slab to deform in the same trend. Deforming grid of vertical and horizontal joints of face slab provided the leakage way (ICOLD, 1989-a). Even in some occasions, articulated structure of face slab could not tolarate the rockfill settlement and cracked yielding an increase for leakage way. Despite the fact that there had been no stability and safety problems in CFRDs suffering from leakage, this type of dams became unfeasible as a result of the remedial operation costs required to prevent unavoidable leakage.

A transition period started as the Earth Core Rockfill Dams, ECRDs, came into the scene by 1940’s. Highly compressible dumped rockfill was admitted to be more compatible to the earth core and its filters. The difficulties of impervious material supply increased the construction cost of ECRDs but the remedial activities required for excessive leakage on operation of CFRDs made the ECRDs first choice of engineers until the mid of 1950’s (ICOLD, 1989-a).

Since mid 1950’s by the invention of vibratory roller, compacted rockfill as a result of developing technology and improving construction techniques, modern period started and CFRDs came back as an alternative for most of the sites (Kleiner, 2005-b). Design of CFRDs is empirical and intensely based on precedent, thus keeping the inherent safety features of traditional design, there is an ongoing progress in design features and construction technology challenging design engineers for further developments to achieve successfully operating higher dams (Cooke, 2000-a).

2.2 Traditional Design Features of CFRD

Until the beginning of modern period in mid 1950s, CFRDs had been designed according to the traditional features which is illustrated in Figure 2.1. These traditional features consist of three main elements which are; dumped rockfill, thick and highly reinforced face slab, and the cut-off wall providing the connection between face slab and the rock foundation.

Each of these elements has some components and characteristics as explained below.

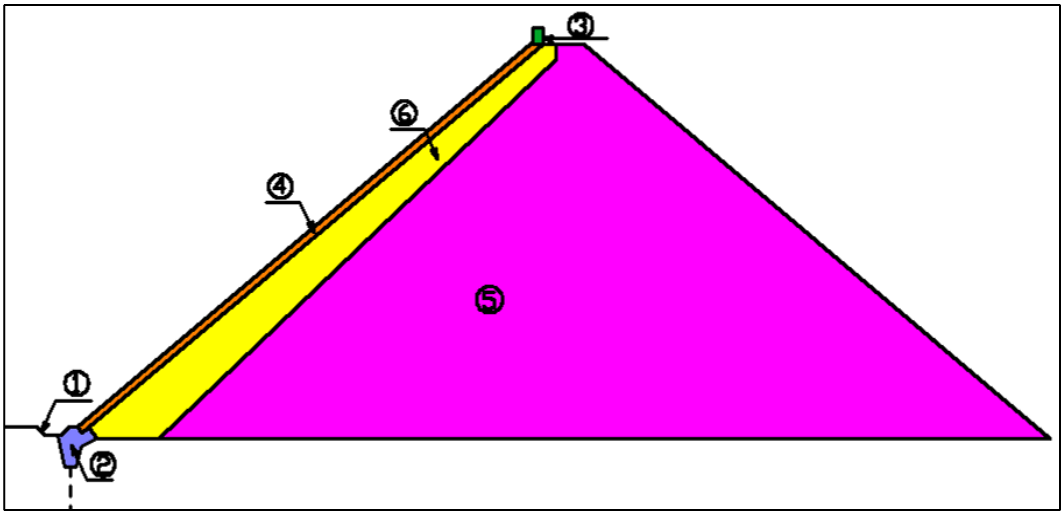

Figure 2.1 Features of traditional CFRD design (ICOLD, 1989-b)

(1) Cut-off Trench	(2) Grout Curtain (cut-off wall)
(3) Parapet Wall	(4) Face Slab
(5) Dumped Rockfill	(6) Manual Placed Large Rocks

1) The first feature is the cut-off wall constructed within the trench socketed into the bedrock along the upstream perimeter. The geometry of this wall is detailed conforming with the face slab thickness in order to provide appropriate contact interface (ICOLD, 1989-b).

2) Thick face slabs, starting with 30 cm at the crest and increasing by 20 cm per each 30 m of dam height (ICOLD, 1989-b).

3) Reinforcement ratios of face slab in both horizontal and vertical directions are 0.5 % of the slab thickness. The width of the slabs are 18 m in most of the cases, which is determined for constructional purposes (ICOLD, 1989-b).

4) Very dense grid of both vertical and horizontal joints in the face slab, and a hinge joint parallel to the perimeter joint. Waterstops and joint fillers of various types are used against compression to achieve articulated slab with a high degree of freedom (ICOLD, 1989-b). Traditional face slab of 40 m high Caritaya-Presa concrete face dumped rockfill dam in Chile which was constructed in 1935 is given in Figure 2.2.

Figure 2.2 Traditional face slab of Caritaya-Presa Dam in Chile (Quezada, 2007)

Details of horizontal and vertical joint characteristics for traditional design are given in Figure 2.3.

5) Adding to the camber, upstream surface is curved in one or two directions in order to reduce the openings of joints in reservoir loading (ICOLD, 1989-b).

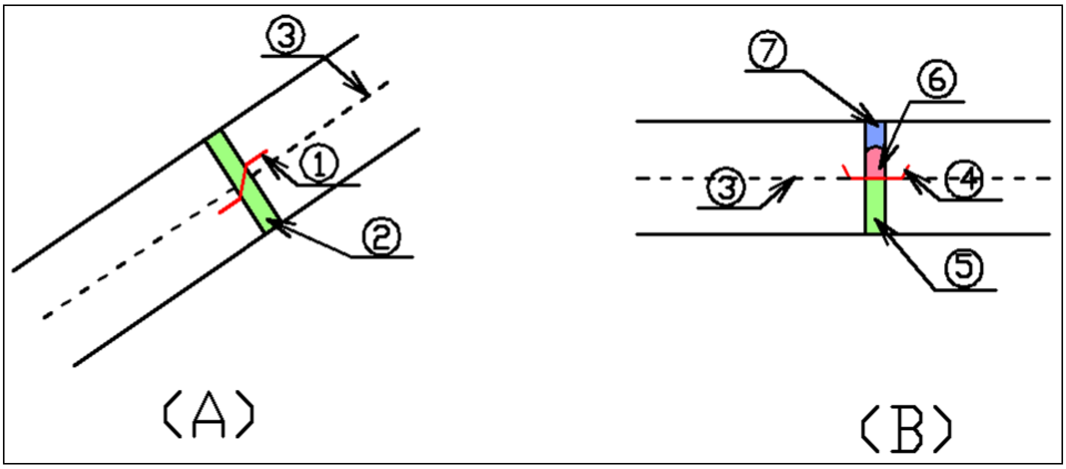

Figure 2.3 Details of vertical and horizontal joints (ICOLD, 1989-b)

(A) Horizontal joint
(B) Vertical joint
(1) Z-shaped copper waterstop
(2) Redwood filler
(3) Reinforcement
(4) U-shaped copper waterstop
(5) Compressible joint filler
(6) Premolded asphalt
(7) Mastic filler

6) The parapet wall, used to prevent overtopping of flood waves, is designed with very small heights about 1.0~1.5 m not to overburden the dumped rockfill (ICOLD, 1989-b).

7) The underlying supporting zone of face slab is consisting of manually placed huge blocks (ICOLD, 1989-b).

8) Rockfill slopes, 1V: 1.3H or 1V: 1.4H, are closer to the natural angle of repose of the selected rock type since rockfill are dumped from 30 m or higher elevations and sluiced afterwards (ICOLD, 1989-b).

Operational performance of Concrete Face Rockfill Dams designed and constructed with the above mentioned traditional features underlined the urgency for improvement of design features for higher dams (ICOLD, 1989-b).

2.3 Development of Modern CFRD

Following the successful performance of first trial on Quoich Dam in 1955, compacted rockfill is accepted to be an efficient material for concrete face rockfill dams. However, vibratory roller compactors used to be a brand new technology which was very costly to afford, hence it was after 1960's that compacted rockfill started to be commonly used for construction of higher concrete face rockfill dams (ICOLD, 1989-c).

Other than compaction of rockfill, traditional design goes under many changes but three main features are kept with small revisions. i) The cut-off wall is taken over by a toe slab called plinth, ii) main structure of rockfill is revised by compaction and appropriate zoning with an increasing size gradation towards downstream and iii) the reinforcement ratio decreases as the face slab got thinner and the details of vertical joints improved against openings while establishment of horizontal joints are avoided unless necessary.

2.4 Current Design Characteristics of CFRD

Current design of CFRDs consists of three inherited primary elements; face slab, plinth and the zoned rockfill also have some secondary elements, such as parapet wall and extruded curbs. Current design characteristics of a conventional CFRD constructed on an appropriate type rock foundation are given in Figure 2.4.

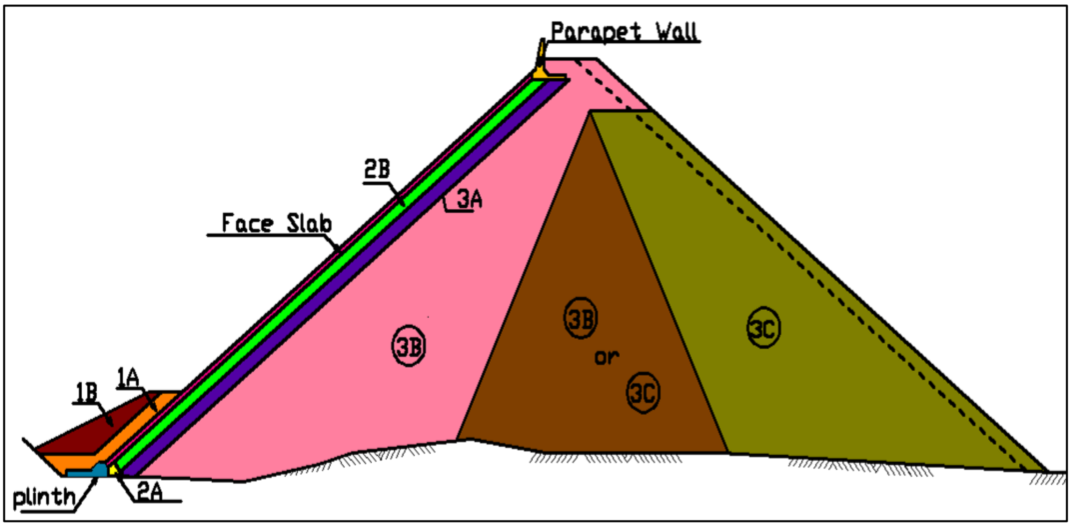

Figure 2.4 Current cross-section of conventional CFRD (Kleiner, 2005-c)

1A- cohesionless fine material zone
1B- random fill zone
2A- perimeter filter zone
2B- filter support zone
3A- transition zone
3B- rockfill zone
3C- rockfill zone

2.4.1 Plinth

Plinth, which connects face slab to the foundation preventing seepage through, is the modern design version of the cut-off trench. It also serves as concrete cap for grouting applied on the underlying foundation. Once the layout alignment is determined, design of plinth cross-section concerns about the selection of width, thickness, confirmation of stability under reservoir loading and impermeability treatment of the foundation.

Plinth segment located on the riverbed is called horizontal plinth because of the levelled foundation while tilted plinth segments on the abutments are called sloping plinths. Examples of horizontal, sloping and very steep abutment plinths are given in Figure 2.5-A, 2.5-B and 2.5-C, respectively.

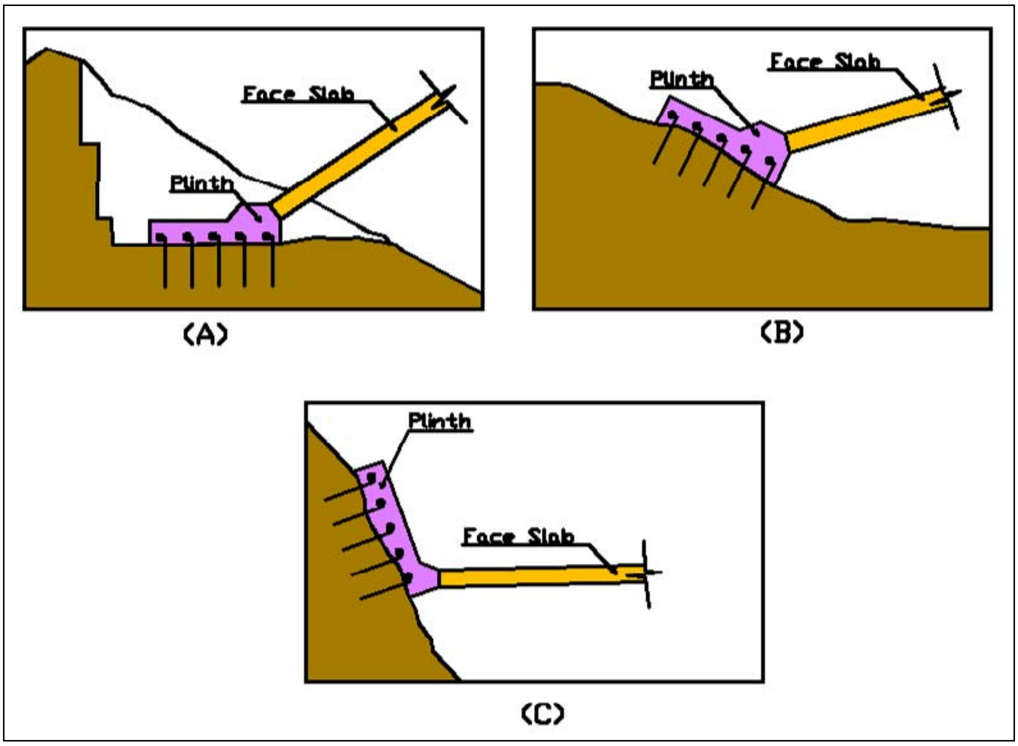

Figure 2.5 Different plinth types (Mori and Mataron, 2000)
(A) Horizontal plinth, (B) Sloping plinth, (C) Vertical plinth

Details of horizontal plinth located on the riverbed are given in Figure 2.6 (Cooke, 2000-b).

Point “O” in Figure 2.6, is the vertical projection of intersection of plinth with face slab, and key point for location of plinth alignment on site. AB plane is set perpendicular to face slab, forming the angle “α“ which varies according to the alignment of each plinth segments (Cooke, 2000-b).

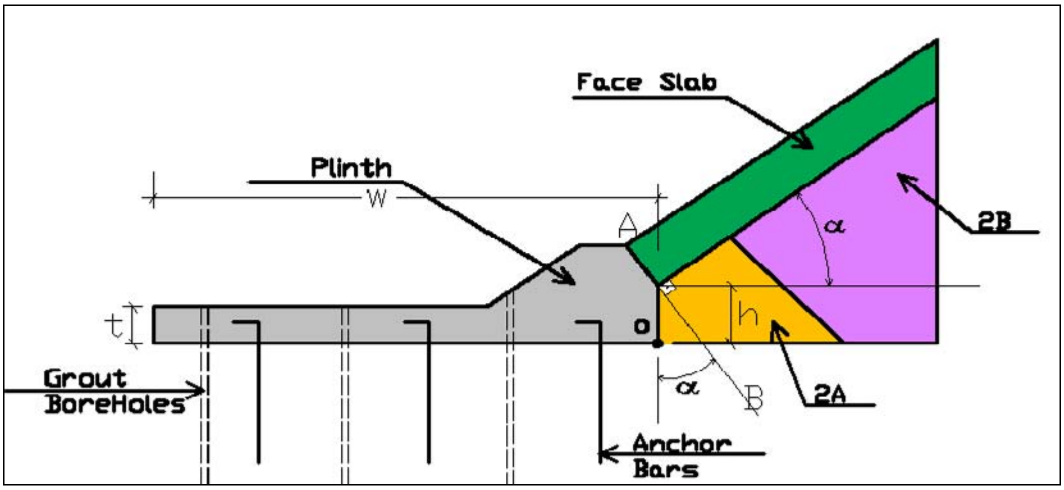

Figure 2.6 Basic geometry of conventional plinth (Cooke, 2000-b).

The experience of design thickness between 0.3 – 0.4 m was satisfactory in recently constructed dams. The maximum value determined for the river cross-section, is applied as a constant thickness all along the perimeter joint for simplification of construction (Cooke, 2000-b).

Thicker cross-sections are formed across very erodible fissured formations requiring overexcavations or a fault zone, through acute depressions of rock surface, road cut excavations or any other irregularity of rock surface causing abrupt local dents (ICOLD, 1989-c). Plinth of Mohale Dam (145 m) in Lesotho, the largest of South Africa, is given in Figure 2.7.

Shiroro Dam (110 m) in Nigeria is another example for usage of regional thicker plinth cross-sections, the reason of which is the existence of a fault zone and weak zones weathered deep to even 10-15 m in patches throughout the plinth alignment (Kleiner, 2005-d).

Yedigöze Dam (140 m) in Turkey currently under construction, has some thick plinth cross-sections as a result of the irregularity of the fresh rock surface of foundation.

Plinths mounted on thick concrete backfills require special design procedure. Plinth slab is fixed to the underlying concrete and work as a monolithic structure. Stability analysis is a casual procedure to be conducted which is not influent on overall design of a conventional plinth placed on a sound rock, but it is a critical step for thicker plinths (Cooke, 2000-b).

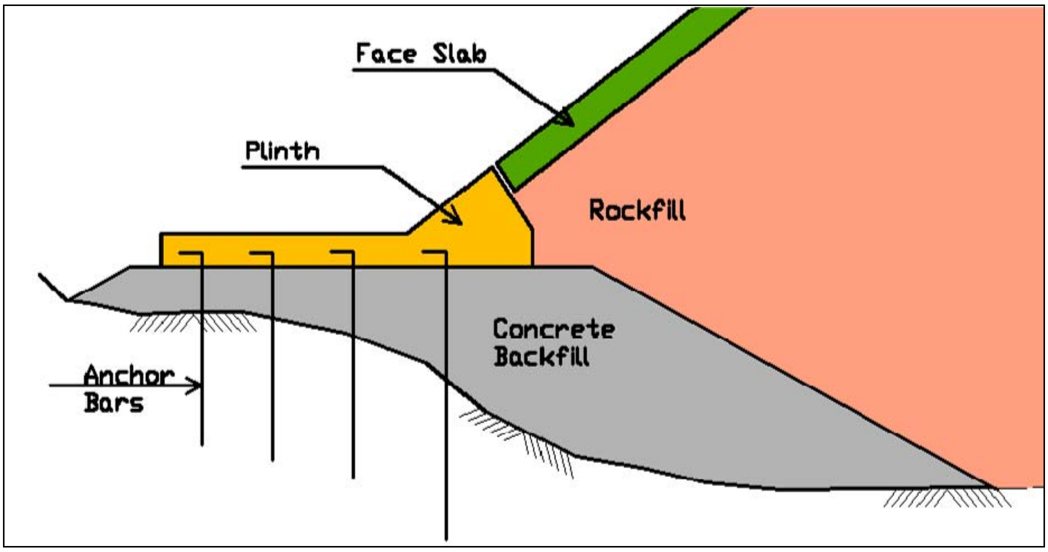

Figure 2.7 Typical cross-section of thick plinth (Kleiner, 2005-d)

Width, denoted by w in Figure 2.6, is directly correlated to the hydraulic gradient through foundation while this acceptable limit of hydraulic gradient differs for various foundation conditions. When foundation conditions demanded wider plinths, internal plinth complements the required length left from thicker external plinth (Kleiner, 2005-d).

Usually external and internal slabs are seperated by a cold joint ornamented with waterstops as done in Corrales Dam (75 m) in Chile, Ita Dam (125 m) in Brasil and El Pescador Dam (42.5 m) in Colombia, but an alternative design of monolithic internal and external plinths are applied to Sugarloaf Dam (17 m) in Australia. A typical cross-section of internal plinth application is given in Figure 2.8 (Kleiner, 2005-d).

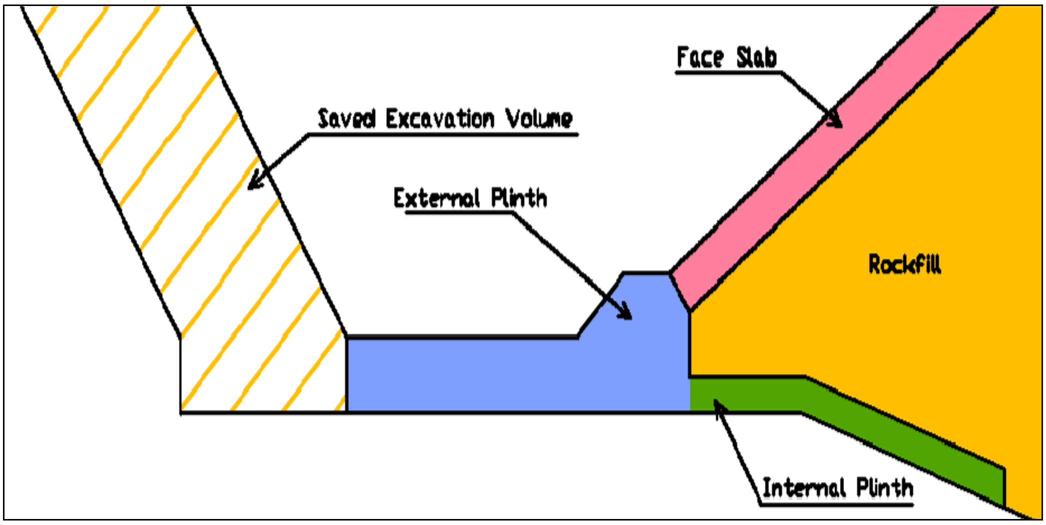

Figure 2.8 Internal and external plinth combination (Kleiner, 2005-d)

There are a lot of CFRDs constructed on deep alluvial soils, such as Puclaro Dam (80 m) in Chile founded on 113 m deep alluvium and Santa Juana Dam (113 m) again in Chile on 30 m deep alluvium. Khao Laem Dam (130 m) in Thailand was constructed on karst and a perminant inspection gallery was added on top of the plinth enabling emergency treatments. Details of Khao Laem plinth are given in Figure 2.9 (Watakeekul et al., 1985).

Diffferent safety measures are applied for various site conditions of precedent but the common practice is to construct articulated plinth tolerating larger differential settlements without any sacrifice of impermeability. Detail of an articulated plinth constructed for Santa Juana

Dam (113 m) and Puclaro Dam (80 m) is given in Figure 2.10 (Mori and Mataron, 2000).

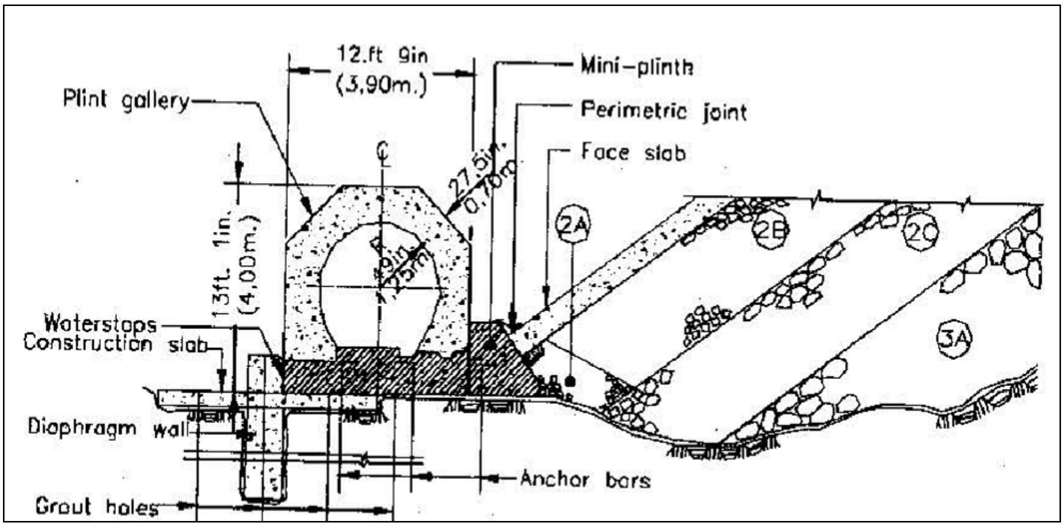

Figure 2.9 Khao Laem Dam plinth gallery (Watakeekul et al., 1985)

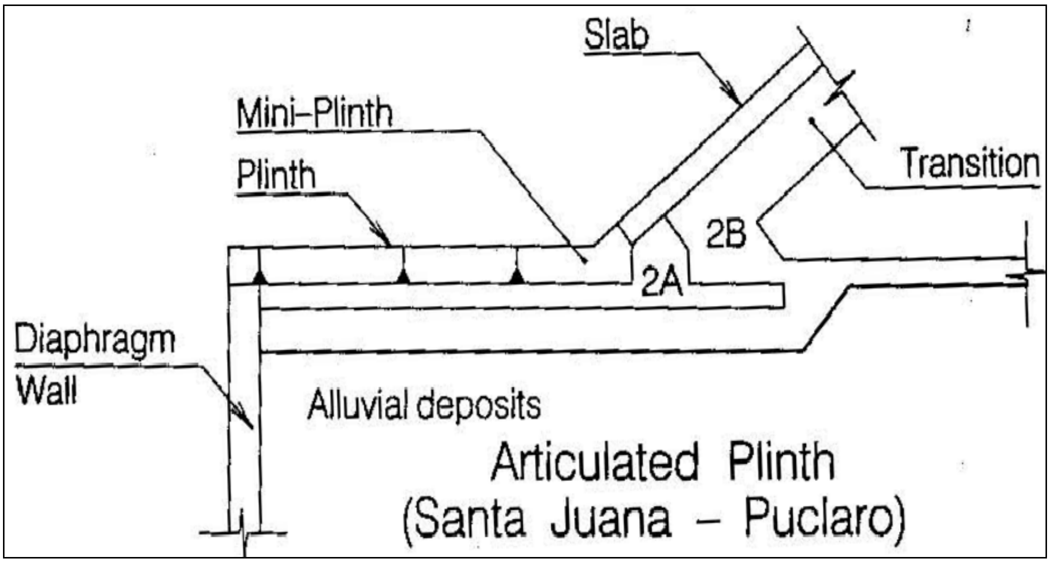

Figure 2.10 Articulated plinth of Santa Juana Dam (Mori and Mataron, 2000)

2.4.2 Perimeter Joint:

The main purpose of the perimeter joint is to connect the toe slab, that is fixed to the underlying foundation, to the face slab whose deformation is dependent on the underlying rockfill embankment. The connection of the perimeter must be qualified for imperviousness under maximum water head of reservoir loading and for safely tolerating the differential deformations of the face slab and plinth (Hedien, 2005-a).

Three dimensional displacements of shearing along joint direction **(t)**, settlement normal to the face slab **(s)** and opening normal to the joint direction **(o)**, given in Figure 2.11 both through and along the plinth alignment, are expected to be compensated by the perimeter joint (Hedien, 2005-a).

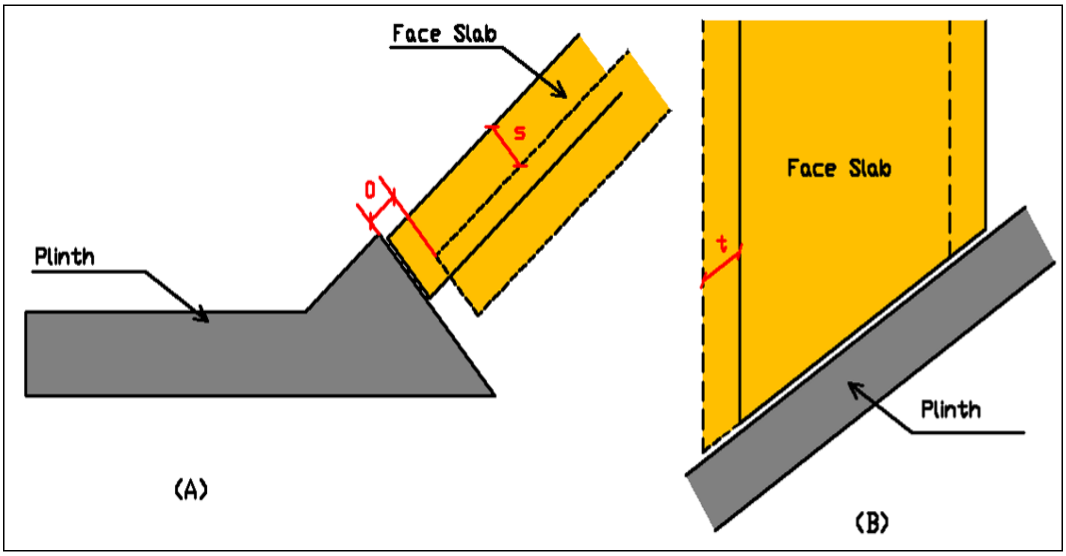

Figure 2.11 Perimeter joint movements (Hedien, 2005-a)

Water barriers are established in different orientations of three available locations of the joint which are upper, middle and bottom regions of the joint. There are existing CFRDs with two water barriers at different locations or three water barriers established with different type of waterstops. Perimeter joint details of Salvajina Dam (148 m) in Colombia are given in Figure 2.12 (Hedien, 2005-a). Salvajina Dam is founded on deep alluvium with an articulated plinth, thus perimeter joint detail of three water barrier layers, was applied on the connection of face slab with the connecting slab.

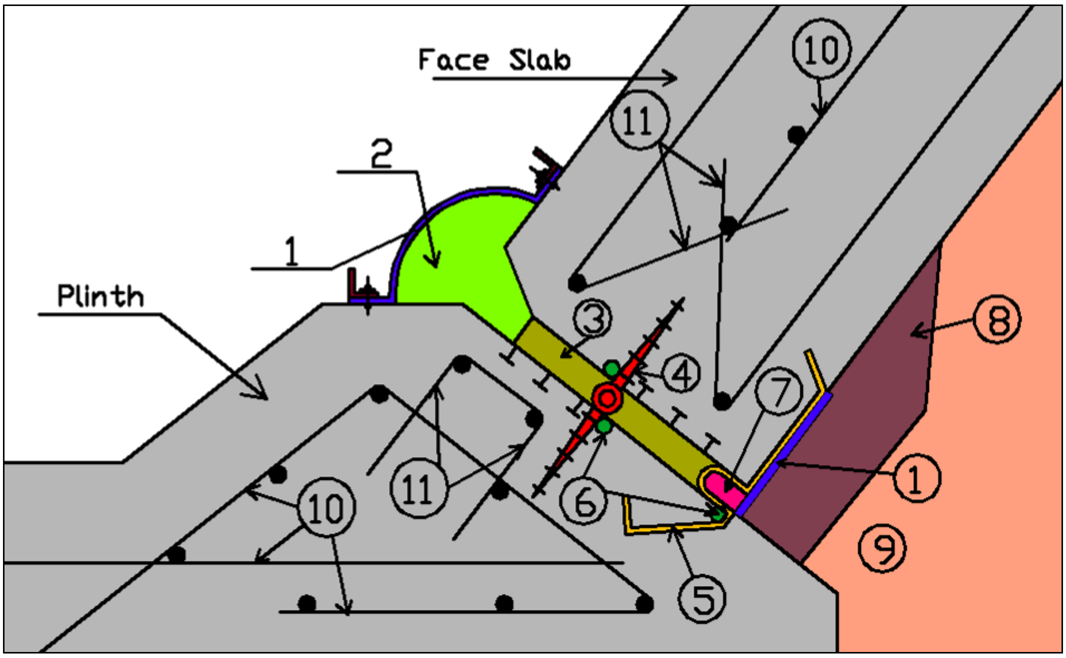

Figure 2.12 Perimeter joint for Salvajina Dam (Hedien, 2005-a)

(1) Hypalon band
(2) Mastic filler
(3) Compressible wood filler
(4) PVC waterstop
(5) Copper waterstop
(6) Neoprene cylinder
(7) Styrofoam filler
(8) Sand-asphalt mixture
(9) Filter zone
(10) Steel reinforcement
(11) Anti-spalling reinforcement

Various combinations and orientations of water barrier systems have been designed and tested, but all have the bottom-barrier in common starting with the satisfactory performance of Alto Anchicaya Dam. No matter what kind of material is used, safety against differential joint displacement without rupture is aimed. Dimensions are determined based on the expected joint movements and requirement of reinforcement placement. The W-shape is preferred in order to provide deformation without rupture. The alteration of mastic filler with deposit of cohesionless fine material is a great invent with brand new self healing characteristics of filler material regardless of the surrounding conditions. The details are given in Figure 2.13 (Hedien, 2005-a).

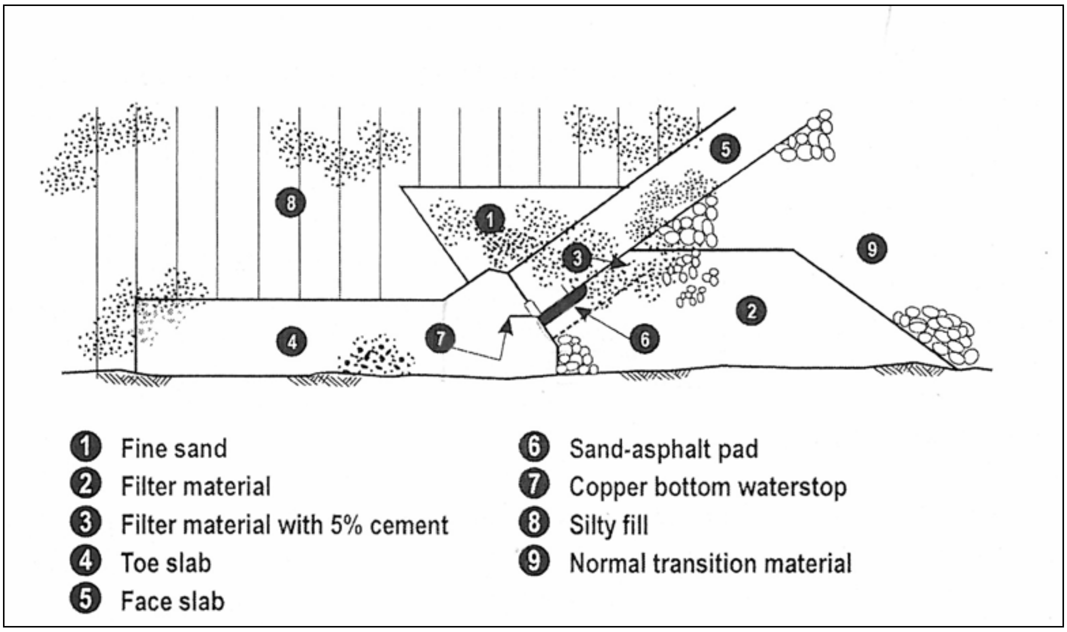

Figure 2.13 Upper barrier of cohesionless fine deposit (Hedien, 2005-a)

2.4.3 Face Slab

For concrete face rockfill dams face slab which is normally 95~99% submerged in reservoir for operation conditions, constitutes the main part of the water barrier by being exposed to reservoir water directly. Disappointing leakage performance of precedent CFRDs put face slab on the focal point of revision studies. Anticipated displacements and deformations are given in Figure 2.14-A, B and C for dam cross- section, in the plane of face slab and relative to the plinth respectively.

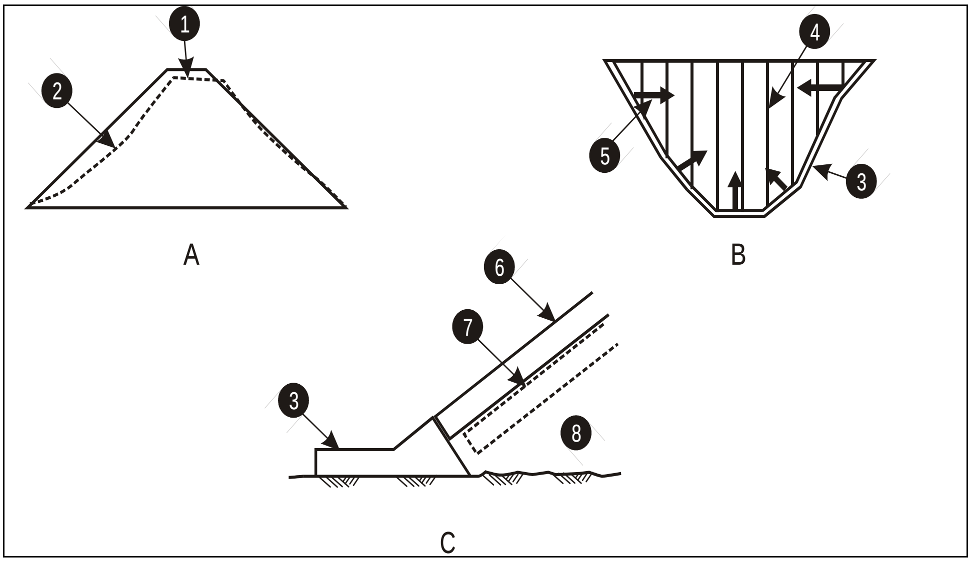

Figure 2.14 Embankment and face slab behavior (Hedien, 2005-b)

(A) Embankment deformations under water load
(B) Movements in the plane of face slab
(C) Face displacement at perimetric joint

(1) Crest settlement	(2) Face settlement
(3) Plinth	(4) Face joints
(5) Direction of movements	(6) Face slab
(7) Face position after water load	(8) Rockfill

Reinforcement ratios in both vertical and horizontal directions are increased down to 0.30~0.40% with satisfactory face slab performance (Heiden, 2005-b).

Face slab designs gets thinner. The thickness (th) of dams higher than 100 m can be calculated from Equation (2.1) (Heiden, 2005-b).

$$th=0.3 + (0.002\sim0.004)H \tag{2.1}$$

Where th is face slab thickness in m, and H is the maximum height of the dam in m.

Concrete slab covering the upstream slope consists of main vertical panels which are constructed with slipforming. The width of the panel mainly depends on the equipment characteristics. In China the common practice was to use slipforms operated by electromechanical winches while accomplishing slipforms by hydraulic jacking is another construction method widely used around the world. In Figure 2.15, the site application of face slab construction with slipforming and the reinforcement placement on tilted upstream face is given.

One of the most significant features of CFRDs is the allowance for staged construction. Face slab construction for many existing CFRDs, such as Tianshengqiao-1 Dam (178 m) in China, Ita Dam (125 m) in Brazil and Aguamilpa Dam (187 m) in Mexico, were completed in more than one stage parallel to the embankment construction (Mori and Mataron, 2000).

Figure 2.15 Site application of face slab construction (Marengo, 2007)

2.4.4 Parapet Wall

Parapet wall application on the crest is one of the main advantages of concrete face rockfill dams which significantly reduces the rockfill embankment volume especially for high dams. This economic saving, can not be disregarded if the embankment material is supplied from rock quarry instead of using excavated materials. In common design practice, flood volume is compensated by the parapet wall. Even though CFRDs are not as vulnerable as ECRDs against overtopping, top elevation of parapet wall is determined in order to prevent overtopping during probable maximum flood (Sundaram and Kleiner, 2005). In Figure 2.16 parapet wall details of Mohale Dam (145 m) in Lesotho are given.

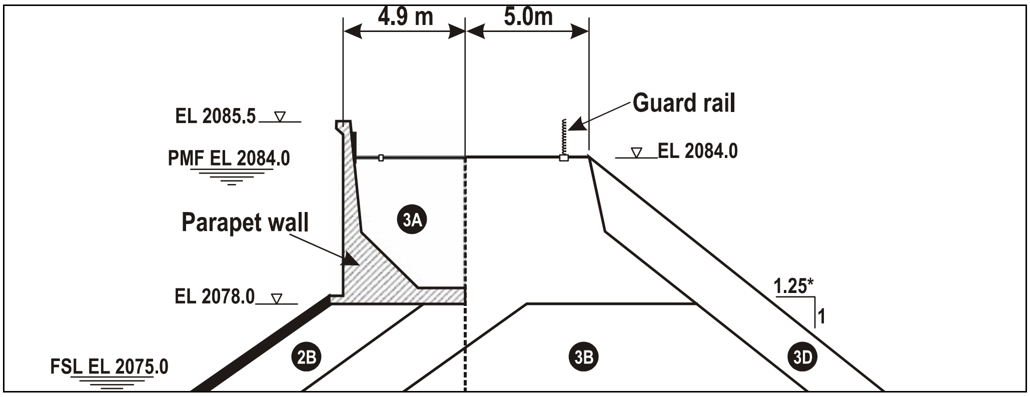

Figure 2.16 Parapet wall of Mohale Dam (Sundaram and Kleiner, 2005)

2.4.5 Embankment and Extruded Curbs

Embankment constitutes the major constituent of the dam body, however it is not directly exposed to the reservoir remaining in dry state with no uplift and pore pressure within the dam body.

Zoned configuration of the embankment is stable against flow passing through the body, especially against passage of seasonal flood flows during construction period but overtopping must be avoided if adequate measures are not taken.

Scheduling dam body construction is a complicated task involving several uncertain parameters. And divisibility of dam construction into many stages depending on the valley shape and river diversion is another important feature of concrete face rockfill dams (Mori and Mataron, 2000).

A priority section composing the majority of the rockfill volume is constructed on the abutments before river diversion and on the riverbed after the diversion and dam can stay stable even under a flood of a 500 year return period the second rainy season of the construction schedule.

The first flood will be controlled by the optimized cofferdam. (Mori and Mataron, 2000).

From upstream towards downstream according to their mission within dam performance, dam zones are further divided into subgroups such as; 1A, 1B, 2A, 2B, 3A, 3B, 3C, 3D and 3E (3D and 3E are optional) depending on the size gradation, location or layer thickness. In Figure 2.4 orientation of these zones is given in detail. Starting from upstream, 1A & 1B zones are located outside the dam body on the plinth, protecting perimeter joint from reservoir impacts and providing additional fine particle reservoir required in case of joint failure. Behind the face slab, 2A and 2B are the primary zones supporting face slab and serving as a filter layer since gradation of these zones requires the most attention, specially 2B zone performance, its evolution from traditional design and its importance for dam behavior were discussed in the previous sections of this study. 3A, 3B and 3C zones are located downstream of 2A and 2B. 3B and 3C consists of very large rock blocks with increasing size towards downstream, completing the dam section. 3A zone is the transition zone from gravel sized 2B to rock boulders of 3B zone, thus its size gradation requires extra attention. 3D and 3E are very pervious zones consisting of very large rock boulders and located at most downstream of the section in order to provide a proper self drainage of the dam and dumped rockfill were accepted in some cases since lower modulus of dumped rockfill at downstream does not affect the performance of face slab (Kleiner, 2005-c).

In current design practice, Zone 1A is defined with fine-grained cohesionless silt and fine sand with isolated gravel and cobble sized rock particles up to 150 mm. Cohesionlessness is particularly important for proper performance of this zone. This zone is expected to easily migrate through prospective face slab cracks and clog openings in 2B zone for preventing further movement of leaking reservoir water (Kleiner, 2005-c).

Zone 1B - random fill zone - which is directly exposed to reservoir water and loading consists of random mix of silts, clays, sands, gravels and cobbles to protect 1A Zone against reservoir impacts. Common practice is to use materials supplied from appurtenant structure excavations and placing in 200 to 300 mm layers (Kleiner, 2005-c).

Zone 2A, which is referred to as “perimeter filter zone”, is the smallest zone in volume but very important for performance of the dam. Located within two-three meter downstream of the perimeter joint (on internal plinth if exists).

This zone is expected to serve as a filter thus gradation is specially defined. Gradation of 2A, is determined in order to capture migrating fine particles of 1A zone and serve as a secondary watertight barrier after being congested by washed 1A particles, without piping into 2B and 3A zone. The zone must be placed in 200 to 400 mm layers and compacted with vibratory compactors, and protected from damage or erosion during construction.

2B zone is very important for the face slab performance, thus its design has evolved considerably parallel to the progressing construction techniques and precedent experience as discussed in the former sections. Smaller maximum sizes and larger fine percantage were adopted. The gradations with maximum size of 250~330 mm and minimum size of 50~75 mm were exposed to severe segregation during construction because of the vibration applied for upper layer constructions. Thus, further size reduction was accepted.

10~15% passing through # 200 sieve material and 35~55% sand was applied for sooner constructed important CFRDs, such as in 1993 Tianshengqiao-1 Dam (178 m) in China and in 1994 for Xingò Dam (150 m) in Brazil (Souza, 2007). The practice experience of both dams underlied the fact that excessive usage of finers leaded cohesion within the zone. Brittle characteristics of cohesive material did not compensate for the deformation with diffrential settlement of the underlying rockfill and unavoidable open cracks were formed during construction. Sealing these cracks with different materials before the slab construction was not enough to prevent re-opening of these underlying cracks leading the face slab to deform in the same manner (Souza, 2007).

The gradation is revised for the above mentioned reasons. The main difference is reduction in the percentage of fine particles passing through # 200 sieve. A maximum of 5~7% non-cohesive fines are recommended. As a common trend crusher-run material with specified gradation is recommended for many CFRDs, but the material must be used after crushing, screening and washing. Mixture of the crusher-run material and natural riverbed sand must be avoided to prevent gap grading (Kleiner, 2005-c)

Preparation of the surface slope was drastically simplified by constructing a concrete curb at the upstream face after every layer using a considerably low cost curb equipment and compacting the following layer against the curb. Concrete curbing was one of the main developments of CFRD construction technique since it provided considerable amount of equipment, labor and material cost savings while yielding a very smooth, clean surface for subsequent operations of form and reinforcement placement and slab construction (Orejuela, 2007). Site application is given in Figure 2.17.

Figure 2.17 Site application of concrete curb casting (Orejuela, 2007)

Zone 3A, which is a transition zone between face slab support 2B zone and rockfill zone 3B with maximum size in the order of 400 mm, is placed in 400 mm layers and compacted by 4 passes of 10 ton or heavier smooth drum vibratory roller compactor. The horizontal width of the layer is determined based on the precedent experience and the same width between 2~4 m is commonly adopted both for 2B and 3A in practice. Their horizontal level is adjusted to the same elevation which is one layer thickness above adjacent 3B zone since transition between these zones is very important in order to prevent any face support material loss in case of any leakage through the face slab (Kleiner, 2005-c).

Zone 3B is located within the two-thirds or three-fourths of the dam shell which transfers the load to the foundation undergoes severe deformation upon reservoir loading. This zone mainly consists of rockfill with maximum size of 1000 mm, placed with 1000 mm thick layers and compacted by 4 passes of 10 ton smooth drum vibratory roller compactor. Watering the rockfill before compaction is another important issue significantly affecting the amount of deformation under reservoir loading (Kleiner, 2005-c).

The precedent experience rockfill compaction required water volume equal to 10~25% of rock volume. For weak rocks used in the rockfill layer thickness is reduced and amount of water addition is increased in order to achieve required rockfill density. The adequecy of the compaction is usually determined by the site tests conducted during compaction.

Zone 3C which takes place in the downstream shell of the body completes the required volume of embankment section and consists of large rock blocks in the order of 2000 mm. As the size of the particles increases the efficiency of the compaction reduces, permeability and compressibility of the zone increases consecutively. These weaker characteristics of zone 3C are not considered to be critical for dam performance especially for slab deformation. The zone, which is designed with more flexible gradation limit, is placed in 2000 mm thick layers and compacted by 4 passes of 10 ton smooth-drum vibratory roller compactor (Kleiner, 2005-c).

3D and 3E zones enable the self-draining of seeping water through embankment. High capacity of self-drainage is a safety key for a CFRD but it is a must for concrete face gravel fill dams, since leakage and seepage may result in hazardous breaching for gravel fills. For this purpose, a continuous chimney drain and a proper underdrain at the base of the dam is required for concrete face gravel fill or poorly drained rockfill dams. But a simple base drain at the riverbed section is satisfactory for well-drained rockfill embankments (Kleiner, 2005-c).

The details of the processes required to prepare the material for placing in the embankment are dependent on the characteristics of the proposed barrow pit or quarry source (Kleiner, 2005-c).

CHAPTER 3

PERFORMANCE OF PRECEDENT CFRDs

3.1 General

Concrete Face Rockfill Dam design is empirical in nature and is based on the precedent performance and experience (Cooke, 2000-a). However, adding to the precedent experience, invention of new construction equipments and development of constructon technique supports the design progress of CFRDs, such as invention of vibratory roller compactor, slip form and extruded concrete curb equipment. Concrete Face Rockfill Dams have some inherent safety features, enabling the empirical design and allowing new design trials, which are (Cooke, 2000-a):

-The zoned rockfill allows flow passing through

-The whole embankment volume is not directly exposed to the reservoir water

-Under normal operation conditions, pore pressure is not anticipated but in case of any leakage through face cracks the rockfill is zoned and designed for self-drain and leakage is not critical for dam stability and safety

-For satisfactory operation of grout curtain, uplift pressures are not involved for concrete face rockfill dams since embankment is not in direct contact with the reservoir water

-Rockfill behind the face slab is very stable under high seismic loading

-Shear strength of the rockfill is very high and steeper slopes are safely adopted for various concrete face rockfill dams

3.2 Performance Experience of Significant Precedents

3.2.1 Salt Springs Dam (1931):

Salt Springs Dam, constructed in 1931 in USA, is the first concrete face dumped rockfill dam reaching a height of 100 m. With its large reservoir, Salt Springs Dam was designed to be very profitable for Pasific Gas and Electric Co., but the excessive amount of leakage observed in 2001 (order of 1200 l/s = 42.4 cfs) diminishing the profitability, was tried to be reduced down to 850 l/s (=30.0 cfs), which is acceptable for refillment of reservoir each year, by 22 repairments between 1938 and 2003 throughout 70 years of operation. The face slab was heavily deteriorated under reservoir impacts as given in Figure 3.1 (Scuero, 2007).

The first repair treatments are replacing spalling concrete patches, but replaced concrete has gone under the same impacts and spalled. Thus, geomembrane placing are agreed to be the best solution for face repairment. After installation of geomembrane over the defected face sections, the leakage was reduced down to 400~450 l/s (=15.0 cfs) which is quite satisfactory compared to the target value of 850 l/s (=30.0 cfs) as given in Figure 3.2.

Figure 3.1 Deteriorated face slab of Salt Springs Dam (Scuero, 2007)

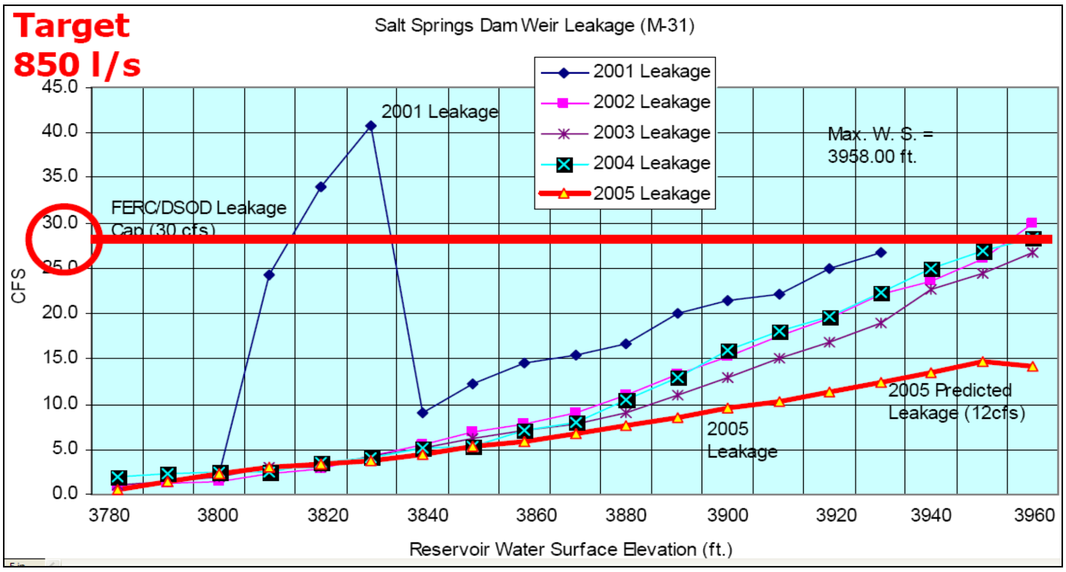

Figure 3.2 Leakage rates of Salt Springs Dam against reservoir head (Scuero, 2007)

3.2.2 Cogoti Dam (1938)

Cogoti Dam is one of the pioneers of concrete face rockfill dams. In the time of its construction, which was about 1938, vibratory roller compactors had not been discovered thus dam was constructed by dumped rockfill (ICOLD, 1989-a).

Cogoti Dam, constructed in Chile with a height of 85 m, was the second heighest of its time after Salt Springs Dam (100 m) in USA and rockfill was placed in high-lifts without any compaction and slucing (Arrau et al., 1985).

Cogoti Dam had gone through severe seismic loadings for four times after its construction was completed. The Richter magnitude of earthquakes were 7.9, 7.1, 7.5, 7.7 and created maximum ground acceleration of 0.19g, 0.04g, 0.05g and 0.03g at dam site respectively (Arrau et al., 1985).

The biggest first earthquake, accelerated the anticipated settlement of dumped rockfill. The other almost same magnitude earthquakes had slight or no effect on the settlement as given in Figure 3.3. The maximum settlement, 108 cm (1.7 % of height), is observed close to the midpoint of the crest where the dam height is only 63 m. Arching effect of the steep abutment, which can be explained as the transfer of a portion of the stress to the abutments, caused smaller settlements than expected at this point (Arrau et al., 1985).

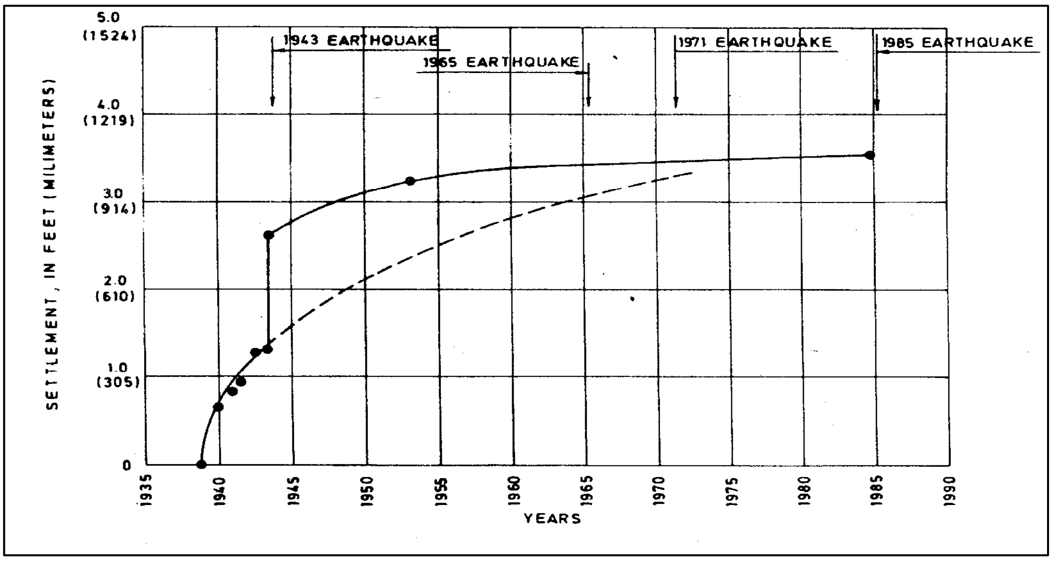

Figure 3.3 Crest settlement curve for Cogoti Dam (Arrau et al., 1985).

As a general view of the seismic performance of Cogoti Dam, the most significant effect was the drastic settlement increase, and despite some longitudinal crest cracks face slab was not damaged. Only repairment was dumping rockfill on the crest in order to compensate for the settlement. Spalling of concrete due to deformable joint grid did not lead to deterioration even after 45 years of operation (Arrau et al., 1985).

3.2.3 Alto Anchicaya Dam (1974)

Alto Anchicaya Dam was constructed in Colombia in 1974 with a height of 140 m for the purpose of energy generation. The abutments of the dam site, especially the right one, was quite steep and the shape factor (SF) of the valley, defined by Equation (3.1) was quite small, such as 1.1. The riverbed cross-section is given in the Figure 3.4 (Materon, 1985).

$SF = A / H^2$ (3.1)

where A is face slab area in m^2 and H is maximum dam height in m.

Design improvement of face slab support zone was first tried on Alto Anchicaya Dam and the large rock boulders were replaced by a compacted zone with a maximum grain size of 300 mm (12”) and an average of 31% passing through 1” sieve mesh (25.4 mm) and 12% of sand passing through seive # 4 (4.76 mm) in order to minimize the deflection of the slab. Perimetral hinge slabs were defined as the common trend for traditional design of early CFRDs. General layout of the face slab with its joints is given in Figure 3.5 (Materon, 1985).

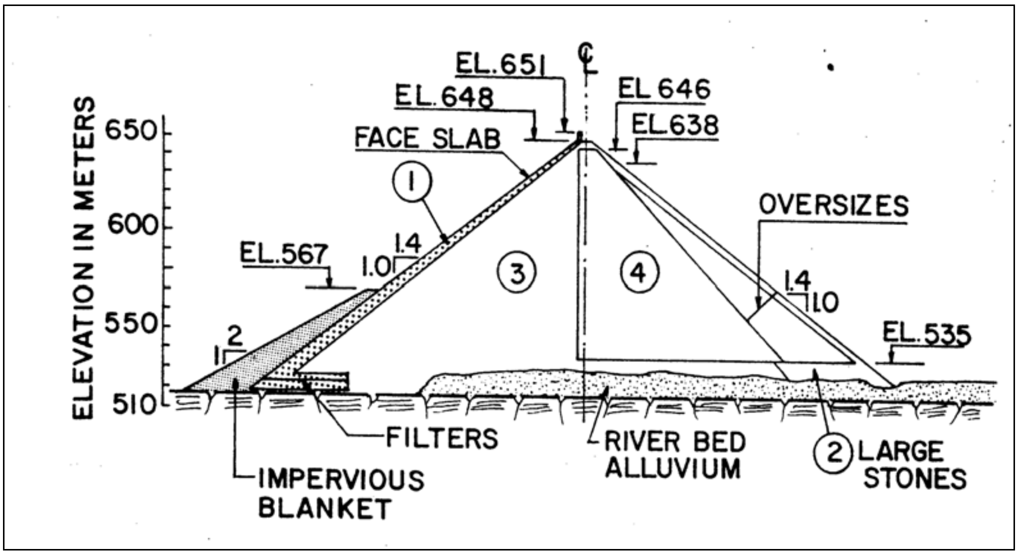

Figure 3.4 Riverbed section of Alto Anchicaya Dam (Materon, 1985)

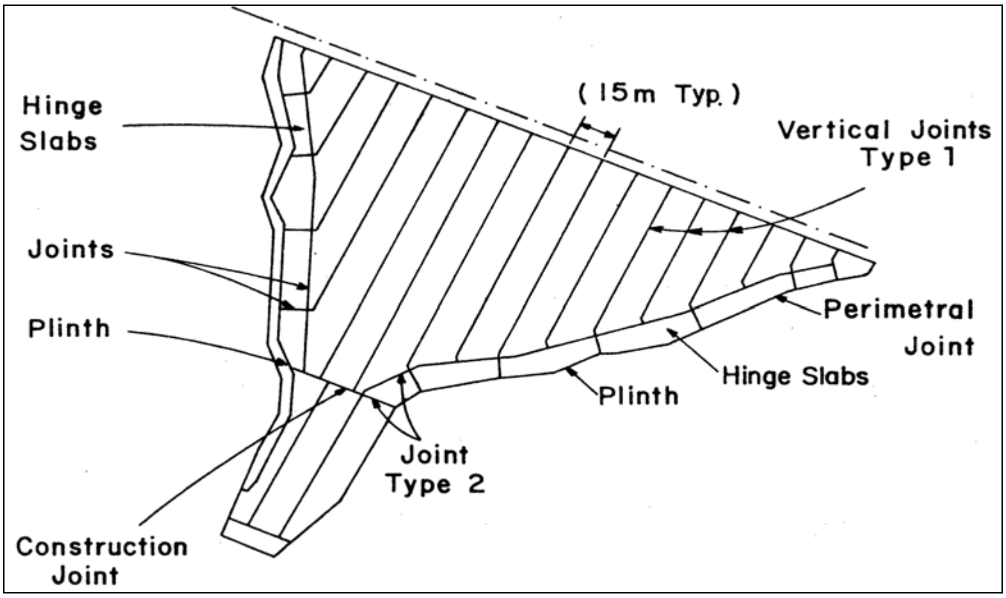

Figure 3.5 Face slab layout of Alto Anchicaya Dam (Materon, 1985).

Because of the rainy characteristics of dam site climate, face slab was placed simultaneously with the embankment construction (Materon, 1985).

Due to first filling of reservoir the leakage rate increased from 14 l/s to 1800 l/s and face slab was inspected both before and after the reservoir was lowered. It was concluded that the face slab successfully compensated the deformation of the underlying rockfill without any severe cracking other than negligeble hair cracks in the central slabs but the main water passage took place through the perimeter joint of both abutments. The amount of openings of steep abutments was very high and the rubber waterstops were not able to deform and compensate for the openings beacuse of the loose adhesion with concrete as a result of isolated void presence under the waterstop. Defected joints were filled with mastic which was covered by asphalt-sand and impervious material mixture reducing the leakage down to 446 l/s and after the second treatment of sand-clay-gravel-bentonite placement the final leakage was reduced down to 180 l/s (Materon, 1985).

3.2.4 Golillas Dam (1984)

Golillas Dam, constructed in Colombia in 1984 with a height of 125 m, was another significant concrete face embankment dam whose performance would provide various experience for the following dams (Marulanda and Amaya, 1985).

Golillas Dam was an extreme example for dams constructed across steep slope canyons with a shape factor smaller than 1.0 (almost 0.9). The construction period experience and operational performance results provided very useful and guiding information for both design and construction of following concrete face embankment dams constructed in a very narrow valley. A general view of Golillas Dam face slab is given in Figure 3.6 (Marulanda and Amaya, 1985)

Golillas Dam was the first dam whose embankment consists of zoned gravel fill. The quality of the available gravel was very important to determine the slopes and the requirement of a vertical chimney drain connected to a horizontal draining layer.

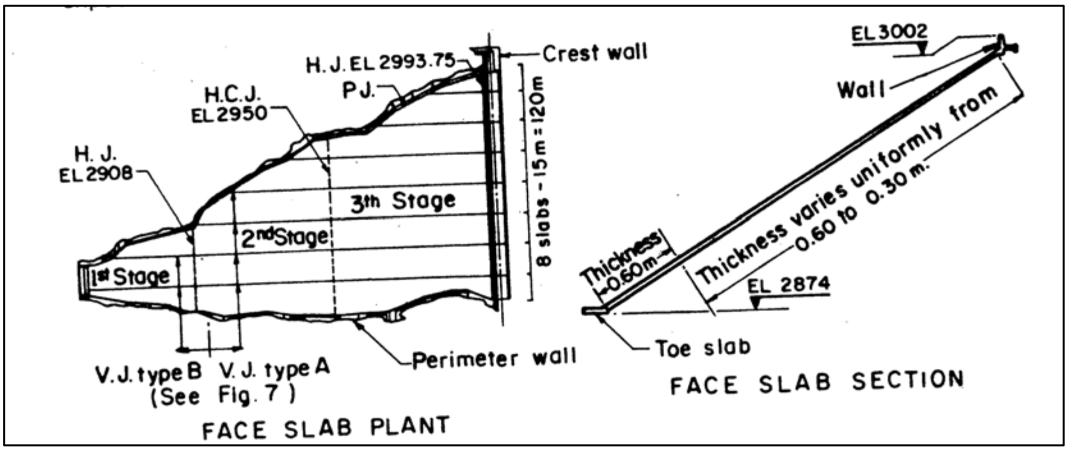

Figure 3.6 Face slab layout of Golillas Dam (Marulanda and Amaya, 1985)

For very narrow canyons with shape factor (A/H^2) equal or less than 3 arching effect takes place and the portion of the stress loading on the embankment are transferred to the abutments and observed settlements are always smaller than expected values which are calculated by the void ratio of the material and the calculated vertical load above the point of interest (Kleiner, 2005-a).

Because of the tough site conditions appurtenant structure construction was not completed by the end of dam body construction. Thus, the embankment settled under gravitational forces only. First impoundment was started after four years of delay and had to be stopped around mid height because of the excessive leakage rates. The material removed and dropped into the reservoir during cleaning of weak abutment zones accumulated on the perimeter joint and flowed trough the joint clogging the semi-pervious 2B zone (this zone was designated by Zone 1 in the time of application) (Marulanda and Amaya, 1985).

This incident brought the concept of controlling leakage by placing sandy and silty material into the scene underlining the importance of size gradation design of face support zone. The maximum grain size of Golillas

Dam 2B Zone (Zone 1 in original designation) was 15 cm and it was understood that filtering material of relatively low permeability would retain the finer particles dumped from outside and create the required seal for joints (Marulanda and Amaya, 1985).

3.2.5 Salvajina Dam (1983)

Salvajina Dam was another significant example of concrete face gravel fill dam with a height of 148 m constructed in 1983. It was constructed on a widely varying foundation and it was not possible to excavate all of the weathered or weak zones of the foundation before placement of the plinth and embankment (Kleiner, 2005-e).

At Salvajina Dam site, deep deposits of residual soil partially covered the rock at the abutments and a thick alluvium (of the order of 30 m) filled the river channel. The upstream cofferdam and plinth excavations executed on the riverbed exposed to the dense deposit with big rock boulders and gravel in a sandy-silty mixture characteristics of the allluvium. Thus it was agreed not to remove the whole volume leaving the downstream part and placing the embankment on the residual soil by required modification in the embankment zoning depending on the fact that deformations of the foundation and the embankment downstream from the axis did not affect the overall performance of the face slab (Ramirez et al., 1985).

The main concern was the design and construction of the toe slab and perimeter joint details (see Figure 2.12 for perimeter joint details of Salvajina Dam). Different plinth sections adopted on various foundation conditions such as residual soil and competent rock are given in Figures 3.7 and 3.8, respectively (Kleiner, 2005-e).

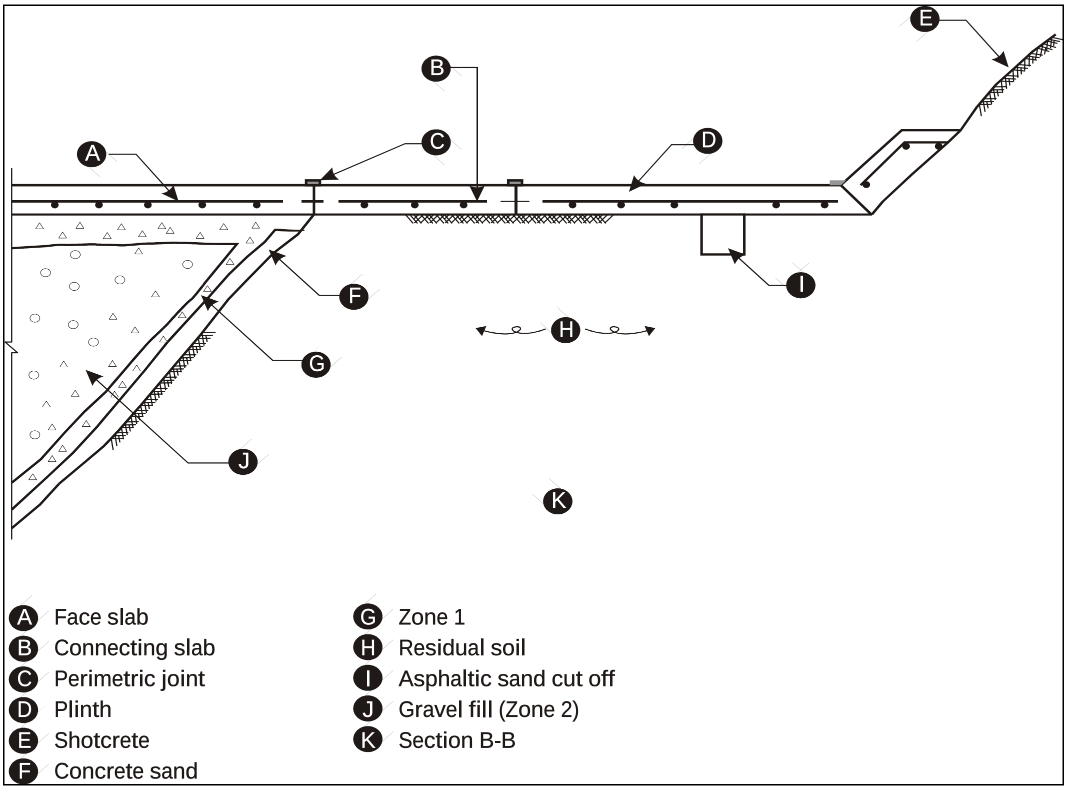

Figure 3.7 Salvajina Dam plinth founded on residual soil (Kleiner, 2005-e).

Upon first filling of the reservoir, the performance of the dam was very successful with the maximum face settlement of 5.0 cm and a total leakage of 75 l/s (60 l/s through face slab and 14 l/s through abutments) were recorded proving that CFRDs were not neccessarily constructed on hard non-erodible rock upon adequate design and proper construction of face slab CFRDs. Salvajina Dam also leaded the usage of low strength rockfill in the downstream shell of the dam (Ramirez et al., 1985).

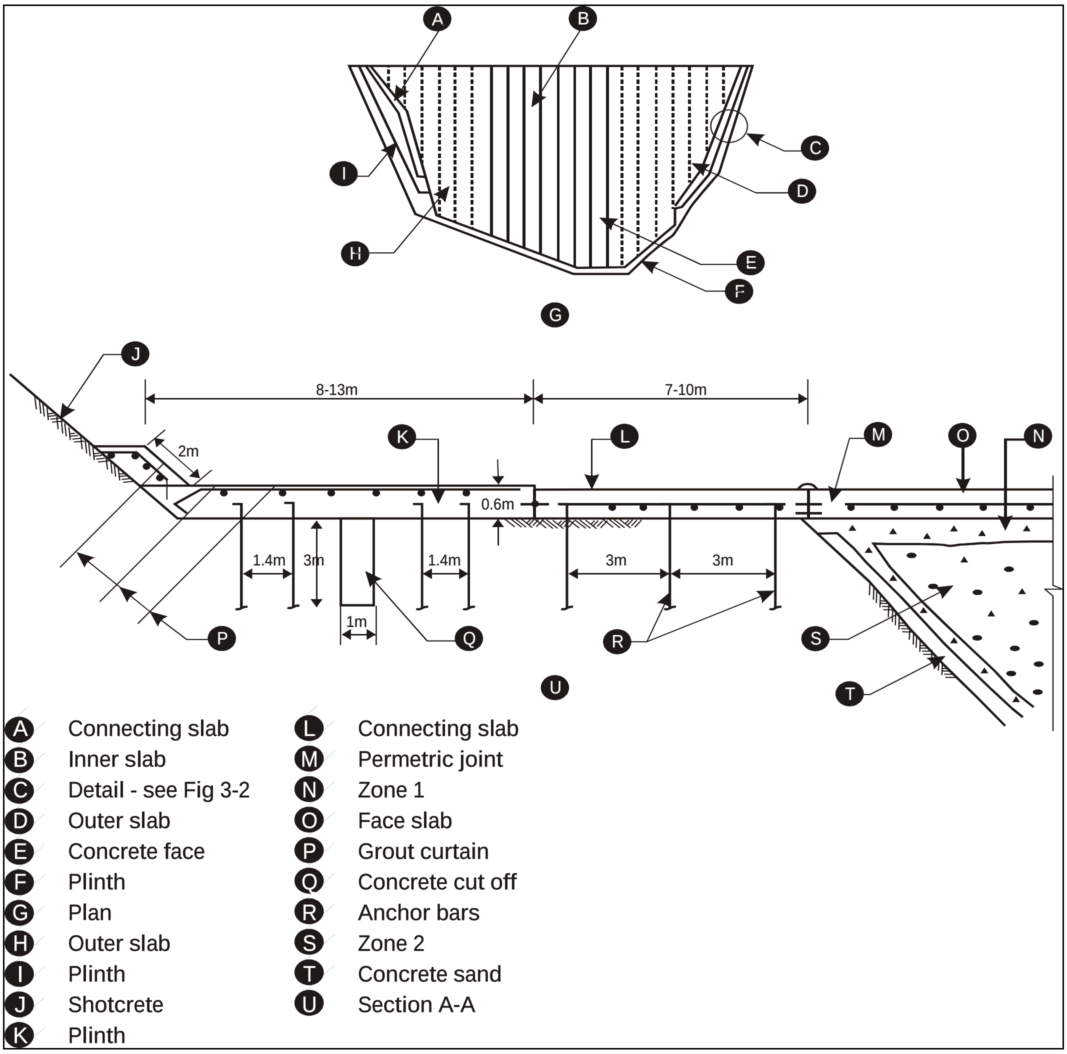

Figure 3.8 Salvajina Dam plinth founded on less competent rock (Kleiner, 2005-e).

3.2.6 Xingó Dam (1994):

The third Brazilian CFRD was Xingó Dam constructed with a height of 140 m. The dam was a conventional application with slopes and other characteristics adopted from the first Brazilian CFRD Foz do Areia Dam, but the size gradation of face support zone and transition was significantly smaller than the precedent experience (Penman, 2000). The main cross-section is given in Figure 3.9.

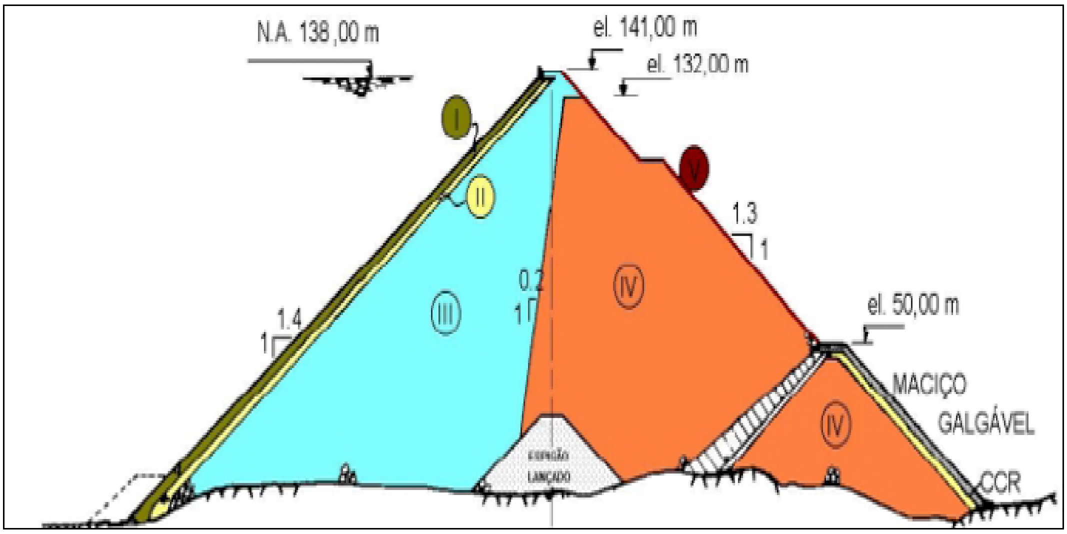

Figure 3.9 Main cross-section of Xingó Dam (Souza, 2007)

The gradation of the Zone 2B (zone 1 in original designation), included 10~15% of particles passing through sieve # 200 and 35~50% of sand. The concrete face slab was placed simultaneously with the embankment construction. During construction period when first stage of the face slab was completed and the embankment was raised almost to design height, facial cracks were observed in 2B Zone on the left abutment as given in Figure 3.10 (Souza, 2007). The initial treatment was facial sealing by mastic and fill placement over the cracks. However, the same cracks re-opened and accompanied by newly formed cracks. Before placement of face concrete, all cracks were filled with sand and compacted by vibratory roller.

Figure 3.10 Cracking of face supporting zone at Xingó Dam (Souza, 2007)

During construction, the rockfill within valley section (which was the most vulnerable against settlement with the highest embankment and reservoir load without supporting of the abutments) settled under compaction and other loading of the upper layers. Consequent to this drop of central region the abutment parts were urged to settle towards the valley resulting tension around abutments. Despite compressible rockfill, face supporting 2B Zone which was very brittle due to high content of fine particles could not compensate for the deformation and cracked (Souza, 2007).

Superficial sealing of these cracks did not prevent the progression of the deformation underneath. Upon first reservoir impoundment, face slab cracked at the location of 2B zone cracks which was identified by underwater inspection executed in consideration of the drastic increase in leakage rates. Schematic presentation of face cracks is given in Figure 3.11, whereas and the underwater inspection view is given in Figure 3.12.

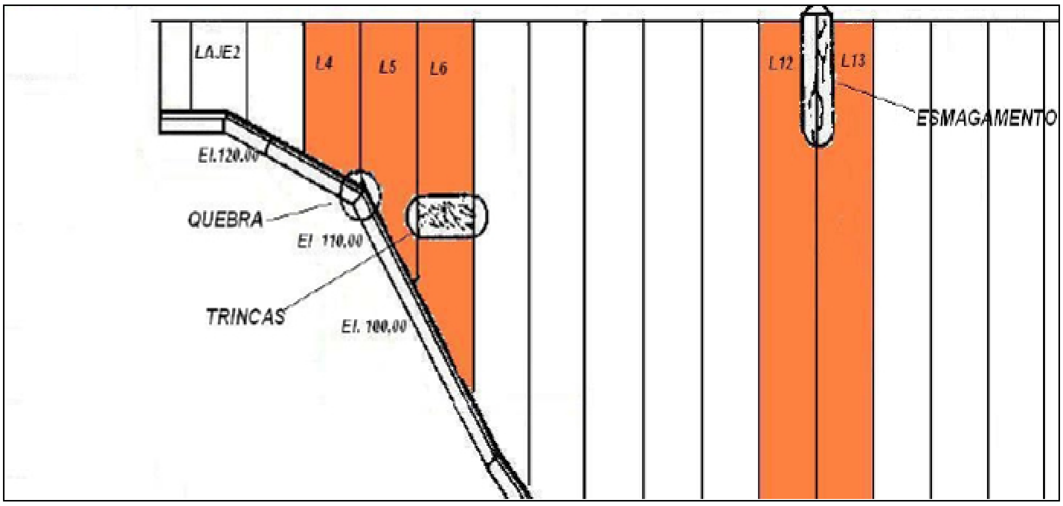

Figure 3.11 Face slab cracks of Xingó Dam (Souza, 2007)

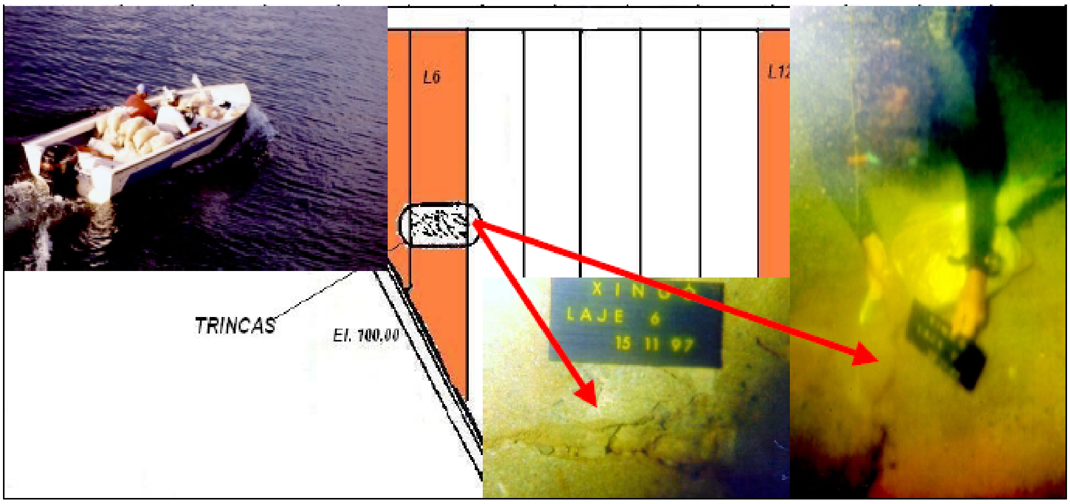

Figure 3.12 Underwater inspection of face cracks at Xingó Dam (Souza, 2007)

Leakage rates increased upto 200 l/s and dumping of dirty sand was not satisfactory for reduction. The settlement were not completed and aggravated the opening of cracks leading increase in leakage. Seeping water reached through the coarser rock zones and fastens the settlements which in return caused cracks to open further.

Xingó experience indicated that higher fine percentage in face supporting zones should be avoided and maximum of 7~8% fine particles passing through sieve # 200 were agreed to yield satisfactory performance for both filtering washed particles and supporting face moderating the underlying rockfill deformations (Pinto and Marulanda, 2000).

3.2.7 Kürtün Dam (2002) and Turkish CFRDs

133 m high Kürtün Dam was the first Turkish concrete face rockfill dam constructed on the Harşit River in the Eastern Black Sea Region. Due to the heavy rainy climate of the dam site, the project was adopted as concrete face rockfill dam (Özkuzukıran, 2005).

The construction of the embankment was initiated in 1997 and paused for 1.5 years after the embankment was completed. This delay was not an inadvertant hault of schedule due to economic, politic or any other anticipated problem but a programmed pause in order to have the embankment complete post-construction settlements avoiding any prospective face cracking due to deformation of the underlying rockfill. The river valley is quite narrow and steep as given in Figure 3.13 (Özkuzukıran, 2005).

Figure 3.13 Upstream view of Kürtün Dam (Özkuzukıran, 2005)

Back analysis of the dam after operation concludes that, Kürtün Dam is successfully operating with anticipated trend of deformation foreseen during design stage and it is also noted that the arching effect of the narrow valley is quite noticable especially towards bottom of the valley due to steepening slopes in this region (Özkuzukıran, 2005).

Other than operating CFRDs there are several CFRD projects under design or construction stages in Turkey. The adoptation of CFRDs for various projects accelerated by successfull performance of Kürtün Dam, and it is followed by Atasu Dam (118 m) in Trabzon in 2002, Gördes Dam (95 m) in Manisa in 2004, Dim Dam (135 m) in Antalya in 2004, Marmaris Dam (49 m) in Muğla and Torul Dam (137 m) in 2007, majority of which are classified as high CFRDs.

None of them was recorded for severe face cracking or excessive leakage rate, indicating the experience of construction technique. Thus, CFRD has become a favorable dam type in Turkey due its well performance and especially due its economic benefit in case of non-availability of impervious material.

There are several CFRDs under construction or design, some of which are Yedigöze Dam (140 m) on the Seyhan River purposed for power generation, irrigation and water supply, Kandil Dam (rockfill, 106 m) and Gravel fill Sarıgüzel Dam (81.5 m) on the Ceyhan River in Kahramanmaraş designed for power generation.

CHAPTER 4

DESIGN CHARACTERISTICS OF CFRD ALTERNATIVE FOR GÖKÇELER DAM

4.1 General

Preliminary studies on the Gazipaşa Plain were initiated by issue of "Investigation Report of Gazipaşa Project" in 1961. Investigation studies for groundwater and other water resources of region involving Gazipaşa followed the initiative studies. Parallel to the development of the studies, construction of Gökçeler Dam was first proposed in "Gazipaşa II. Stage Project Gökçeler Dam and Irrigation Preliminary Report" which was published in 1993 (Hidro Dizayn, 2007-a).

As a result of progressive studies, "Gazipaşa II. Stage Project Gökçeler Dam and Irrigation Planning Report" was prepared by the XIII. District Office of State Hydraulic Works in 1998. Within this report, Gökçeler Dam was suggested to be Concrete Face Rockfill Dam with a height of the order of 100 m (Hidro Dizayn, 2007-a).

4.2 Location and Aim of the Project

Gökçeler Dam site is planned on the Gökçeler River within the Gazipaşa district of Antalya province in the Eastern Mediterrenean Basin in Turkey. The location of the project is given in Figure 4.1.

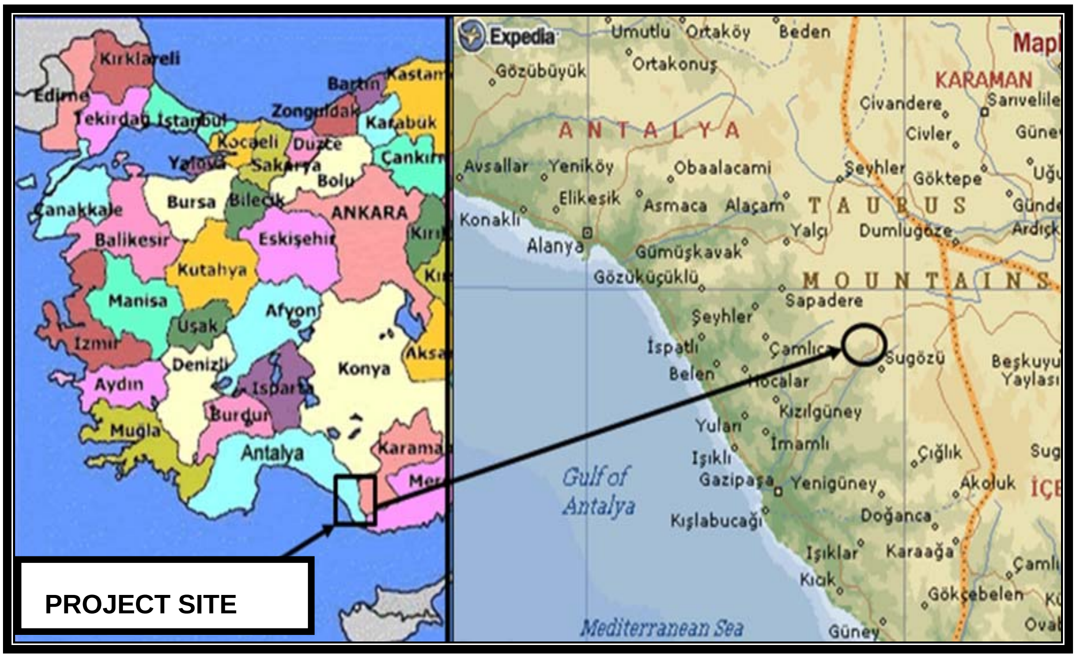

Figure 4.1 Location of Gökçeler Dam (Hidro Dizayn, 2007-b)

The aim of the project is to provide water both for irrigation of the Gazipaşa Plain which is one of the most productive agricultural lands of Turkey and for drinking, municipal and sanitary use of Gazipaşa town and surrounding villages.

For the above mentioned purposes Gökçeler Dam is proposed at 103.0 m riverbed elevation with a total height of 96.0 m upto 199.0 m crest elevation and crest length of 486.0 m. The proposed dam body placement across the river valley is demonstrated in Figure 4.2.

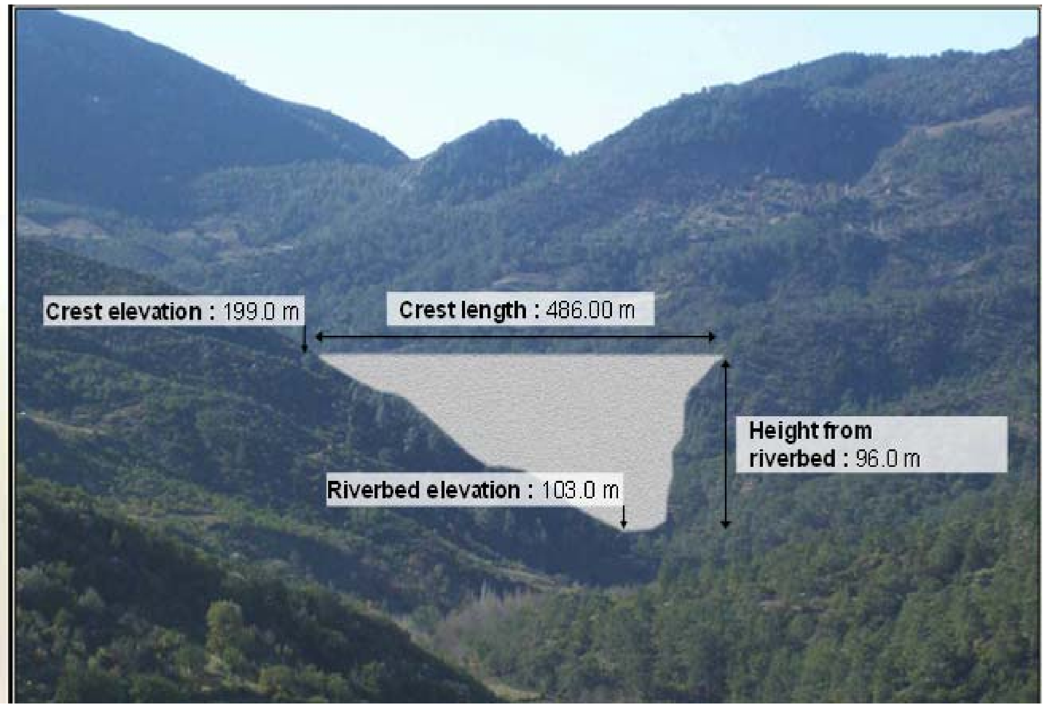

Figure 4.2 Proposed dam placement across Gökçeler River valley (Hidro Dizayn, 2007-b)

4.3 Properties of Dam Site

4.3.1 Topographic Characteristics

The project is located on mountainous terrain of the Eastern Taurus Mountains. The shape of the valley is mainly dependent on the topography of the region. The shape factor of Gökçeler Dam, which is formerly defined for Alto Anchicaya and Golillas Dams in Sections 3.2.4 and 3.2.6 respectively, is calculated to be 4.2. River valleys with shape factors less than or equal to 3 are accepted to be narrow and likely to exert arching effect on the embankment.

4.3.2 Geologic and Seismic Characteristics

Foundation of the dam site is mainly consisted of impervious to semi-pervious schists. Schist formation is mainly impervious but along weak zones impermeability decreases. Main rock formation is overlaid by alluvium along river bed and slope debris on the abutments. The depth of the alluvium on riverbed is 2.5 m at deepest section and maximum slope debris depth reaches 10.5 m on the left abutment. Weak formation within the upper 2-5 m of the main bedrock is suggested to be excavated along with the debris and alluvium which are permeable and very feeble (Hidro Dizayn, 2007-a). Geological layout of the dam site is given in Figure 4.3.

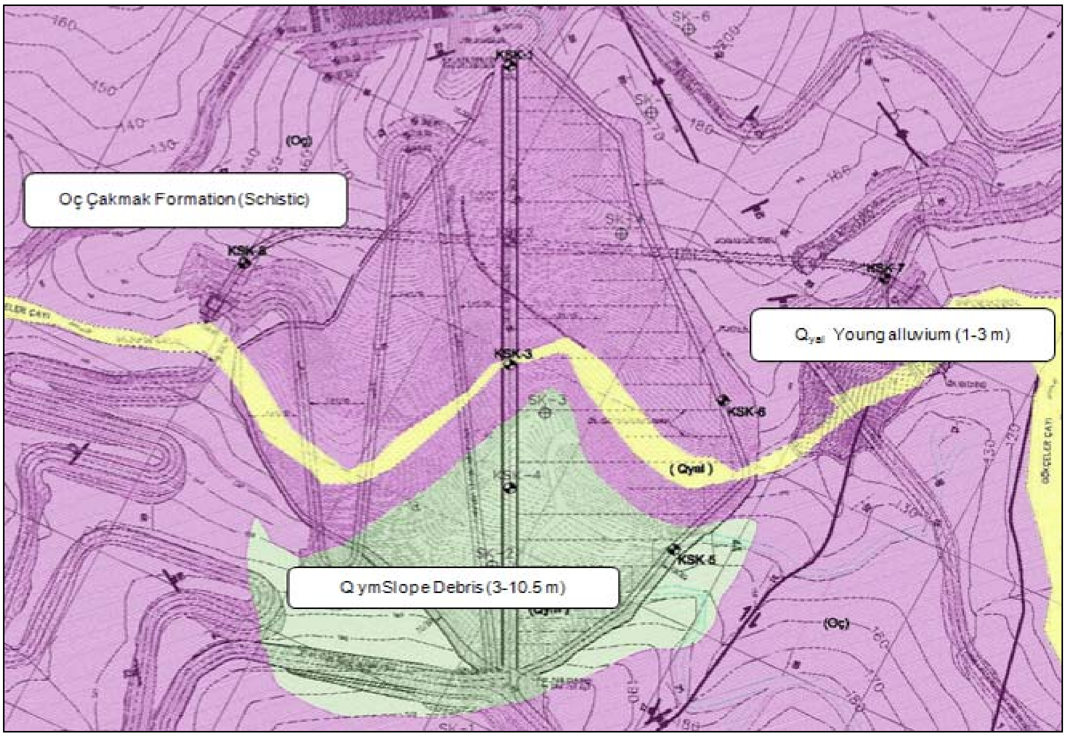

Figure 4.3 Geologic layout of Gökçeler Dam site (Hidro Dizyn, 2007-b)

The project site is located in the 4th degree earthquake zone according to the “Map of Earthquake Regions in Turkey” published by the Ministry of the Public Work and Settlement in 1996. Earthquake map of Antalya including project site is given in Figure 4.4.

For Gökçeler Dam site the Maximum Credible Earthquake, MCE, possible to take place within the region of interest, is selected to be the earthquake acceleration with 10% possibility of exceedence in 50 years of operation and determined as 0.10 g, while OBE, which is Operation Based Earthquake, is selected to be the earthquake acceleration with 50% possibility of exceedence in 100 years of operation and determined as 0.07 g (Hidro Dizayn, 2007-b).

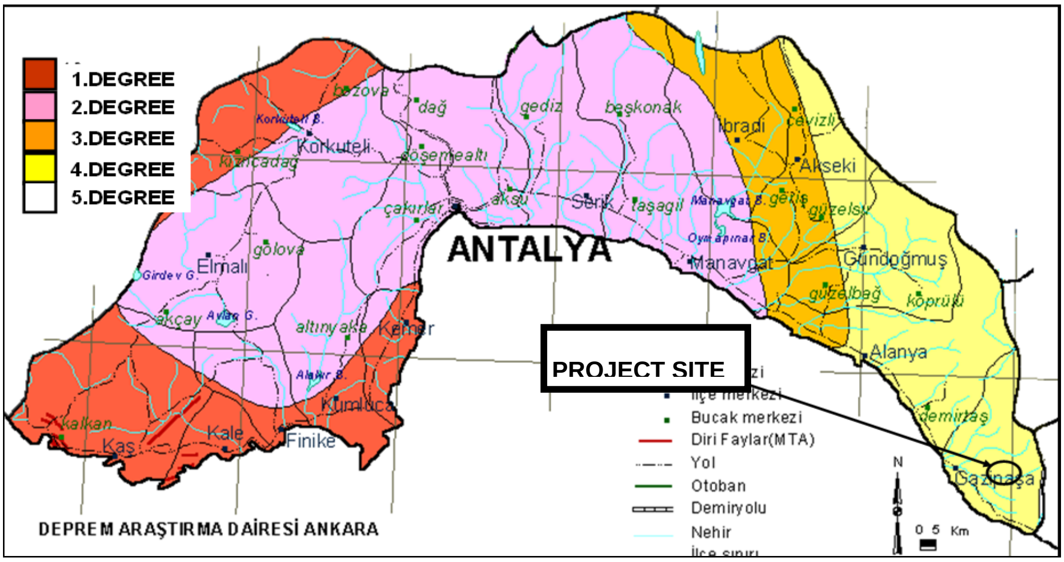

Figure 4.4 Earthquake map of Antalya (Hidro Dizayn, 2007-b)

Selection of rockfill for the embankment material is because of the presence of good quality rock quarries in vicinity of the dam site. However, for concrete face rockfill dams, in addition to the rock quarry material, excavation material of appurtenant structures, such as spillway and diversion tunnel can be used for the downstream coarser and more pervious parts of the embankment as mentioned in Chapters 2 and 3.

Layout plan of material source areas relating with the dam site and reservoir are given in Figures 4.5 and 4.6.

4.3.3 Climatic Characteristics

Mediterranean climate characteristics with hot, droughty summers and warm, rainy winters are observed at the project location.

Gazipaşa Plain located on the east of Antalya Bay, is shielded against atmospheric circulations by the embracing high mountains and consequently recieves less precipitation than its surrounding, on the contrary project site recieves high precipitation rates because of the orientation of the topography around dam site. Moisted air mass, trapped by Taurus Mountains leaves majority of the precipitation on the Mediterranean-side slopes of these mountains (Hidro Dizayn, 2007-a).

Project scheduling of the construction is likely to be interrupted due to the anticipated heavy rain if appropriate type is not selected. CFRD construction enables proceeding in the rainy season, but for ECRD, adequate scheduling must be executed in order to avoid any delays due to weather conditions. During long dry summer season with high temperature, it is important to sustain placement of concrete with intensive after-curing especially for massive concrete structures, such as roller compacted concrete type dam bodies.

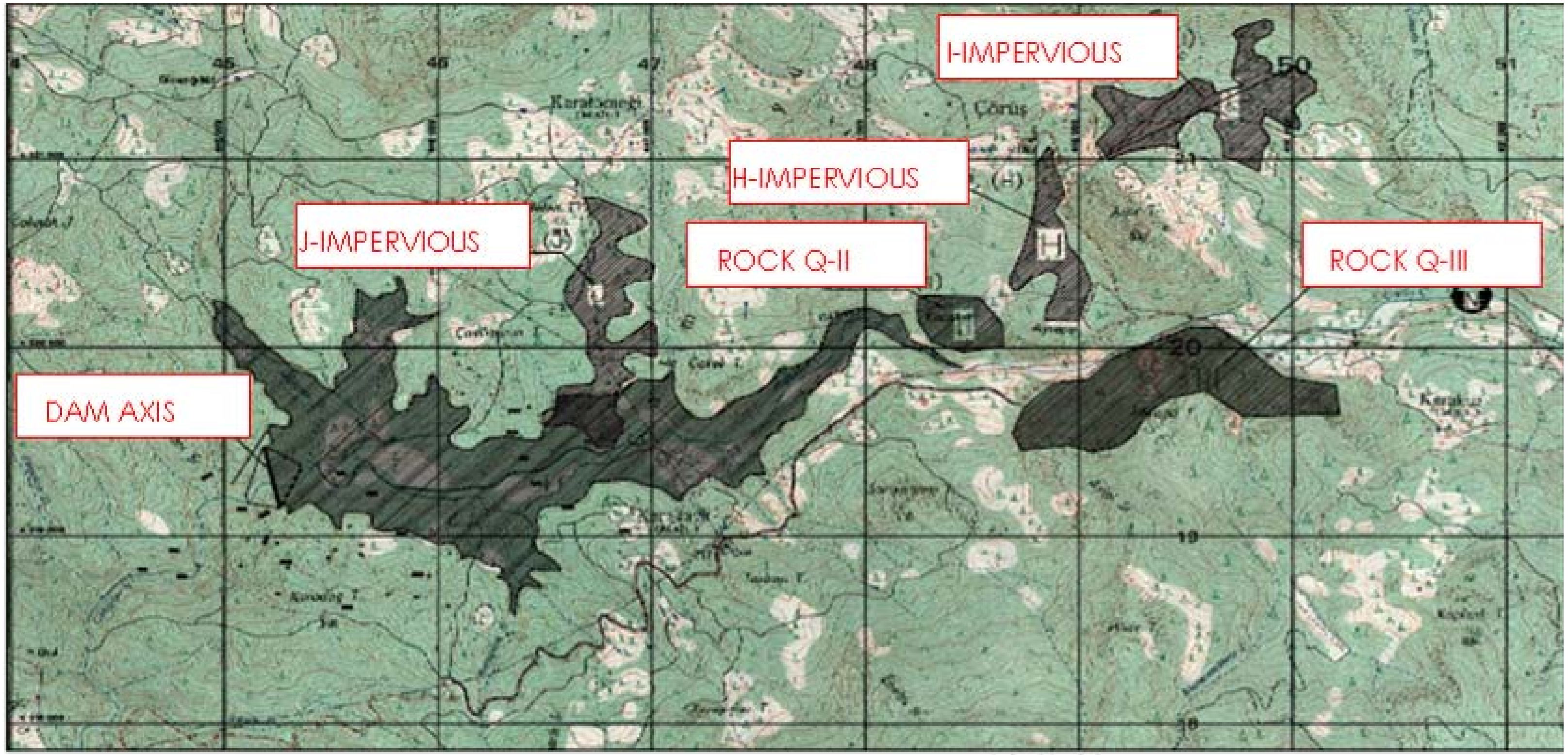

Figure 4.5 Layout of impervious barrow areas (H, I, J) and rock quarries (II, III)

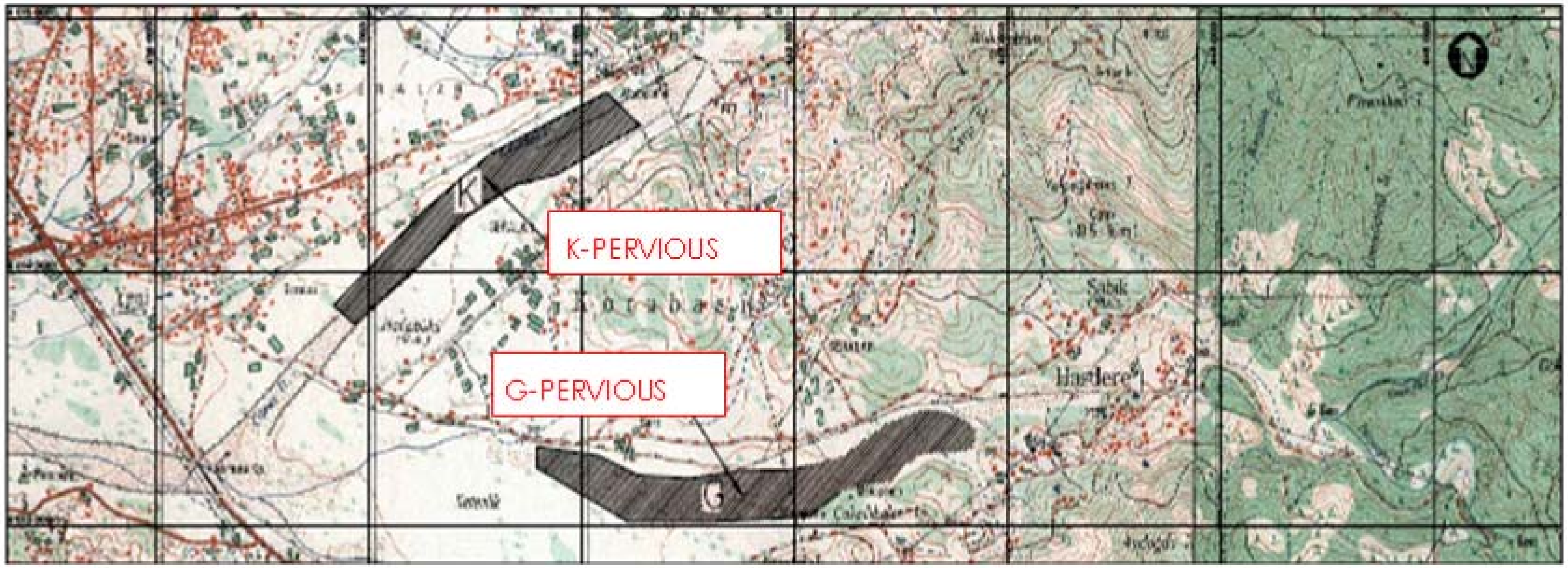

Figure 4.6 Layout of pervious barrow areas (G, K)

4.3.4 Meteorologic Characteristics

Adequate number of meteorology stations are available within the drainage basin. The average annual temperature value recorded by these stations is in the range of 11.5^{o}C~19.2 ^{o}C with maximum of 43.3^{o}C and minimum of -4.3^{o}C. And average annual precipitation value changes between 716.3 mm and 1081.7 mm. Maximum flood discharges with various return periods are given in Table 4.1 (Hidro Dizayn, 2007-b).

Table 4.1 Flood peak discharges of various return periods (Hidro Dizayn, 2007-b)

Drainage Area (km^2)	Method	Return Period (year)	Discharge (m^3/s)
128.9	State Hydraulic Works Synthetic Method	2	92.4
		5	130.3
		10	155.2
		25	186.3
		50	209.2
		100	232.1
		1000	308.1
		10000	384.2
		PMF(Probable Maximum Flood)	775.8

Maximum flood discharges with 10, 25 and 50 years return periods are taken into account for optimization of diversion facilities. ECRDs are very vulnerable against overtopping and passing flood through the embankment volume, thus factor of safety is higher for upstream cofferdam for this dam type in order to control the flood volumes of 25 and 50 years return periods, with and without freeboard, respectively. A lower upstream cofferdam capable to withstand flood wave of 10 years return period is designed for CFRD and RCC dam which are resistant against heavy weather conditions, overtopping or flood water passing through the body, as discussed in detail in Chapters 2 and 3.

The maximum probable flood discharge value is routed for spillway design of ECRDs and CFRDs, while reservoir routing of maximum flood discharge with 10000 years return period is used for optimization of RCC dams (Hidro Dizayn, 2007-a).

4.4 Design Characteristics of CFRD Alternative

General layout of the Concrete Face Rockfill Dam formulation is given in Figure 4.7. Dam body, cofferdams, spillway, diversion tunnel and valve chamber are involved in this formulation.

Design of Concrete Face Rockfill Dam alternative for Gökçeler Dam Project conforms to the recent design features of modern period. The plinth, face slab and embankment, directly related to the dam body, and other specific appurtenant structures are discussed in the following sections.

Hydraulic design of appurtenant structures, such as spillway, cofferdams and diversion tunnel are dependent on the performance of the dam body. Cofferdams and diversion tunnels are designed for small peak flood discharges, depending on good performance of precedent CFRDs for passing flood discharge through the dam body safely.

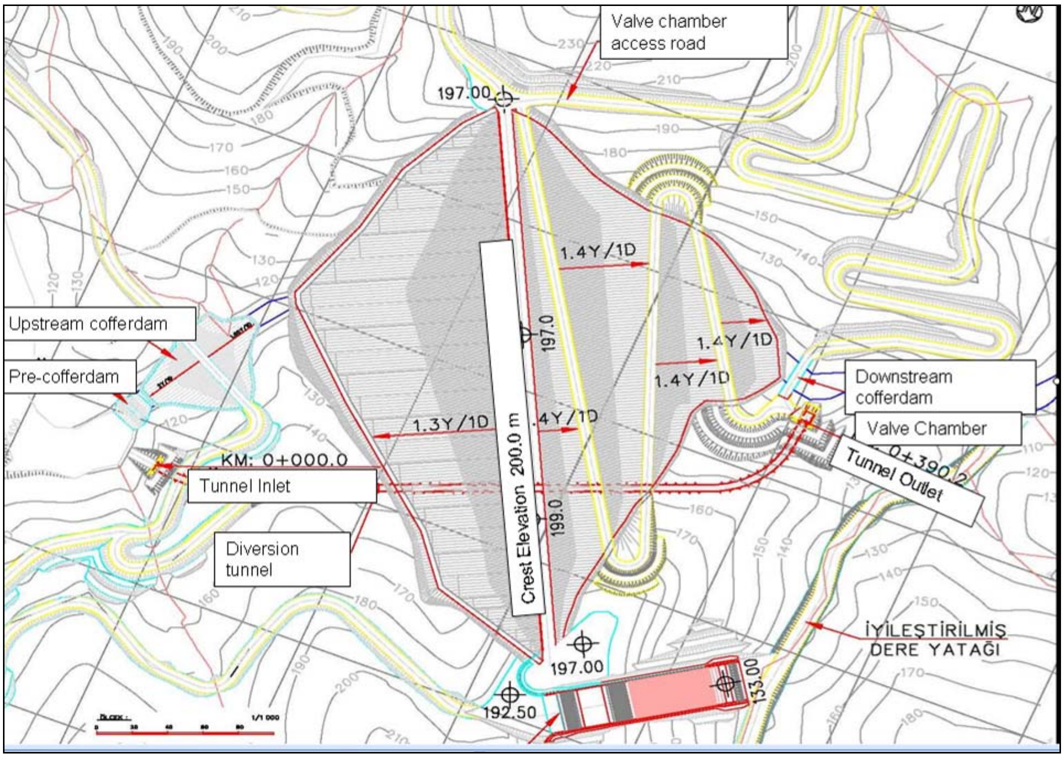

Figure 4.7 General layout of CFRD formulation (Hidro Dizayn, 2007-b)

The upstream and downstream slopes of dam body are 1.3 H : 1.0 Y and 1.4 H : 1.0 Y, respectively (Hidro Dizayn, 2007-a). In Section 2.3, the rockfill slopes are stated to range between 1 V : 1.3 H and 1 V : 1.4 which are close to the natural angle of repose of rock material.

Low quality impervious material can be used both for the impervious fill 1A zone of the dam body and earth core of the upstream embankment. Excavated rockfill material extracted from both spillway and diversion tunnel locations are used for the 3B zone of the main body satisfying the specifications of embankment zones for CFRD as discussed in detail in Chapter 2. The main cross-section of dam body is given in Figure 4.8 (Hidro Dizayn, 2007-a).

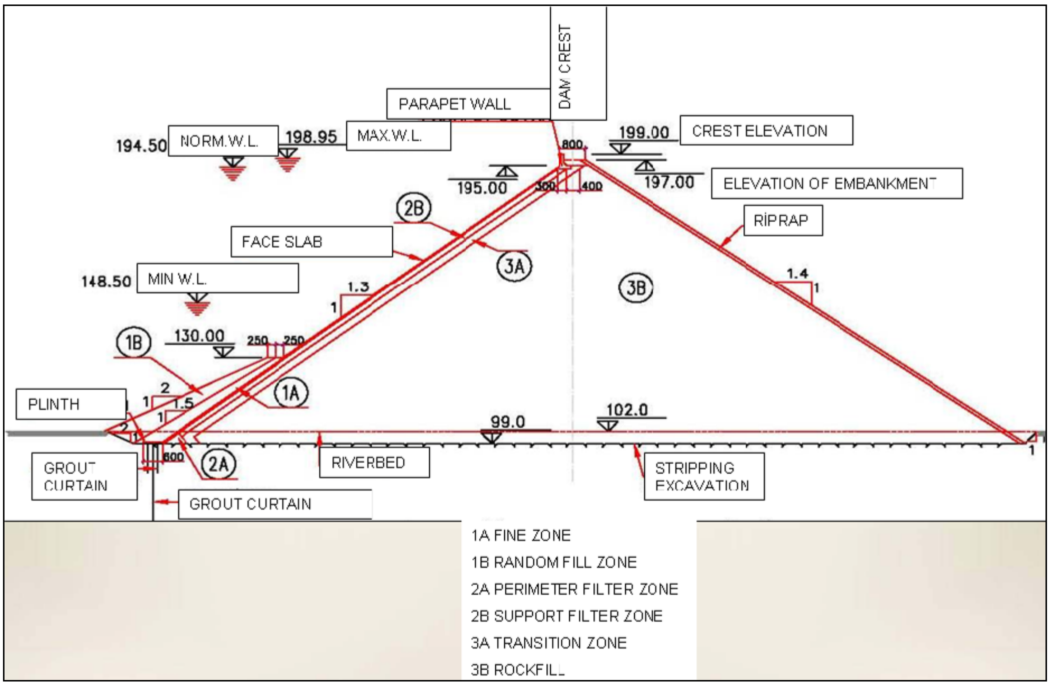

Figure 4.8 Main cross-section of CFRD type (Hidro Dizayn, 2007-b)

4.4.1 Plinth and Perimeter Joint

The plinth of Gökçeler Dam is consisting of internal and external parts, which save from the excavation volume, conforms to the modern design explained in Section 2.4.1. An external plinth of 4 m length is left and remaining plinth width is established as internal plinth underneath the rockfill. As a result of the appropriate geologic and topographic characteristics of the foundation along plinth alingment, high plinth sections or vertical plinth orientations are not required. Horizontal plinth cross-section and details of perimeter joint is given in Figure 4.9 (Hidro Dizayn, 2007-c).

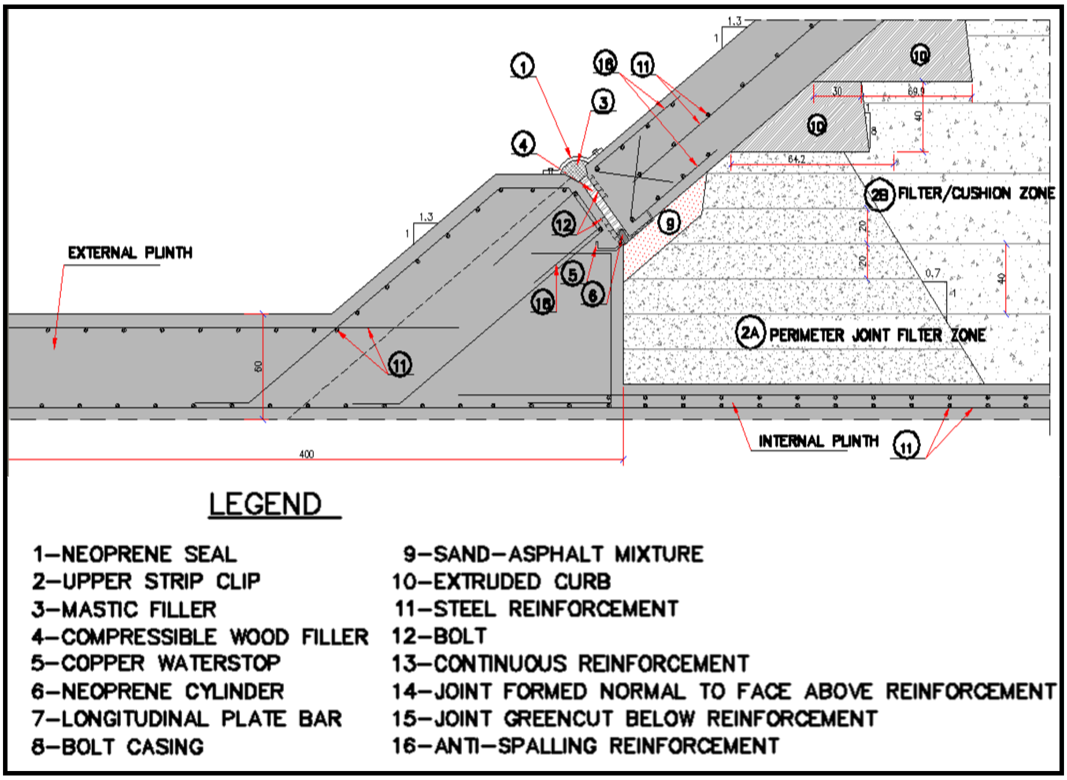

Figure 4.9 Horizontal plinth cross-section of Gökçeler Dam (Hidro Dizayn, 2007-c)

Perimeter joint of Gökçeler Dam is ornamented with two water barrier systems. Copper waterstop is located at the bottom of the joint and mastic filler reservoir is located on top of the joint. In the recent designs, fly ash reservoir has also been used as discussed in Section 2.4.2, but mastic filler is preferred because of the difficulties in cohesionless silt size material supply.

Plinth construction is independent of the dam body construction and other facilities on site. Only exception is riverbed plinth which has to follow diversion of river. The external plinth also serves as the grouting cap.Thus, foundation grouting may be initiated as soon as the construction of the external plinth is compeleted.

4.4.2 Face Slab and Vertical Joints

The face slab of the Gökçeler Dam consists of vertical panels forming either expansion or contraction vertical joints in between. Details of vertical expansion joints, designed similar to the perimeter joint, and vertical compression joints, established towards the center of the face slab, are given in Figures 4.10 and 4.11, respectively. Horizontal joints except construction joints are not established within the design of face slab. Details of horizontal construction joints are given in Figure 4.11 as well.

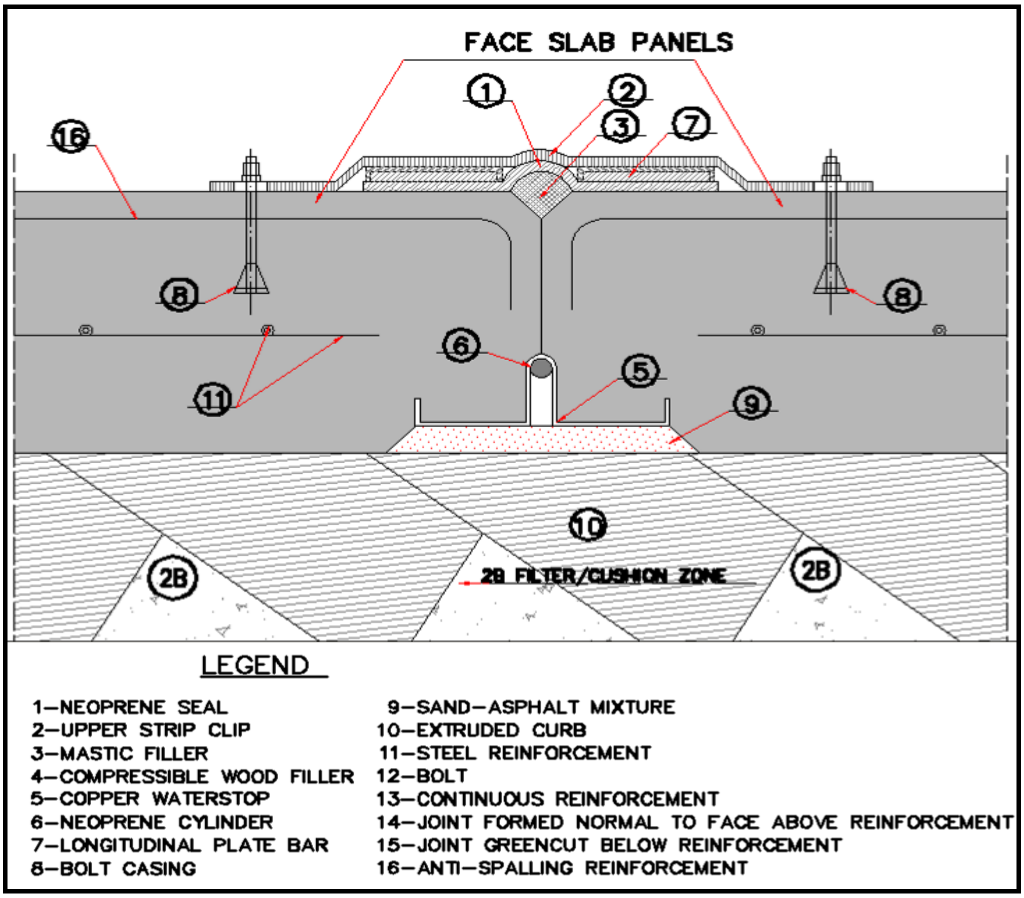

Figure 4.10 Details of vertical expansion joints of Gökçeler Dam (Hidro Dizayn, 2007-c)

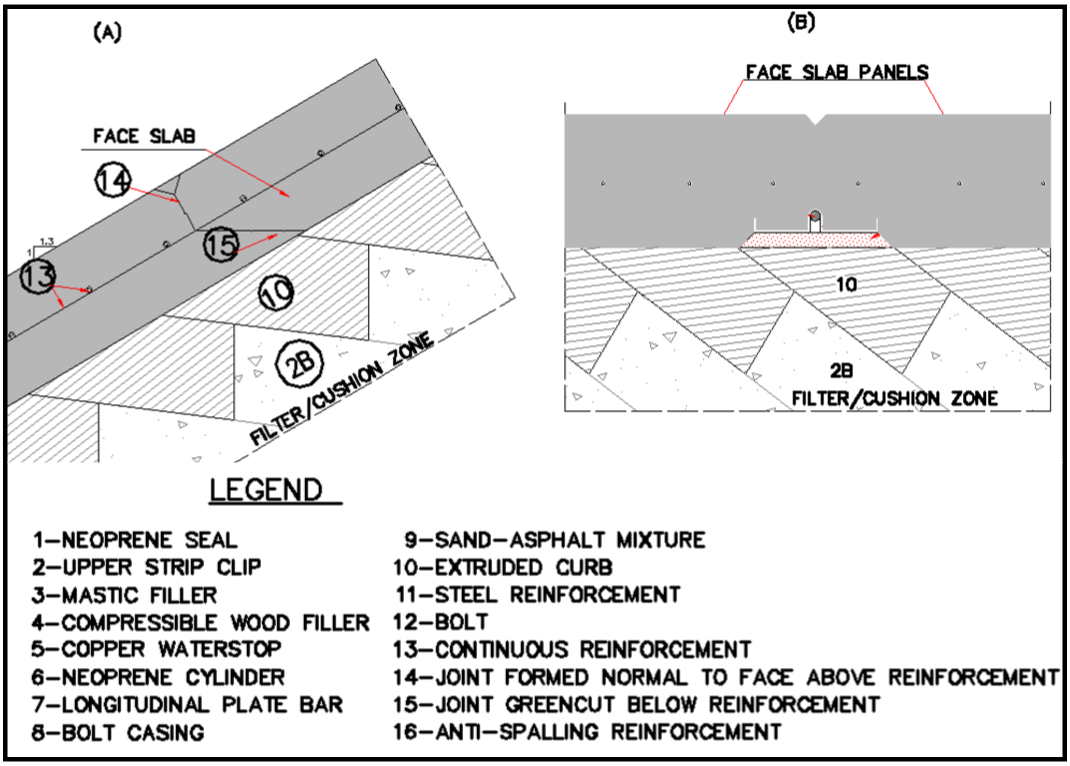

Figure 4.11 Horizontal construction and vertical contraction joints (Hidro Dizayn, 2007-c)

The reinforcement ratio of the face slab conforms to the reduced reinforcement ratios of modern design discussed in Section 2.3. In 10 m vicinity of the plinth reinforcement ratio is 0.40% in both horizontal and vertical direction and this ratio is further reduced down to 0.35% in vertical and 0.30% in horizontal direction elsewhere in the face slab (Hidro Dizayn, 2007-c).

Thickness of the face slab is 0.30 m at the parapet wall connection and increases upto 0.60 m at the plinth section which is calculated by the Equation (2.1) given in Section 2.4.3.

4.4.3 Embankment and Parapet Wall

Gökçeler Dam embankment zoning details are given in Figure 4.8. Extruded curbs are designed because of 2B support zone protection and other advantages stated in Section 2.4.5. The body has steeper upstream and downstream slopes compared to earth core rockfill dams. Foundation area of the dam body is significantly reduced as a result of steeper slopes.

Embankment volume is further reduced by the parapet wall designed on the upstream side of the crest, details of which are given in Figure 4.12 (Hidro Dizayn, 2007-c). The parapet wall designed for Gökçeler Dam is 5.0 m high which is dependent on the good performance of precedent CFRDs as given in Sections 2.4.4.

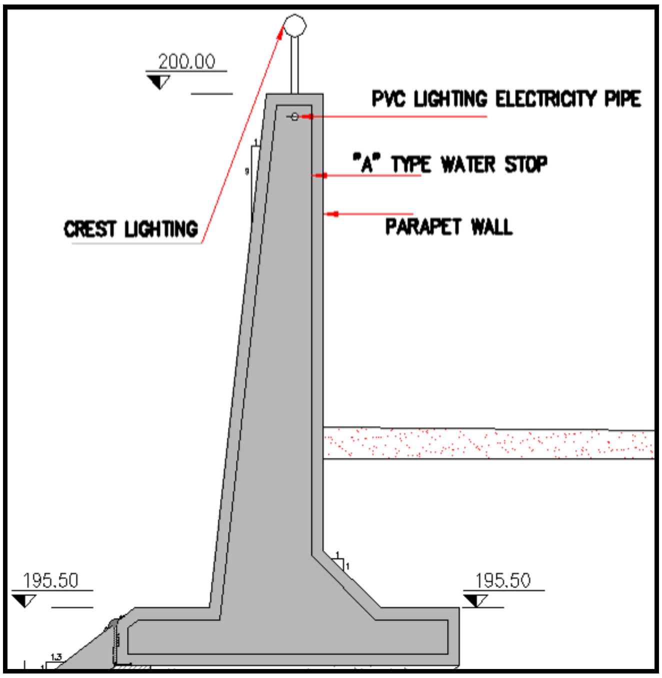

Figure 4.12 Details of parapet wall for Gökçeler Dam

4.4.4 Upstream Cofferdam, Diversion Tunnel and Spillway

The upstream cofferdam is designed with a height of 24 m considering peak flood discharge with 10 years return period. In case of higher flood volumes occurence during construction, the priority section of the CFRD embankment is likely to pass through the excessive flood volume without serious problems at the dam site, as discussed in Chapter 2.
Its type is earth core rockfill type but it is lower compared to the upstream cofferdam of the ECRD alternative which is designed considering peak flood discharges with 25 and 50 years return periods.The amount of impervious material is ignorable compared to the impervious material amount required for ECRD formulation (Hidro Dizayn, 2007-c).

The diversion tunnel is designed to divert 145 m^3/s safely. Its has a circular cross-section with an inner diameter of 4.0 m. Diversion tunnel length is dependent on the general layout of the project formulation. Its length is shortened as the foundation area of the dam body is reduced.

The dimensions and the layout of the spillway are given in Figure 4.7 (Hidro Dizayn, 2007-a). It is discharging to the side branch of the Gökçeler River in order to save from excavation volume. And spillway of the Gökçeler Dam is designed with a stepped chute channel by routing maximum probable flood discharge conforming to the common practice of embankment dam design.

CHAPTER 5

TYPE SELECTION FOR GÖKÇELER DAM PROJECT

5.1 General

In order to conduct type selection for Gökçeler Dam Project total investment costs, annual expense / irrigation benefits, internal rate of returns are determined and a comparative study is conducted between CFRD formulation and other alternatives which are ECRD and RCC formulations. All economic calculations are executed depending on a specially Defined Unit Cost (DUC), in order to avoid working any specific unit which may lead to inconsistencies in future.

Defined Unit Cost (DUC) is determined considering the total of the cost of 1 hour operation of excavation equipment, 1 man-hour cost of excavator operator and 1 man-hour costs of 2 labors. This selection is dependent on the basic requirements of construction procedure on the site.

After DUC is determined, basic stages of construction, such as excavation of impervious material or placement of filter material, are separately analysed and unit prices are determined individually. Summary of these construction stages are given in Table 5.1 with unit price codes, explanation of the work performed, unit of the work and the corresponding unit prices.

The details of the Unit price analysis conducted for each of the construction stages are given in Appendix A.

Table 5.1 Summary of specified unit prices

CODE	DEFINITON OF THE UNIT PRICE	UNIT	UNIT PRICE (DUC)
GKL-01	Excavation of pervious (except rock) and impervious foundation material and haulage for 1 km.	m^3	3.24
GKL-02	Excavation of rock foundation and haulage of rock for 1 km.	m^3	11.25
GKL-03	Preparation of embankment foundation for placement of fill material.	m^3	7.93
GKL-04	Extraction of impervious fill material from barrow areas, placement within the fill and haulage for 3 km.	m^3	6.37
GKL-05	Extraction of pervious fill material from barrow areas, placement within the fill and haulage for 23 km.	m^3	12.51
GKL-06	Extraction of rockfill material from quarries, placement within the fill and haulage for 2.5 km.	m^3	12.77
GKL-07	Placement of excavated pervious or impervious found. material within embnk. and haulage for 1 km.	m^3	2.25
GKL-08	Placement of excavated rock foundation within the rockfill and haulage for 1 km.	m^3	2.66
GKL-09	Preparation of filter material and haulage for 23 km.	m^3	18.80
GKL-10	Sluicing and compaction of pervious material (except rock)	m^3	0.69
GKL-11	Sluicing and compaction of impervious material.	m^3	0.85
GKL-12	Sluicing and compaction of rockfill material.	m^3	0.58
GKL-13	Placement of surface protection from rockfill and haulage for 2.5 km.	m^3	16.19
GKL-14	Preparation of aggregate mixed in concrete mortar and haulage for 23 km .	m^3	9.94
GKL-15	Supply of cement mixed in concrete mortar and haulage for 199 km.	ton	182.70
GKL-16	Supply of construction steel and haulage for 499 km.	ton	1,507.49
GKL-17	Preparation and placement of concrete	m^3	294.11
GKL-18	Grouting of every kind and class of formation	m	161.04
GKL-19	Preparation and placement of roller compacted concrete	m^3	46.53

5.2 Estimated Cost of Facilities

Using above mentioned specified unit prices, estimated costs of diversion tunnel, cofferdams, spillway, dam body and grouting are calculated. Cost of irrigation and drainage facilities are also taken into account because irrigation benefits will be used for the following calculations but included in the cost calculations in the final stages without analysing the individual facilities.

Estimated costs of each facilities and total estimated cost of concrete face rockfill dam, CFRD, formulation are given between Tables 5.2 and 5.7.

Estimated total cost analysis are also conducted for earth core rockfill dam, ECRD, and roller compacted concrete, RCC, formulations. Summary of estimated cost analysis are given in Tables 5.8 and 5.9 for earth core rockfill dam and roller compacted concrete dam formulations, respectively.

Details of estimated cost analysis for each of the facilities within earth core rockfill dam formulation are given between Tables B.1~B.5 in Appendix B, while estimated cost tables of roller compacted concrete dam formulation are given between Tables C.1~Table C.5 in Appendix C.

General layout plans and maximum dam body cross-sections are also supplied in the corresponding Appendices for these formulations.

Table 5.2 Estimated cost analysis of cofferdams

ESTIMATED COST ANALYSIS TABLE FOR CFRD PRE-COFFERDAM, UPSTREAM AND DOWNSTREAM COFFERDAMS					
UNIT PRICE CODE	DEFINITION OF THE WORK	QUANTITY	UNIT	UNIT PRICE (DUC)	TOTAL (DUC)
GKL-08	Placement of excavated rock foundation within the rockfill and haulage for 1 km . (For pre-cofferdam)	1,100	m^3	2.66	2,931
GKL-08	Placement of excavated rock foundation within the rockfill and haulage for 1 km . (For upstream cofferdam)	15,700	m^3	2.66	41,835
GKL-12	Sluicing and compaction of rockfill material.	16,800	m^3	0.58	9,757
GKL-04	Extraction of impervious fill material from barrow areas, placement within the fill and haulage for 3 km.	5,700	m^3	6.37	36,295
GKL-11	Sluicing and compaction of impervious material.	5,700	m^3	0.85	4,859
GKL-09	Preparation of filter material and haulage for 23 km.	3,350	m^3	18.80	62,972
GKL-10	Sluicing and compaction of pervious material (except rock)	3,350	m^3	0.69	2,314
GKL-14	Preparation of aggregate mixed in concrete mortar and haulage for 23 km . (For downstream coff.dam)	1,375	m^3	9.94	13,664
GKL-15	Supply of cement mixed in concrete mortar and haulage for 199 km. (For downstream cofferdam)	330	ton	182.70	60,292
GKL-16	Supply of construction steel and haulage for 499 km. (For downstream cofferdam)	33	ton	1,507.49	49,747
GKL-17	Preparation and placement of concrete. (For downstream cofferdam)	1,100	m^3	294.11	323,521
		SUB TOTAL =	608,187		
TOTAL ESTIMATED COST OF COFFERDAMS (DUC) = 608,187					

Table 5.3 Estimated cost analysis of diversion tunnel

UNIT PRICE CODE	DEFINITION OF THE WORK	QUANTITY	UNIT	UNIT PRICE (DUC)	TOTAL (DUC)
ESTIMATED COST ANALYSIS TABLE FOR CFRD DIVERSION TUNNEL (L=385 m)					
GKL-02	Excavation of rock foundation and haulage of rock for 1 km.	7,950	m^3	11.25	89,459
	Increase of excavation cost by 1% per 100 m of tunnel length exceeding 300 m	1		0.01	760
GKL-14	Preparation of aggregate mixed in concrete mortar and haulage for 23 km .	3,813	m^3	9.94	37,887
GKL-15	Supply of cement mixed in concrete mortar and haulage for 199 km.	915	ton	182.70	167,173
GKL-16	Supply of construction steel and haulage for 499	244	ton	1,507.49	367,826
GKL-17	Preparation and placement of concrete	3,050	m^3	294.11	897,036
GKL-19	Preparation and placement of roller compacted concrete	1,950	m	161.04	314,027
		SUB TOTAL =		1,874,169	

TOTAL ESTIMATED COST OF DIVERSION TUNNEL (DUC) = 2,342,711

Table 5.4 Estimated cost analysis of grouting

ESTIMATED COST ANALYSIS TABLE FOR CFRD GROUTING					
UNIT PRICE CODE	DEFINITION OF THE WORK	QUANTITY	UNIT	UNIT PRICE (DUC)	TOTAL (DUC)
GKL-18	Grouting of every kinds and classes of formation	8,400	m	161.04	1,352,734
			SUB TOTAL =	1,352,734	
TOTAL ESTIMATED COST OF GROUTING (DUC) = 1,352,734					

Table 5.5 Estimated cost analysis of dam body

ESTIMATED COST ANALYSIS TABLE FOR CFRD DAM BODY					
NIT PRICE CODE	DEFINITION OF THE WORK	QUANTITY	UNIT	UNIT PRICE (DUC)	TOTAL (DUC)
GKL-03	Preparation of embankment foundation for placement of fill material.	184,500	m^3	7.93	1,463,395
GKL-08	Placement of excavated rock foundation within the rockfill and haulage for 1 km . (From tunnel and spillway excavation for 3C Zone and some portion	34,750	m^3	2.66	92,596
GKL-06	Extraction of rockfill material from quarries, placement within the fill and haulage for 2.5 km. (For remaining portion of 3B Zone and 3A Zone)	1,710,250	m^3	12.77	21,835,617
GKL-12	Sluicing and compaction of rockfill material.	1,745,000	m^3	0.58	1,013,438
GKL-07	Placement of excavated pervious or impervious foundation material within embankment and haulage for 1 km. (1A and 1B Zones)	105,000	m^3	5.77	605,596
GKL-11	Sluicing and compaction of impervious material.	105,000	m^3	0.85	89,513

Table 5.5 Estimated cost analysis of dam body (Continued)

GKL-09	Preparation of filter material and haulage for 23 km (For 2A and 2B Zones)	50,000	m^3	18.80	939,880
GKL-10	Sluicing and compaction of pervious material (except rock)	50,000	m^3	0.69	34,530
GKL-13	Placement of surface protection from rockfill and haulage for 2.5 km.	25,000	m^3	16.19	404,625
GKL-14	Preparation of aggregate mixed in concrete mortar and haulage for 23 km .	31,875	m^3	9.94	316,761
GKL-15	Supply of cement mixed in concrete mortar and haulage for 199 km.	7,650	ton	182.70	1,397,677
GKL-16	Supply of construction steel and haulage for 499 km.	127	ton	1,507.49	191,451
GKL-17	Preparation and placement of concrete (For handrail poles on the crest)	25,500	m^3	294.11	7,499,805
		SUB TOTAL =		35,884,882	
TOTAL COST OF DAM BODY(DUC) = 35,884,882					

Table 5.6 Estimated cost analysis of spillway

ESTIMATED COST ANALYSIS TABLE FOR CFRD SPILLWAY					
UNIT PRICE CODE	DEFINITION OF THE WORK	QUANTITY	UNIT	UNIT PRICE (DUC)	TOTAL (DUC)
GKL-01	Excavation of pervious (except rock) and impervious foundation material and haulage for 1	58,500	m^3	3.24	189,784
GKL-02	Excavation of rock foundation and haulage of rock for 1 km.	25,000	m^3	11.25	281,317
GKL-13	Placement of surface protection from rockfill and haulage for 2.5 km.	190	m^3	16.19	3,075
GKL-14	Preparation of aggregate mixed in concrete mortar and haulage for 23 km .	8,500	m^3	9.94	84,470
GKL-15	Supply of cement mixed in concrete mortar and haulage for 199 km.	2,040	ton	182.70	372,714
GKL-16	Supply of construction steel and haulage for 499	408	ton	1,507.49	615,054
GKL-17	Preparation and placement of concrete	6,800	m^3	294.11	1,999,948
		SUB TOTAL =	3,546,361		

TOTAL ESTIMATED COST OF SPILLWAY (DUC) = 3,546,361

Table 5.7 Summary of estimated total cost of CFRD formulation

SUMMARY OF TOTAL ESTIMATED COSTS CONCRETE FACE ROCKFILL DAM FORMULATION		
	NAME OF THE FACILITY	TOTAL COST (DUC)
1	COFFERDAMS	608,187
2	DIVERSION TUNNEL	2,342,711
3	SPILLWAY	3,546,361
4	DAM BODY	35,884,882
5	GROUTING	1,352,734
TOTAL ESTIMATED COST OF FORMULATION (DUC) = 43,734,874		

Table 5.8 Summary of estimated total cost of ECRD formulation

SUMMARY OF TOTAL ESTIMATED COSTS EARTH CORE ROCKFILL DAM FORMULATION		
	NAME OF THE FACILITY	TOTAL COST (DUC)
1	COFFERDAMS	451,119
2	DIVERSION TUNNEL	2,712,859
3	SPILLWAY	3,546,361
4	DAM BODY	35,633,398
5	GROUTING	1,449,357
TOTAL ESTIMATED COST OF FORMULATION (DUC) = 43,793,095		

Table 5.9 Summary of estimated total cost of RCC formulation

SUMMARY OF TOTAL ESTIMATED COSTS ROLLER COMPACTED CONCRETE DAM FORMULATION		
	NAME OF THE FACILITY	TOTAL COST (DUC)
1	COFFERDAMS	751,965
2	DIVERSION TUNNEL	621,240
3	SPILLWAY	5,986,244
4	DAM BODY	40,350,515
5	GROUTING	1,401,045
TOTAL ESTIMATED COST OF FORMULATION (DUC) = 49,111,009		

5.3 Preparation of Work Schedule

Work schedule is prepared considering the capacity of an average construction site and assuming 2 shifts of 8 working hours a day, 30 days a month without delays for national holiday durations. Construction work schedule for CFRD formulation is given in Figure 5.1.

Construction of diversion tunnels are started before initiation of cofferdam construction. Block lengths of tunnel is 6 m for Gökçeler Dam diversion tunnel. On the assumption of construction of 1 block per day, diversion tunnel is completed in 2 months.

An ordinary truck used in dam construction sites has a capacity of 10 m^3. Cofferdam construction is assumed to be started as the excavation of the tunnel is completed which corresponds to 1 month later than the initiation of the tunnel construction and completed in 4 months by employment of 10-15 trucks.

For CFRD embankment construction, considering the location of material barrow areas and rock quarries it is assumed to take an average of 1 hour for loading, reloading and haulage from source areas. It is also accepted that placement and compaction of material has no delaying effect on the overall construction duration. On assumption of employment of a total number of 20 trucks, daily embankment construction capacity of the site is calculated as 3200 m^3/day. Thus, 1925000 m^3 of total embankment volume is completed in 20 months without any delays. In case of any unavoidable delay, capacity is increased and work schedule is completed in suggested date.

Construction of dam body is initiated before completion of cofferdams and diversion of river as a consequence of discussion in Section 2.4.5 on availability of staged construction of embankment.

One of the most significant feature of CFRD scheduling is the independency of grouting application from dam body construction. Plinth construction and grouting application is executed apart from the dam body scheduling.

For Gökçeler Dam site, discharge of spillway is designed to be on branch of the Gökçeler River. Hence, excavation and other constructional activities are not affecting the dam body construction. Spillway construction is assumed to be finished in 11 months.

Irrigation and drainage facilities are generally constructed on a very wide surface area apart from the dam site, and it is assumed not to be affecting the overall construction period and completed within the construction duration of dam body and appurtenant structures.

Construction work schedules of ECRD and RCC formulations are also prepared considering the same factors and given in Figures 5.2 and 5.3, respectively.

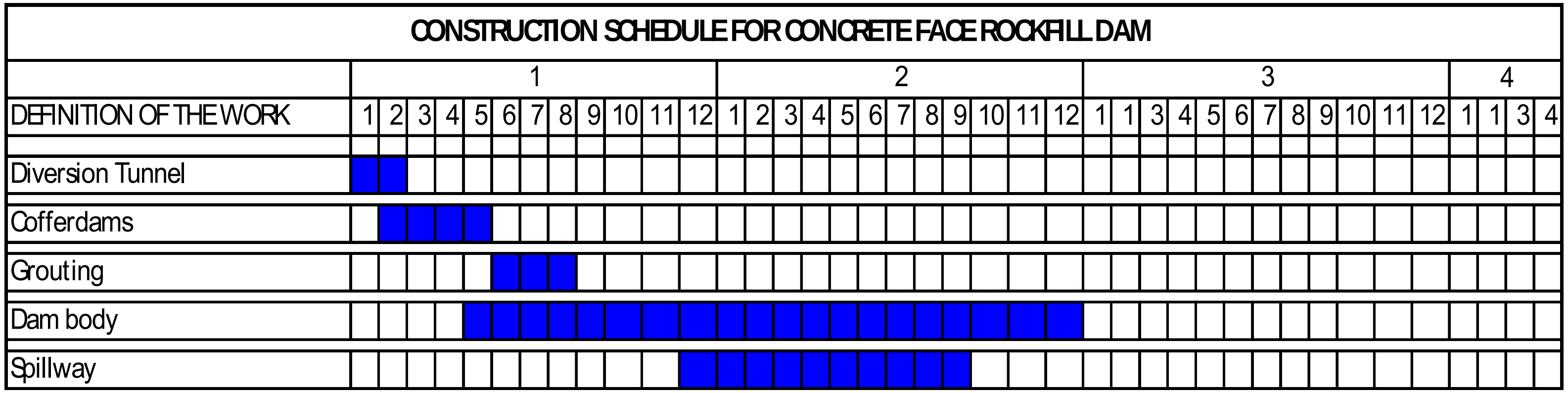

Figure 5.1 Construction shcedule for CFRD formulation

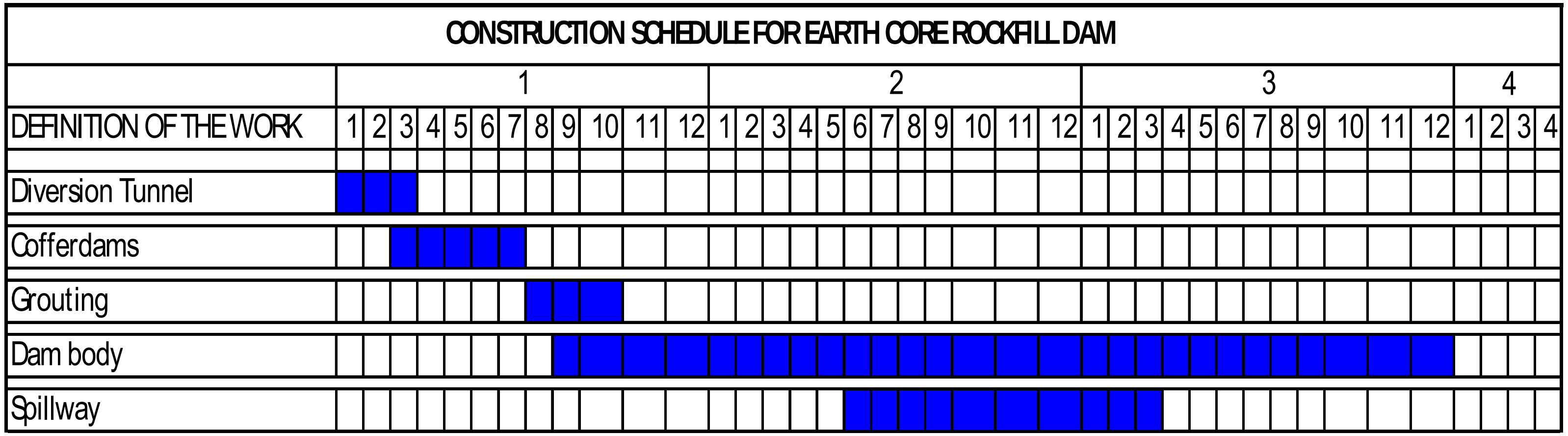

Figure 5.2 Construction shcedule for ECRD formulation

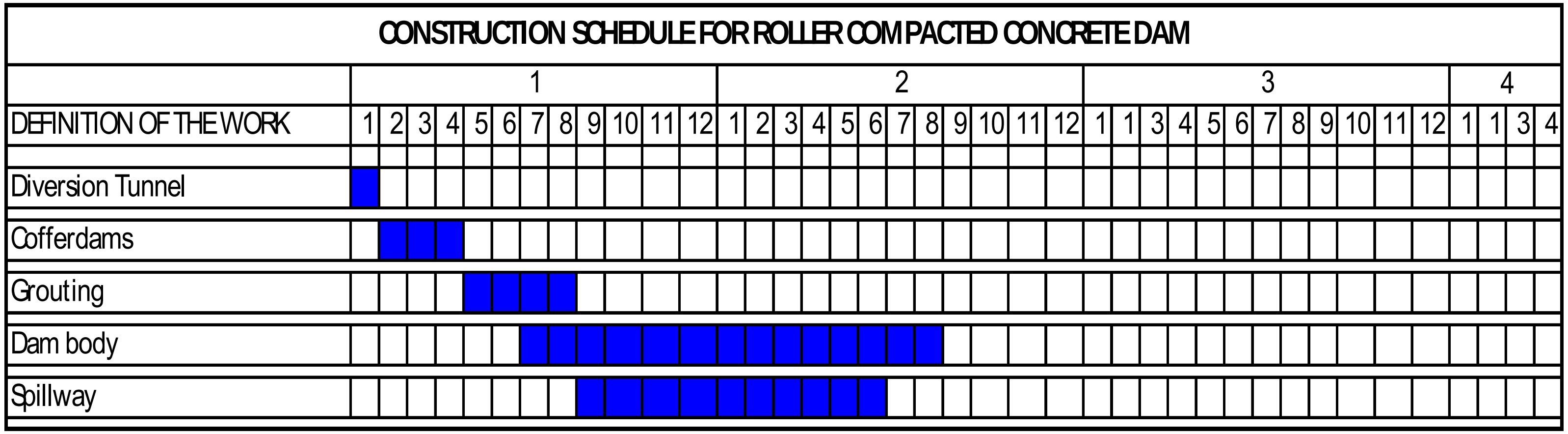

Figure 5.3 Construction shcedule for RCC Dam formulation

5.4 Total Investment Cost

Duration of the overall construction is not the same for different project formulations, but the irrigation benefits will be the same because of the constant value of irrigation area. Total investment cost of the formulation is calculated in order to calculate the internal rate of return for better comparison between studied formulations. Total investment cost analysis calculations are given in Table 5.10.

Construction duration is divided into 4 periods each of which represents "6 months" of total duration. In the first 6 rows, fractions of estimated costs of facilities corresponding to the "6 months" periods are calculated individually. Irrigation and drainage facilities are accepted to be finished within the construction period and their constant estimated cost are divided into 4 equal fractions. Total estimated costs corresponding to each of these "6 months" periods are also calculated within the table. Construction costs of each period is calculated by adding contingency costs to estimated costs. Contingency costs are assumed to be 15% of the total estimated cost while project control costs are assumed to be 15% of the construction cost as a common trend of State Hydraulic Works applications. The unit price for expropriation is taken as 5 DUC/m^2. Expropriating costs are included within the first "6 months" period of the schedule because expropriating has to be handled before the initiation of the construction work on the site.

Project cost is calculated by adding, construction cost, project control costs and expropriating costs.

Table 5.10 Total investment cost of CFRD formulation

INTEREST RATE : **%5.00**

		TOTAL	1. "6 Months"	2. "6 Months"	3. "6 Months"	4. "6 Months"
COFFERDAMS	(1)	608 187	608 187	-	-	-
DIVERSION TUNNEL	(2)	2 342 711	2 342 711	-	-	-
SPILLWAY	(3)	3 546 361	-	354 636	2 127 817	1 063 908
DAM BODY	(4)	35 884 882	3 588 488	10 765 465	10 765 465	10 765 465
GROUTING	(5)	1 352 734	450 911	901 822	-	-
IRRIGATION AND DRAINAGE FACILITIES	(6)	29 966 247	7 491 562	7 491 562	7 491 562	7 491 562
ESTIMATED COST	**(7)=(1)+...+(6)**	**73 701 122**	**14 481 859**	**19 513 485**	**20 384 843**	**19 320 935**
CONTINGENCY	(8)=(7)*0.15	11 055 168	2 172 279	2 927 023	3 057 726	2 898 140
CONSTRUCTION COST	**(9)=(7)+(8)**	**84 756 290**	**16 654 138**	**22 440 508**	**23 442 570**	**22 219 075**
PROJECT CONTROL	(10)=(9)*0.15	12 713 443	2 498 121	3 366 076	3 516 385	3 332 861
EXPROPRIATING	(11)	922 500	922,500			
PROJECT COST	**(12)=(9)+(10)+(11)**	**98 392 233**	**20 074 758**	**25 806 584**	**26 958 955**	**25 551 936**
INTEREST DURING CONSTRUCTION	(13)	5 996 107	2 057 663	1 959 490	1 347 948	631 007
PERIODICAL CUMMULATIVE DEBT	(14)		22 132 421	49 898 494	78 205 397	104 388 341
TOTAL INVESTMENT COST	**(15)=(12)+(13)**	**104 388 341**	**22 132 421**	**27 766 073**	**28 306 903**	**26 182 943**

Annual interest rate is taken as 5% which is defined by State Hydraulic Works for projects with irrigation purposes. Interest values of project cost during construction are calculated according to Equation (5.1) and using 5% interest rate.

$$i = C_p\left(1.05^n - 1\right) \tag{5.1}$$

where i is interest value, C_p is project cost and n is construction period

Total investment costs are calculated by adding project cost and interest values during construction period. Calculation of total investment costs for ECRD and RCC formulations are given in Appendices B and C, respectively.

5.5 Internal Rate of Return

In order to calculate Internal rate of return, firstly annual project expense is determined. Using the pre-determined annual project expense value, rantability is calculated.

Total annual project expense calculation for CFRD formulation is given in Table 5.11 in detail. Interest and amortization factors, renewal factors and operation and maintenance factors of facilities are based on the values determined by State Hydraulic Works. Details of rantability calculation for CFRD formulation is given in Table 5.12.

Annual Project Expenses, Rantability of ECRD and RCC formulations are also calculated using the same multiplication factors because the type of the facilities are the same with CFRD formulation. Details of calculations for these formulations are given in Appendices B and C, respectively.

Table 5.11 Annual project expense of CFRD formulation

INTEREST RATE : %5.00

ALL UNITS ARE IN DUC

No.	TYPE OF WORK	CONST. COST	CONTINGENCY	COST	INVESTMENT COST	INTEREST & AMORT. FACTOR	RENEWAL FACTOR	OPERATION & MAINTENANCE FACTOR	ANNUAL COST INTEREST & AMORT.	RENEWAL	OPERATION & MAINTEN.	TOTAL
		(DUC)							(DUC)			
1	COFFERDAMS	608 187	91 228	699 415	742 038	0.05478	0.0001300	0.002000	40 649	91	1 399	42 139
2	TUNNEL	2 342 711	351 407	2 694 118	2 858 299	0.05478	0.0001300	0.005000	156 578	350	13 471	170 398
3	SPILLWAY	3 546 361	531 954	4 078 315	4 326 851	0.05478	0.0001300	0.010000	237 025	530	40 783	278 338
4	DAM BODY	35 884 882	5 382 732	41 267 614	43 782 498	0.05478	0.0001300	0.002000	2 398 405	5 365	82 535	2 486 305
5	GROUTING	1 352 734	202 910	1 555 644	1 650 446	0.05478	0.0001300	0.020000	90 411	202	31 113	121 727
6	IRRIGATION AND DRAINAGE FACILITIES	29 966 247	4 494 937	34 461 184	36 561 279	0.05478	0.0032300	0.019900	2 002 827	111 310	685 778	2 799 914
7	**CONSTRUCTION COST**			**84 756 290**					**4 925 895**	**117 848**	**855 078**	**5 898 821**
8	PROJECT CONTROL			12 713 443	13 488 212	0.05478	-		738 884			738 884
9	EXPROPRIATING			922 500	978 718	0.05478	-		53 614			53 614
10	**PROJECT COST**			**98 392 233**					**5 718 393**		**855 078**	**6 691 320**
11	INTEREST DURING CONSTRUCTION			5 996 107			-					
12	**TOTAL INVESTMENT COST**			**104 388 341**	**104 388 341**				**5 718 393**	**117 848**	**855 078**	**6 691 320**

Table 5.12 Rantability of the CFRD formulation

INTEREST RATE : %5.00 — ***ALL UNITS ARE IN DUC***

YEAR	EXPENSE			INCOME			PRESENT VALUE	
	PROJECT COST	OPERATION MAINT., RENEWAL	TOTAL	EXISTING SITUATION	FUTURE SITUATION	INCOME INCREASE	EXPENSE	INCOME
(n)	(DUC)							
	(1)	**(2)**	**(3)=(1)+(2)**	**(4)**	**(5)**	**(6)**	**$(7)=(3)/1.05^n$**	**$(8)=(6)/1.05^n$**
1	45 881 342		45 881 342				43 696 516	
2	52 510 891		52 510 891				47 628 926	
3	0	972 926	972 926	14 649 000	20 408 367	5 759 367	840 450	4 975 158
4	0	972 926	972 926	14 649 000	27 011 432	12 362 432	800 429	10 170 603
5	0	972 926	972 926	14 649 000	29 533 498	14 884 498	762 313	11 662 394
6	0	972 926	972 926	14 649 000	31 784 128	17 135 128	726 013	12 786 496
7	0	972 926	972 926	14 649 000	32 199 195	17 550 195	691 441	12 472 596
8	0	972 926	972 926	14 649 000	32 588 462	17 939 462	658 515	12 142 134
9	0	972 926	972 926	14 649 000	33 273 369	18 624 369	627 157	12 005 434
10	0	972 926	972 926	14 649 000	33 813 559	19 164 559	597 292	11 765 377
11	0	972 926	972 926	14 649 000	34 158 292	19 509 292	568 850	11 406 679
12	0	972 926	972 926	14 649 000	34 294 200	19 645 200	541 762	10 939 182
13	0	972 926	972 926	14 649 000	34 294 200	19 645 200	515 964	10 418 269
14	0	972 926	972 926	14 649 000	34 294 200	19 645 200	491 394	9 922 161
15	0	972 926	972 926	14 649 000	34 294 200	19 645 200	467 994	9 449 677
16	0	972 926	972 926	14 649 000	34 294 200	19 645 200	445 709	8 999 692
17	0	972 926	972 926	14 649 000	34 294 200	19 645 200	424 484	8 571 136
18	0	972 926	972 926	14 649 000	34 294 200	19 645 200	404 271	8 162 986
19	0	972 926	972 926	14 649 000	34 294 200	19 645 200	385 020	7 774 273
20	0	972 926	972 926	14 649 000	34 294 200	19 645 200	366 686	7 404 069
21	0	972 926	972 926	14 649 000	34 294 200	19 645 200	349 224	7 051 495
22	0	972 926	972 926	14 649 000	34 294 200	19 645 200	332 595	6 715 709
23	0	972 926	972 926	14 649 000	34 294 200	19 645 200	316 757	6 395 913
24	0	972 926	972 926	14 649 000	34 294 200	19 645 200	301 673	6 091 346
25	0	972 926	972 926	14 649 000	34 294 200	19 645 200	287 308	5 801 282
26	0	972 926	972 926	14 649 000	34 294 200	19 645 200	273 626	5 525 030
27	0	972 926	972 926	14 649 000	34 294 200	19 645 200	260 597	5 261 934
28	0	972 926	972 926	14 649 000	34 294 200	19 645 200	248 187	5 011 365
29	0	972 926	972 926	14 649 000	34 294 200	19 645 200	236 369	4 772 729
30	0	972 926	972 926	14 649 000	34 294 200	19 645 200	225 113	4 545 456
31	0	972 926	972 926	14 649 000	34 294 200	19 645 200	214 394	4 329 006
32	0	972 926	972 926	14 649 000	34 294 200	19 645 200	204 184	4 122 863
33	0	972 926	972 926	14 649 000	34 294 200	19 645 200	194 461	3 926 536
34	0	972 926	972 926	14 649 000	34 294 200	19 645 200	185 201	3 739 558
35	0	972 926	972 926	14 649 000	34 294 200	19 645 200	176 382	3 561 484
36	0	972 926	972 926	14 649 000	34 294 200	19 645 200	167 983	3 391 889
37	0	972 926	972 926	14 649 000	34 294 200	19 645 200	159 984	3 230 371
38	0	972 926	972 926	14 649 000	34 294 200	19 645 200	152 365	3 076 544
39	0	972 926	972 926	14 649 000	34 294 200	19 645 200	145 110	2 930 042
40	0	972 926	972 926	14 649 000	34 294 200	19 645 200	138 200	2 790 516
41	0	972 926	972 926	14 649 000	34 294 200	19 645 200	131 619	2 657 634
42	0	972 926	972 926	14 649 000	34 294 200	19 645 200	125 351	2 531 080
43	0	972 926	972 926	14 649 000	34 294 200	19 645 200	119 382	2 410 552
44	0	972 926	972 926	14 649 000	34 294 200	19 645 200	113 697	2 295 764
45	0	972 926	972 926	14 649 000	34 294 200	19 645 200	108 283	2 186 442
46	0	972 926	972 926	14 649 000	34 294 200	19 645 200	103 127	2 082 326
47	0	972 926	972 926	14 649 000	34 294 200	19 645 200	98 216	1 983 167
48	0	972 926	972 926	14 649 000	34 294 200	19 645 200	93 539	1 888 731
49	0	972 926	972 926	14 649 000	34 294 200	19 645 200	89 085	1 798 791
50	0	972 926	972 926	14 649 000	34 294 200	19 645 200	84 843	1 713 135
51	0	972 926	972 926	14 649 000	34 294 200	19 645 200	80 803	1 631 557
52	0	972 926	972 926	14 649 000	34 294 200	19 645 200	76 955	1 553 864
	98,392,233						**107 435 800**	**298 032 428**

TOTAL INCOME / TOTAL EXPENSE = 2.774

Income values, indicated by existing situation in Table 5.12, represent the current income of the gross irrigation area before construction of irrigation facilities. And income values indicated by future situation represents the income of the net irrigation area including irrigation benefits. The future situation income increase is gradational considering the fact that maximum efficiency will only be achieved after the crops has reached an optimum size. These values are based on the ones taken from Planning Report (Hidro Dizayn, 2007-a). The CFRD formulation benefit balances its total cost after 9 years of operation as given in Figure 5.4.

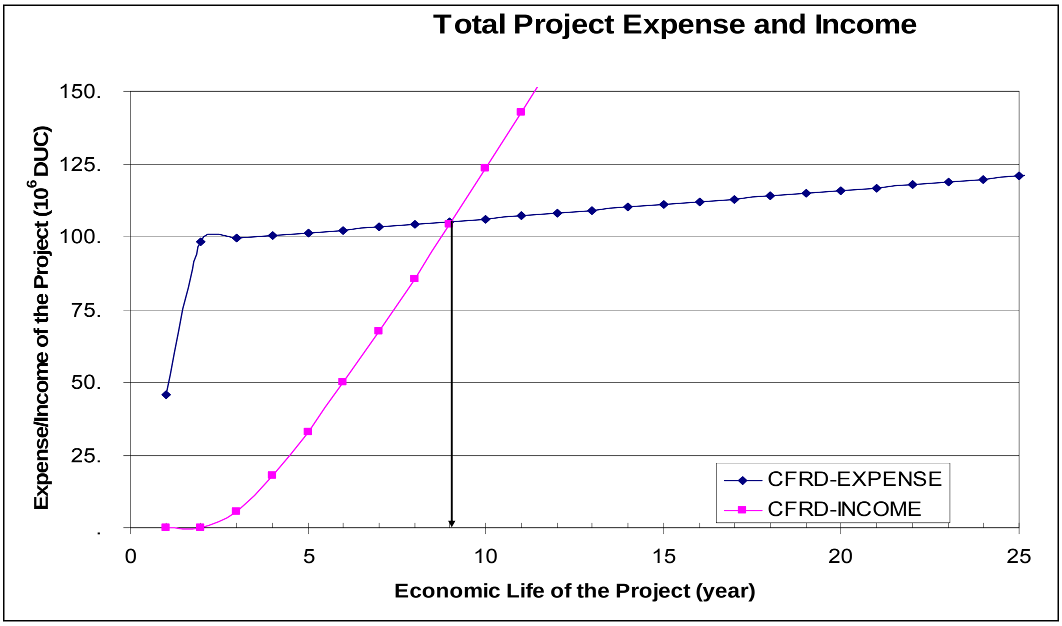

Figure 5.4 Comparison of total expanse with total benefit

Details of calculation of internal rate of return for CFRD formulation is given in Table 5.13. The same calculations conducted for ECRD and RCC formulations are given in Appendices B and C, respectively.

Table 5.13 Internal rate of return for CFRD formulation

YEAR	EXPENSE			INCOME INCREASE	DIFFERENCE	PRESENT VALUE
	PROJECT COST	OPREATION & MAINTENANCE	TOTAL			IRR = 14.37%
	(DUC)			(DUC)		
	(1)	**(2)**	**(3)=(1)+(2)**	**(4)**	**(5)=(4)-(3)**	**(6)**
1	45 881 342		45 881 342		-45 881 342	-40 116 523
2	52 510 891		52 510 891		-52 510 891	-40 144 285
3	0	972 926	972 926	5 759 367	4 786 441	3 199 442
4	0	972 926	972 926	12 362 432	11 389 505	6 656 617
5	0	972 926	972 926	14 884 498	13 911 572	7 109 058
6	0	972 926	972 926	17 135 128	16 162 202	7 221 436
7	0	972 926	972 926	17 550 195	16 577 269	6 476 244
8	0	972 926	972 926	17 939 462	16 966 535	5 795 495
9	0	972 926	972 926	18 624 369	17 651 442	5 271 870
10	0	972 926	972 926	19 164 559	18 191 632	4 750 544
11	0	972 926	972 926	19 509 292	18 536 366	4 232 368
12	0	972 926	972 926	19 645 200	18 672 274	3 727 720
13	0	972 926	972 926	19 645 200	18 672 274	3 259 346
14	0	972 926	972 926	19 645 200	18 672 274	2 849 821
15	0	972 926	972 926	19 645 200	18 672 274	2 491 752
16	0	972 926	972 926	19 645 200	18 672 274	2 178 672
17	0	972 926	972 926	19 645 200	18 672 274	1 904 930
18	0	972 926	972 926	19 645 200	18 672 274	1 665 583
19	0	972 926	972 926	19 645 200	18 672 274	1 456 309
20	0	972 926	972 926	19 645 200	18 672 274	1 273 329
21	0	972 926	972 926	19 645 200	18 672 274	1 113 340
22	0	972 926	972 926	19 645 200	18 672 274	973 453
23	0	972 926	972 926	19 645 200	18 672 274	851 142
24	0	972 926	972 926	19 645 200	18 672 274	744 199
25	0	972 926	972 926	19 645 200	18 672 274	650 693
26	0	972 926	972 926	19 645 200	18 672 274	568 936
27	0	972 926	972 926	19 645 200	18 672 274	497 452
28	0	972 926	972 926	19 645 200	18 672 274	434 949
29	0	972 926	972 926	19 645 200	18 672 274	380 299
30	0	972 926	972 926	19 645 200	18 672 274	332 516
31	0	972 926	972 926	19 645 200	18 672 274	290 736
32	0	972 926	972 926	19 645 200	18 672 274	254 206
33	0	972 926	972 926	19 645 200	18 672 274	222 266
34	0	972 926	972 926	19 645 200	18 672 274	194 339
35	0	972 926	972 926	19 645 200	18 672 274	169 921
36	0	972 926	972 926	19 645 200	18 672 274	148 571
37	0	972 926	972 926	19 645 200	18 672 274	129 904
38	0	972 926	972 926	19 645 200	18 672 274	113 582
39	0	972 926	972 926	19 645 200	18 672 274	99 311
40	0	972 926	972 926	19 645 200	18 672 274	86 833
41	0	972 926	972 926	19 645 200	18 672 274	75 923
42	0	972 926	972 926	19 645 200	18 672 274	66 383
43	0	972 926	972 926	19 645 200	18 672 274	58 042
44	0	972 926	972 926	19 645 200	18 672 274	50 750
45	0	972 926	972 926	19 645 200	18 672 274	44 373
46	0	972 926	972 926	19 645 200	18 672 274	38 798
47	0	972 926	972 926	19 645 200	18 672 274	33 923
48	0	972 926	972 926	19 645 200	18 672 274	29 661
49	0	972 926	972 926	19 645 200	18 672 274	25 934
50	0	972 926	972 926	19 645 200	18 672 274	22 675
51	0	972 926	972 926	19 645 200	18 672 274	19 826
52	0	972 926	972 926	19 645 200	18 672 274	17 335

For type selection, internal rate of return values of three formulations are compared, rather than comparing construction costs of formulation facilities. Comparison of internal rate of return values are plotted in Figure 5.5.

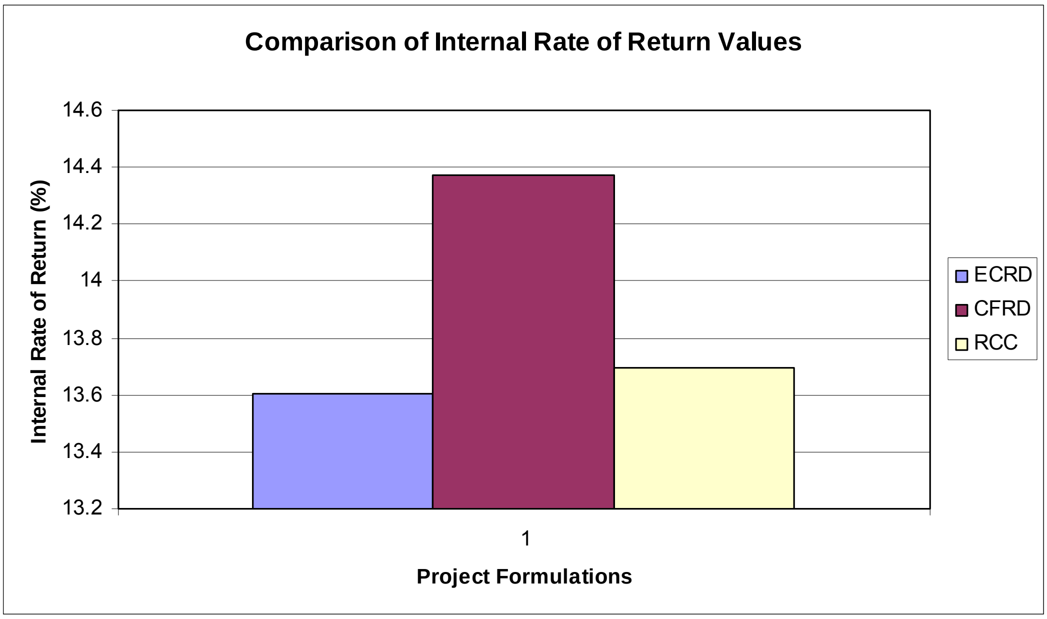

Figure 5.5 Comparison of internal rate of return values

Gökçeler Dam site concrete face rockfill dam formulation is selected with the maximum internal rate of return value cost and easiness in adoptation to climatic charateristics of the project area.

CHAPTER 6

CONCLUSIONS

In this present study, development of concrete face rockfill dam type and the evolution of its design features are overviewed. Current design characteristics of this type of dams are discussed. And CFRD alternative is evaluated for dam type selection of Gökçeler Dam site in Antalya. In order to calculate internal rate of return value, total construction cost of facilities, project cost of formulation and total investment cost are calculated. Internal rate of return value of CFRD formulation is compared with those of ECRD and RCC formulations which are the other two alternatives of Gökçeler Dam Project formulation.

Conclusions of the conducted evaluation and cost comparison studies can be stated as follows:

1. CFRD formulation has the maximum rantability and internal rate of return value. Hence it is selected for the Gökçeler Dam project.

2. Total investment costs of the three alternatives are almost close to each other. However, dam type selection depending only on the total investment cost may mislead the result because construction period significantly affects the rantability of the project.

This study can be further developed by;

- Increasing the number of formulation alternatives, such as considering composite dam types

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APPENDIX A

Specified construction stages are analysed and their unit prices are calculated referring to the Defined Unit Cost (DUC) details of which are given in Tables A.1 ~A.19.

Table A.1 Unit price analysis for GKL-01

UNIT PRICE ANALYSIS TABLE FOR GKL-01 (DUC / m^3)				
EXCAVATION OF PERVIOUS AND IMPERVIOUS FOUNDATION				
DEFINITON OF THE WORK	QUANTITY	Unit	Unit Price (DUC)	Total (DUC)
Excavation of all kinds and classes of foundation except rock and placement in deposit site	1	m^3	1.38	1.38
Hauling of excavated material to dumping site (1 km)	1	m^3	1.86	1.86
SUB TOTAL =				3.24
UNIT PRICE FOR GKL-01 (DUC / m^3) = 3.24				

Table A.2 Unit price analysis for GKL-02

UNIT PRICE ANALYSIS TABLE FOR GKL-02 (DUC / m^3)				
EXCAVATION OF ROCKY FOUNDATION				
DEFINITON OF THE WORK	QUANTITY	Unit	Unit Price (DUC)	Total (DUC)
Excavation of all kinds and classes of rock foundations and placement in deposit site	1	m^3	8.56	8.56
Hauling of excavated rock material to dumping site (1 km)	1	m^3	2.69	2.69
SUB TOTAL =				11.25
UNIT PRICE FOR GKL-02 (DUC / m^3) = 11.25				

Table A.3 Unit price analysis for GKL-03

UNIT PRICE ANALYSIS TABLE FOR GKL-03 (DUC / m^3)				
PREPARATION OF FOUNDATION FOR FILL PLACEMENT				
DEFINITON OF THE WORK	QUANTITY	Unit	Unit Price (DUC)	Total (DUC)
Excavation of all kinds and classes of foundation except rock and placement in deposit site	0.45	m^3	1.38	0.62
Excavation of all kinds and classes of rock foundations and placement in deposit site	0.30	m^3	8.56	2.57
Excavation of marshy foundation and placement in deposit site	0.25	m^3	2.80	0.70
Treatment and cleaning of excavation surface	1	m^2	1.35	1.35
Hauling of foundation excavation material to the placement site (1km)	1	m^3	2.69	2.69
		SUB TOTAL =		7.93
UNIT PRICE FOR GKL-03 (DUC / m^3) = 7.93				

Table A.4 Unit price analysis for GKL-04

UNIT PRICE ANALYSIS TABLE FOR GKL-04 (DUC / m^3)				
PLACEMENT OF IMPERVIOUS FILL MATERIAL				
DEFINITON OF THE WORK	QUANTITY	Unit	Unit Price (DUC)	Total (DUC)
Excavation of impervious fill material from barrow area and placement with in the embankment	1	m^3	2.78	2.78
Hauling of excavated impervious material to the placement site (3 km)	1	m^3	3.59	3.59
		SUB TOTAL =		6.37
UNIT PRICE FOR GKL-04 (DUC / m^3) = 6.37				

Table A.5 Unit price analysis for GKL-05

UNIT PRICE ANALYSIS TABLE FOR GKL-05 (DUC / m^3)				
PLACEMENT OF PERVIOUS FILL MATERIAL				
DEFINITON OF THE WORK	QUANTITY	Unit	Unit Price (DUC)	Total (DUC)
Excavation of pervious fill material from barrow area and placement with in the embankment	1	m^3	2.18	2.18
Hauling of excavated pervious material to the placement site (23 km)	1	m^3	10.33	10.33
		SUB TOTAL =		12.51
UNIT PRICE FOR GKL-05 (DUC / m^3) = 12.51				

Table A.6 Unit price analysis for GKL-06

UNIT PRICE ANALYSIS TABLE FOR GKL-06 (DUC / m^3)				
PLACEMENT OF ROCK FILL MATERIAL				
DEFINITON OF THE WORK	QUANTITY	Unit	Unit Price (DUC)	Total (DUC)
Excavation of rock fill material from quarries and placement with in the rockfill	1	m^3	8.51	8.51
Hauling of excavated pervious material to the placement site (2.5 km)	1	m^3	4.26	4.26
		SUB TOTAL =		12.77
UNIT PRICE FOR GKL-06 (DUC / m^3) = 12.77				

Table A.7 Unit price analysis for GKL-07

UNIT PRICE ANALYSIS TABLE FOR GKL-07 (DUC / m^3)				
ACEMENT OF EXCAVATED IMPERVIOUS OR PERVIOUS MATERIAL WITHIN EMBANKME				
DEFINITON OF THE WORK	QUANTITY	Unit	Unit Price (DUC)	Total (DUC)
Excavation of all kinds and classes of foundation except rock and placement within the embankment	1	m^3	0.79	0.79
Hauling of excavated material to placement location (0.5 km)	1	m^3	1.46	1.46
		SUB TOTAL =		2.25
UNIT PRICE FOR GKL-07 (DUC / m^3) = 2.25				

Table A.8 Unit price analysis for GKL-08

UNIT PRICE ANALYSIS TABLE FOR GKL-08 (DUC / m^3)				
PLACEMENT OF EXCAVATED ROCK MATERIAL WITHIN EMBANKMENT				
DEFINITON OF THE WORK	QUANTITY	Unit	Unit Price (DUC)	Total (DUC)
Excavation of all kinds and classes of rock foundations and placement within embankment	1	m^3	1.20	1.20
Hauling of excavated rock material to placement location (0.5 km)	1	m^3	1.46	1.46
		SUB TOTAL =		2.66
UNIT PRICE FOR GKL-08 (DUC / m^3) = 2.66				

Table A.9 Unit price analysis for GKL-09

UNIT PRICE ANALYSIS TABLE FOR GKL-09 (DUC / m^3)				
PREPARATION AND PLACEMENT OF FILTER MATERIAL				
DEFINITON OF THE WORK	QUANTITY	Unit	Unit Price (DUC)	Total (DUC)
Extraction of filter material from barrow area, preparation and placing within the embankment	1	m^3	8.86	8.86
Washing of filter material	1	m^3	0.64	0.64
Hauling of filter material to placement location (23 km)	1	m^3	9.30	9.30
		SUB TOTAL =		18.80
UNIT PRICE FOR GKL-09 (DUC / m^3) = 18.80				

Table A.10 Unit price analysis for GKL-10

UNIT PRICE ANALYSIS TABLE FOR GKL-10 (DUC / m^3)				
COMPACTION OF PERVIOUS FILL MATERIAL				
DEFINITON OF THE WORK	QUANTITY	Unit	Unit Price (DUC)	Total (DUC)
Compaction of pervious embankment material by vibratory roller compactors	1	hour	98.79	98.79
Sluicing and washing of pervious fill material	7.5	m^3	0.64	4.80
		SUB TOTAL (for 150 m^3)=		103.59
UNIT PRICE FOR GKL-10 (DUC / m^3) = 0.69				

Table A.11 Unit price analysis for GKL-11

UNIT PRICE ANALYSIS TABLE FOR GKL-11 (DUC / m^3)				
COMPACTION OF IMPERVIOUS FILL MATERIAL				
DEFINITON OF THE WORK	QUANTITY	Unit	Unit Price (DUC)	Total (DUC)
Compaction of impervious embankment material by vibratory sheep-foot compactors	1	hour	62.45	62.45
Sluicing and washing of impervious fill material	10	m^3	2.28	22.80
SUB TOTAL (for 100 m^3)=				85.25
UNIT PRICE FOR GKL-11 (DUC / m^3) = 0.85				

Table A.12 Unit price analysis for GKL-12

UNIT PRICE ANALYSIS TABLE FOR GKL-12 (DUC / m^3)				
COMPACTION OF ROCKFILL MATERIAL				
DEFINITON OF THE WORK	QUANTITY	Unit	Unit Price (DUC)	Total (DUC)
Compaction of rockfill material by vibratory roller compactors	1	hour	98.79	98.79
Washing of rockfill material with high pressurized water	29.25	m^3	1.09	31.88
SUB TOTAL (for 225 m^3)=				130.67
UNIT PRICE FOR GKL-12 (DUC / m^3) = 0.58				

Table A.13 Unit price analysis for GKL-13

UNIT PRICE ANALYSIS TABLE FOR GKL-13 (DUC / m^3)				
PLACEMENT OF SURFACE PROTECTION				
DEFINITON OF THE WORK	QUANTITY	Unit	Unit Price (DUC)	Total (DUC)
Preparation of qualified rock boulders extracted from quarries	1	m^3	12.91	12.91
Hauling of extracted protection material to the placement site (2.5 km)	1	m^3	3.28	3.28
		SUB TOTAL=		16.19
UNIT PRICE FOR GKL-13 (DUC / m^3) = 16.19				

Table A.14 Unit price analysis for GKL-14

UNIT PRICE ANALYSIS TABLE FOR GKL-14 (DUC / m^3)				
PREPARATION OF CONCRETE AGGREGATE				
DEFINITON OF THE WORK	QUANTITY	Unit	Unit Price (DUC)	Total (DUC)
Preparation of conrete aggregate by washing	1	m^3	0.64	0.64
Hauling of aggregate to the concrete plant (23 km)	1	m^3	9.30	9.30
		SUB TOTAL=		9.94
UNIT PRICE FOR GKL-14 (DUC / m^3) = 9.94				

Table A.15 Unit price analysis for GKL-15

UNIT PRICE ANALYSIS TABLE FOR GKL-15 (DUC / ton)				
SUPPLY OF CEMENT FOR CONCRETE				
DEFINITON OF THE WORK	QUANTITY	Unit	Unit Price (DUC)	Total (DUC)
Cost of cement	1	ton	156.38	156.38
Hauling of cement to the concrete plant (199 km)	1	ton	26.32	26.32
		SUB TOTAL=		182.70
UNIT PRICE FOR GKL-15 (DUC / ton) = 182.70				

Table A.16 Unit price analysis for GKL-16

UNIT PRICE ANALYSIS TABLE FOR GKL-16 (DUC / ton)				
SUPPLY OF CONSTRUCTIONAL STEEL				
DEFINITON OF THE WORK	QUANTITY	Unit	Unit Price (DUC)	Total (DUC)
Cost of steel bars	1	ton	1437.40	1437.40
Hauling of steel bars to the site workshop (499 km)	1	ton	70.09	70.09
		SUB TOTAL=		1,507.49
UNIT PRICE FOR GKL-16 (DUC / ton) = 1,507.49				

Table A.17 Unit price analysis for GKL-17

UNIT PRICE ANALYSIS TABLE FOR GKL-17 (DUC / m^3)				
PREPARATION AND PLACEMENT OF CONCRETE				
DEFINITON OF THE WORK	QUANTITY	Unit	Unit Price (DUC)	Total (DUC)
Preparation of concrete with required compressive strength	1	m^3	86.99	86.99
Formwork for curved surfaces which expose water directly	2	m^2	64.99	129.98
Supply and placement of PVC waterstops	7.6	kg	10.15	77.14
		SUB TOTAL=		294.11
UNIT PRICE FOR GKL-17 (DUC / m^3) = 294.11				

Table A.18 Unit price analysis for GKL-18

UNIT PRICE ANALYSIS TABLE FOR GKL-18 (DUC / m)				
FOUNDATION GROUTING				
DEFINITON OF THE WORK	QUANTITY	Unit	Unit Price (DUC)	Total (DUC)
Drilling of bore holes without sampling	1	m	119.14	119.14
Grout mix injection from boreholes	0.0565	m^3	676.87	38.24
Supply of required cement for grout mix	0.02	ton	156.38	3.13
Hauling of grout mix cement	0.02	ton	26.32	0.53
		SUB TOTAL=		161.04
UNIT PRICE FOR GKL-18 (DUC / m) = 161.04				

Table A.19 Unit price analysis for GKL-19

UNIT PRICE ANALYSIS TABLE FOR GKL-19 (DUC / m^3)				
PREPARATION AND PLACEMENT OF ROLLER COMPACTED CONCRETE				
DEFINITON OF THE WORK	QUANTITY	Unit	Unit Price (DUC)	Total (DUC)
Preparation of concrete aggregate by washing	1.25	m^3	0.64	0.80
Hauling of aggregate to the concrete plant (23 km)	1.25	m^3	9.30	11.62
Cost of cement	0.08	ton	156.38	12.51
Hauling of cement to the concrete plant (199 km)	0.08	ton	26.32	2.11
Compaction of roller compacted concrete by vibratory roller compactors	0.01	hour	98.79	0.99
Laboring	5	hour	3.70	18.50
		SUB TOTAL=		46.53
UNIT PRICE FOR GKL-19 (DUC / m^3) = 46.53				

APPENDIX B

Earth Core Rockfill formulation is composed of dam body, spillway, cofferdams, diversion tunnel, and valve chamber. General layout plan and main cross-section of dam body for earth core rockfill dam formulation is given in Figures B.1 and B.2, respectively (Hidro Dizayn, 2007-b).

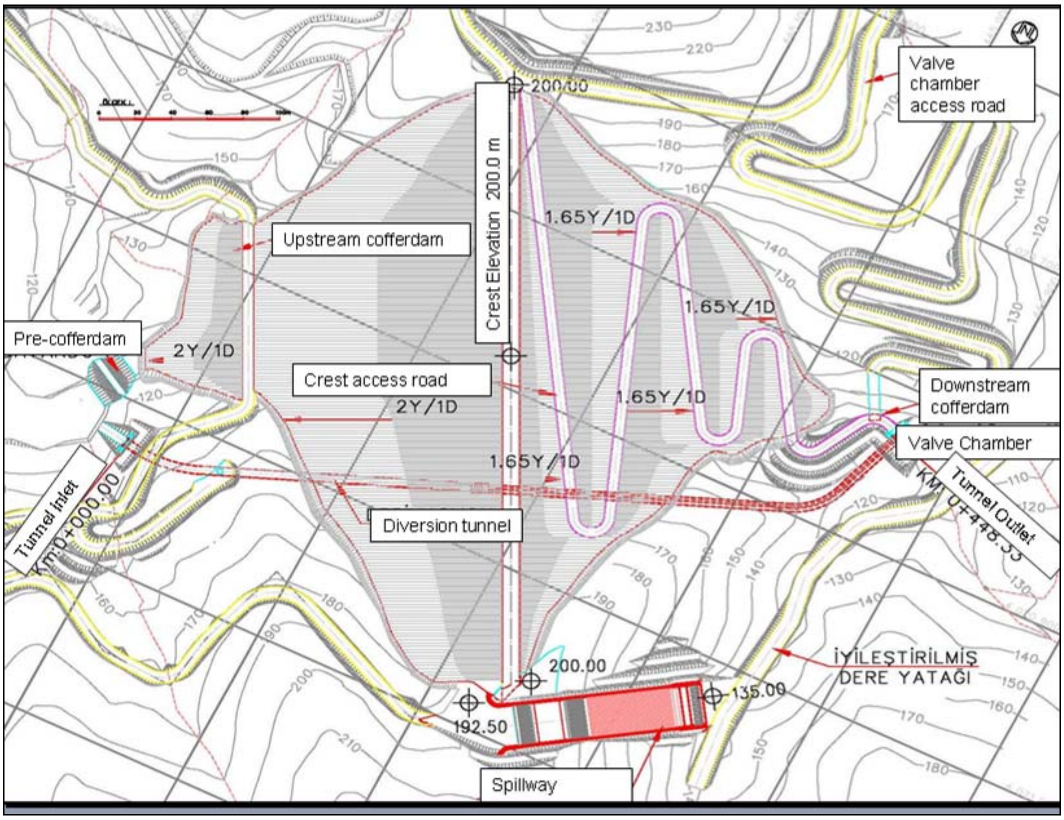

Figure B.1 General layout of ECRD formulation (Hidro Dizayn, 2007-b)

The upstream cofferdam is planned to be combined to the dam embankment as commonly practiced for earth core rockfill dams. Along with the dam body, spillway, diversion tunnel and cofferdams are also taken into consideration for cost comparison.

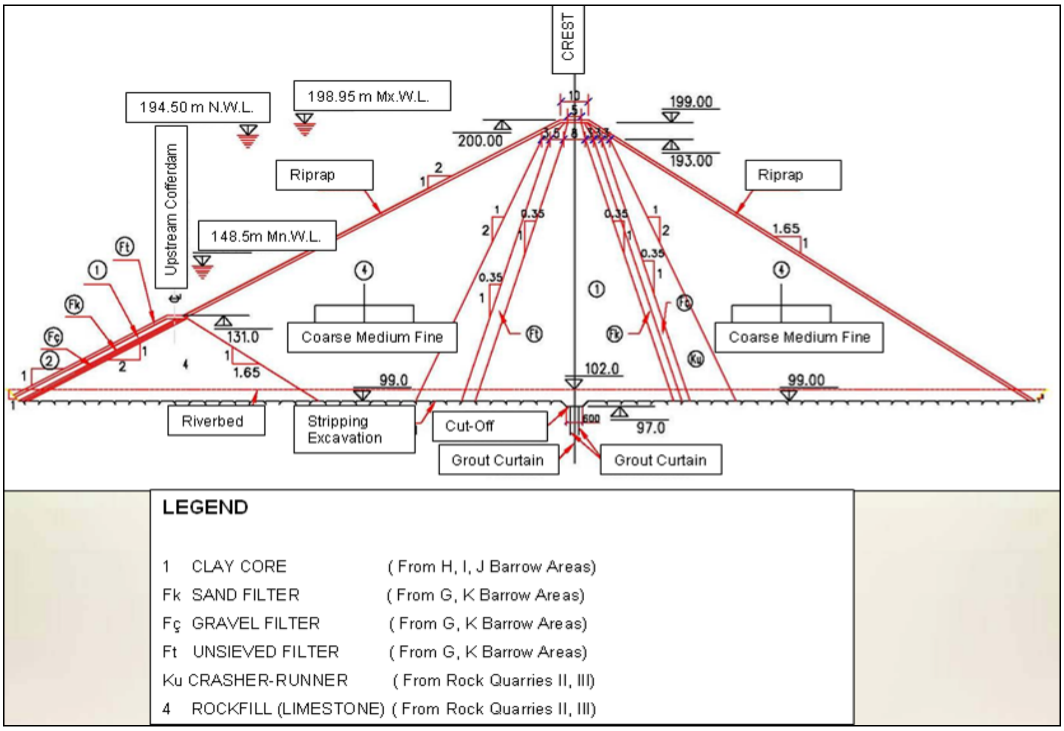

Figure B.2 Main cross-section ECRD type (Hidro Dizayn, 2007-b)

Estimated cost of facilities taking place within the Earth Core Rockfill Dam for this formulation is given in detail in Tables B.1 ~B.5.

Table B.1 Estimated cost analysis of cofferdams

COST ANALYSIS TABLE FOR ECRD PRE-COFFERDAM AND DOWNSTREAM COFFERDAM					
UNIT PRICE CODE	DEFINITION OF THE WORK	QUANTITY	UNIT	UNIT PRICE (DUC)	TOTAL (DUC)
GKL-08	Placement of excavated rock foundation within the rockfill and haulage for 1 km . (For pre-cofferdam)	1,200	m^3	2.66	3,198
GKL-12	Sluicing and compaction of rockfill material. (For pre-cofferdam)	1,200	m^3	0.58	697
GKL-14	Preparation of aggregate mixed in concrete mortar and haulage for 23 km . (For downstream coff.dam)	1,375	m^3	9.94	13,664
GKL-15	Supply of cement mixed in concrete mortar and haulage for 199 km. (For downstream cofferdam)	330	ton	182.70	60,292
GKL-16	Supply of construction steel and haulage for 499 km. (For downstream cofferdam)	33	ton	1,507.49	49,747
GKL-17	Preparation and placement of concrete. (For downstream cofferdam)	1,100	m^3	294.11	323,521
		SUB TOTAL =	451,119		
TOTAL COST OF COFFERDAMS (DUC) = 451,119					

Table B.2 Estimated cost analysis of diversion tunnel

COST ANALYSIS TABLE FOR ECRD DIVERSION TUNNEL (L=450 m)					
UNIT PRICE CODE	DEFINITION OF THE WORK	QUANTITY	UNIT	UNIT PRICE (DUC)	TOTAL (DUC)
GKL-02	Excavation of rock foundation and haulage of rock for 1 km.	9,200	m^3	11.25	103,525
	Increase of excavation cost by 1% per 100 m of tunnel length exceeding 300 m	2		0.01	1,553
GKL-14	Preparation of aggregate mixed in concrete mortar and haulage for 23 km .	4,406	m^3	9.94	43,788
GKL-15	Supply of cement mixed in concrete mortar and haulage for 199 km.	1,058	ton	182.70	193,208
GKL-16	Supply of construction steel and haulage for 499 km.	282	ton	1,507.49	425,111
GKL-17	Preparation and placement of concrete	3,525	m^3	294.11	1,036,738
GKL-19	Preparation and placement of roller compacted concrete	2,275	m	161.04	366,365
			SUB TOTAL =	2,170,287	
TOTAL COST OF DIVERSION TUNNEL (DUC) = 2,712,859					

Table B.3 Estimated cost analysis of spillway

COST ANALYSIS TABLE FOR ECRD SPILLWAY					
UNIT PRICE CODE	DEFINITION OF THE WORK	QUANTITY	UNIT	UNIT PRICE (DUC)	TOTAL (DUC)
GKL-01	Excavation of pervious (except rock) and impervious foundation material and haulage for 1 km.	58,500	m^3	3.24	189,784
GKL-02	Excavation of rock foundation and haulage of rock for 1 km.	25,000	m^3	11.25	281,317
GKL-13	Placement of surface protection from rockfill and haulage for 2.5 km.	190	m^3	16.19	3,075
GKL-14	Preparation of aggregate mixed in concrete mortar and haulage for 23 km .	8,500	m^3	9.94	84,470
GKL-15	Supply of cement mixed in concrete mortar and haulage for 199 km.	2,040	ton	182.70	372,714
GKL-16	Supply of construction steel and haulage for 499 km.	408	ton	1,507.49	615,054
GKL-17	Preparation and placement of concrete	6,800	m^3	294.11	1,999,948
			SUB TOTAL =	3,546,361	
TOTAL COST OF SPILLWAY (DUC) = 3,546,361					

Table B.4 Estimated cost analysis of dam body

COST ANALYSIS TABLE FOR ECRD DAM BODY					
UNIT PRICE CODE	DEFINITION OF THE WORK	QUANTITY	UNIT	UNIT PRICE (DUC)	TOTAL (DUC)
GKL-03	Preparation of embankment foundation for placement of fill material.	273,000	m^3	7.93	2,165,348
GKL-06	Extraction of rockfill material from quarries, placement within the fill and haulage for 2.5 km.	1,905,000	m^3	12.77	24,322,088
GKL-12	Sluicing and compaction of rockfill material.	1,905,000	m^3	0.58	1,106,361
GKL-04	Extraction of impervious fill material from barrow areas, placement within the fill and haulage for 3 km.	547,000	m^3	6.37	3,483,068
GKL-11	Sluicing and compaction of impervious material.	547,000	m^3	0.85	466,318
GKL-09	Preparation of filter material and haulage for 23 km.	161,500	m^3	18.80	3,035,811
GKL-10	Sluicing and compaction of pervious material (except rock)	161,500	m^3	0.69	111,532
GKL-13	Placement of surface protection from rockfill and haulage for 2.5 km.	57,000	m^3	16.19	922,545
GKL-14	Preparation of aggregate mixed in concrete mortar and haulage for 23 km .	63	m^3	9.94	621
GKL-15	Supply of cement for concrete mortar and haulage for 199 km.	15	ton	182.70	2,741
GKL-16	Supply of construction steel and haulage for 499 km.	2	ton	1,507.49	2,261
GKL-17	Preparation and placement of concrete (For handrail poles on the crest)	50	m^3	294.11	14,706
		SUB TOTAL =		35,633,398	
TOTAL COST OF DAM BODY(DUC) = 35,633,398					

Table B.5 Estimated cost analysis of grouting

COST ANALYSIS TABLE FOR ECRD GROUTING					
UNIT PRICE CODE	DEFINITION OF THE WORK	QUANTITY	UNIT	UNIT PRICE (DUC)	TOTAL (DUC)
GKL-18	Grouting of every kinds and classes of formation	9,000	m	161.04	1,449,357
			SUB TOTAL =	1,449,357	
TOTAL COST OF GROUTING (DUC) = 1,449,357					

Table B.6 Total investment cost of ECRD formulation

		Toplam	1. "6 Ay"	2. "6 Ay"	3. "6 Ay"	4. "6 Ay"	5. "6 Ay"	6. "6 Ay"
COFFERDAMS	(1)	451 119	451 119	-	-	-	-	-
DIVERSION TUNNEL	(2)	2 712 859	2 712 859	-	-	-	-	-
SPILLWAY	(3)	3 546 361	-	-	354 636	2 127 817	1 063 908	-
DAM BODY	(4)	35 633 398	-	5 090 485	7 635 728	7 635 728	7 635 728	7 635 728
GROUTING	(5)	1 449 357	-	1 449 357	-	-	-	-
IRRIGATION AND DRAINAGE FACILITIES	(6)	29 966 247	4 994 375	4 994 375	4 994 375	4 994 375	4 994 375	4 994 375
ESTIMATED COST	**(7)=(1)+...+(6)**	**73 759 342**	**8 158 352**	**11 534 217**	**12 984 739**	**14 757 919**	**13 694 011**	**12 630 103**
CONTINGENCY	(8)=(7)*0.15	11 063 901	1 223 753	1 730 133	1 947 711	2 213 688	2 054 102	1 894 515
CONSTRUCTION COST	**(9)=(7)+(8)**	**84 823 243**	**9 382 105**	**13 264 350**	**14 932 450**	**16 971 607**	**15 748 113**	**14 524 618**
PROJECT CONTROL	(10)=(9)*0.15	12 723 487	1 407 316	1 989 653	2 239 867	2 545 741	2 362 217	2 178 693
EXPROPRIATING	(11)	1 365 000	1,365,000					
PROJECT COST	**(12)=(9)+(10)+(11)**	**98 911 730**	**12 154 421**	**15 254 003**	**17 172 317**	**19 517 348**	**18 110 330**	**16 703 311**
INTEREST DURING CONSTRUCTION	(13)(*)	8 454 804	1 915 841	1 978 846	1 760 163	1 481 949	905 516	412 490
PERIODICAL CUMMULATIVE DEBT	(14)		14 070 262	31 303 110	50 235 589	71 234 887	90 250 733	107 366 534
TOTAL INVESTMENT COST	**(15)=(12)+(13)**	**107 366 534**	**14 070 262**	**17 232 848**	**18 932 480**	**20 999 297**	**19 015 846**	**17 115 800**

Table B.7 Annual project expense of ECRD formulation

INTEREST RATE : %5.00

ALL UNITS ARE IN DUC

No.	TYPE OF WORK	CONSTR. COST	CONTINGENCY	COST	INVESTMENT COST	INTEREST & AMORT. FACTOR	RENEWAL FACTOR	OPERATION & MAINTENANCE FACTOR	ANNUAL COST: INTEREST & AMORT.	ANNUAL COST: RENEWAL	ANNUAL COST: OPERATION & MAINTEN.	ANNUAL COST: TOTAL
		(DUC)							(DUC)			
1	COFFERDAMS	451 119	67 668	518 786	563 131	0.05478	0.0001300	0.002000	30 848	67	1 038	31 953
2	DIVERSION TUNNEL	2 712 859	406 929	3 119 788	3 386 462	0.05478	0.0001300	0.005000	185 510	406	15 599	201 515
3	SPILLWAY	3 546 361	531 954	4 078 315	4 426 923	0.05478	0.0001300	0.010000	242 507	530	40 783	283 820
4	DAM BODY	35 633 398	5 345 010	40 978 408	44 481 172	0.05478	0.0001300	0.002000	2 436 679	5 327	81 957	2 523 963
5	GROUTING	1 449 357	217 404	1 666 761	1 809 233	0.05478	0.0001300	0.020000	99 110	217	33 335	132 662
6	IRRIGATION AND DRAINAGE FACILITIES	29 966 247	4 494 937	34 461 184	37 406 867	0.05478	0.0032300	0.019900	2 049 148	111 310	685 778	2 846 235
7	**CONSTRUCTION COST**			**84 823 243**					**5 043 802**	**117 857**	**858 489**	**6 020 148**
8	PROJECT CONTROL			12 723 487	13 811 068	0.05478	-		756 570			756 570
9	EXPROPRIATING			1 365 000	1 481 678	0.05478	-		81 166			81 166
10	**PROJECT COST**			**98 911 730**					**5 881 539**		**858 489**	**6 857 885**
11	INTEREST DURING CONSTRUCTION			8 454 804			-					
12	**TOTAL INVESTMENT COST**			**107 366 534**	**107 366 534**				**5 881 539**	**117 857**	**858 489**	**6 857 885**

Table B.8 Rantability of ECRD formulation

YEAR	EXPENSE			INCOME			PRESENT VALUE	
	PROJECT COST	OPERATION MAINT., RENEW	TOTAL	EXISTING SITUATION	FUTURE SITUATION	INCOME INCREASE	EXPENSE	INCOME
(n)	(DUC)							
	(1)	**(2)**	**(3)=(1)+(2)**	**(4)**	**(5)**	**(6)**	**(7)=(3)/1.05^n**	**(8)=(6)/1.05^n**
1	27 408 424		27 408 424				26 103 260	
2	36 689 666		36 689 666				33 278 608	
3	34 813 641		34 813 641				30 073 332	
4	0	976 346	976 346	14 649 000	20 408 367	5 759 367	803 242	4 738 245
5	0	976 346	976 346	14 649 000	27 011 432	12 362 432	764 993	9 686 289
6	0	976 346	976 346	14 649 000	29 533 498	14 884 498	728 564	11 107 042
7	0	976 346	976 346	14 649 000	31 784 128	17 135 128	693 871	12 177 616
8	0	976 346	976 346	14 649 000	32 199 195	17 550 195	660 829	11 878 663
9	0	976 346	976 346	14 649 000	32 588 462	17 939 462	629 361	11 563 937
10	0	976 346	976 346	14 649 000	33 273 369	18 624 369	599 392	11 433 747
11	0	976 346	976 346	14 649 000	33 813 559	19 164 559	570 849	11 205 121
12	0	976 346	976 346	14 649 000	34 158 292	19 509 292	543 666	10 863 504
13	0	976 346	976 346	14 649 000	34 294 200	19 645 200	517 777	10 418 269
14	0	976 346	976 346	14 649 000	34 294 200	19 645 200	493 121	9 922 161
15	0	976 346	976 346	14 649 000	34 294 200	19 645 200	469 639	9 449 677
16	0	976 346	976 346	14 649 000	34 294 200	19 645 200	447 275	8 999 692
17	0	976 346	976 346	14 649 000	34 294 200	19 645 200	425 977	8 571 136
18	0	976 346	976 346	14 649 000	34 294 200	19 645 200	405 692	8 162 986
19	0	976 346	976 346	14 649 000	34 294 200	19 645 200	386 373	7 774 273
20	0	976 346	976 346	14 649 000	34 294 200	19 645 200	367 975	7 404 069
21	0	976 346	976 346	14 649 000	34 294 200	19 645 200	350 452	7 051 495
22	0	976 346	976 346	14 649 000	34 294 200	19 645 200	333 764	6 715 709
23	0	976 346	976 346	14 649 000	34 294 200	19 645 200	317 870	6 395 913
24	0	976 346	976 346	14 649 000	34 294 200	19 645 200	302 734	6 091 346
25	0	976 346	976 346	14 649 000	34 294 200	19 645 200	288 318	5 801 282
26	0	976 346	976 346	14 649 000	34 294 200	19 645 200	274 588	5 525 030
27	0	976 346	976 346	14 649 000	34 294 200	19 645 200	261 513	5 261 934
28	0	976 346	976 346	14 649 000	34 294 200	19 645 200	249 060	5 011 365
29	0	976 346	976 346	14 649 000	34 294 200	19 645 200	237 200	4 772 729
30	0	976 346	976 346	14 649 000	34 294 200	19 645 200	225 904	4 545 456
31	0	976 346	976 346	14 649 000	34 294 200	19 645 200	215 147	4 329 006
32	0	976 346	976 346	14 649 000	34 294 200	19 645 200	204 902	4 122 863
33	0	976 346	976 346	14 649 000	34 294 200	19 645 200	195 145	3 926 536
34	0	976 346	976 346	14 649 000	34 294 200	19 645 200	185 852	3 739 558
35	0	976 346	976 346	14 649 000	34 294 200	19 645 200	177 002	3 561 484
36	0	976 346	976 346	14 649 000	34 294 200	19 645 200	168 573	3 391 889
37	0	976 346	976 346	14 649 000	34 294 200	19 645 200	160 546	3 230 371
38	0	976 346	976 346	14 649 000	34 294 200	19 645 200	152 901	3 076 544
39	0	976 346	976 346	14 649 000	34 294 200	19 645 200	145 620	2 930 042
40	0	976 346	976 346	14 649 000	34 294 200	19 645 200	138 686	2 790 516
41	0	976 346	976 346	14 649 000	34 294 200	19 645 200	132 082	2 657 634
42	0	976 346	976 346	14 649 000	34 294 200	19 645 200	125 792	2 531 080
43	0	976 346	976 346	14 649 000	34 294 200	19 645 200	119 802	2 410 552
44	0	976 346	976 346	14 649 000	34 294 200	19 645 200	114 097	2 295 764
45	0	976 346	976 346	14 649 000	34 294 200	19 645 200	108 664	2 186 442
46	0	976 346	976 346	14 649 000	34 294 200	19 645 200	103 489	2 082 326
47	0	976 346	976 346	14 649 000	34 294 200	19 645 200	98 561	1 983 167
48	0	976 346	976 346	14 649 000	34 294 200	19 645 200	93 868	1 888 731
49	0	976 346	976 346	14 649 000	34 294 200	19 645 200	89 398	1 798 791
50	0	976 346	976 346	14 649 000	34 294 200	19 645 200	85 141	1 713 135
51	0	976 346	976 346	14 649 000	34 294 200	19 645 200	81 087	1 631 557
52	0	976 346	976 346	14 649 000	34 294 200	19 645 200	77 225	1 553 864
53	0	976 346	976 346	14 649 000	34 294 200	19 645 200	73 548	1 479 870
	98,911,730						**104 852 328**	**283 840 408**

TOTAL INCOME / TOTAL EXPENSE **2.707**

Table B.9 Internal rate of return for ECRD formulation

YEAR	EXPENSE			INCOME INCREASE	DIFFERENCE	PRESENT VALUE
	PROJECT COST	OPREATION & MAINTENANCE	TOTAL			IRR = 13.6%
	(DUC)			(DUC)		
	(1)	(2)	(3)=(1)+(2)	(4)	(5)=(4)-(3)	(6)
1	27 408 424		27 408 424		-27 408 424	-24 126 796
2	36 689 666		36 689 666		-36 689 666	-28 429 874
3	34 813 641		34 813 641		-34 813 641	-23 746 314
4	0	976 346	976 346	5 759 367	4 783 021	2 871 869
5	0	976 346	976 346	12 362 432	11 386 086	6 018 003
6	0	976 346	976 346	14 884 498	13 908 152	6 470 874
7	0	976 346	976 346	17 135 128	16 158 782	6 617 862
8	0	976 346	976 346	17 550 195	16 573 849	5 975 140
9	0	976 346	976 346	17 939 462	16 963 116	5 383 267
10	0	976 346	976 346	18 624 369	17 648 023	4 930 057
11	0	976 346	976 346	19 164 559	18 188 213	4 472 615
12	0	976 346	976 346	19 509 292	18 532 946	4 011 728
13	0	976 346	976 346	19 645 200	18 668 854	3 557 298
14	0	976 346	976 346	19 645 200	18 668 854	3 131 381
15	0	976 346	976 346	19 645 200	18 668 854	2 756 459
16	0	976 346	976 346	19 645 200	18 668 854	2 426 426
17	0	976 346	976 346	19 645 200	18 668 854	2 135 909
18	0	976 346	976 346	19 645 200	18 668 854	1 880 175
19	0	976 346	976 346	19 645 200	18 668 854	1 655 060
20	0	976 346	976 346	19 645 200	18 668 854	1 456 899
21	0	976 346	976 346	19 645 200	18 668 854	1 282 463
22	0	976 346	976 346	19 645 200	18 668 854	1 128 913
23	0	976 346	976 346	19 645 200	18 668 854	993 748
24	0	976 346	976 346	19 645 200	18 668 854	874 766
25	0	976 346	976 346	19 645 200	18 668 854	770 029
26	0	976 346	976 346	19 645 200	18 668 854	677 833
27	0	976 346	976 346	19 645 200	18 668 854	596 676
28	0	976 346	976 346	19 645 200	18 668 854	525 236
29	0	976 346	976 346	19 645 200	18 668 854	462 349
30	0	976 346	976 346	19 645 200	18 668 854	406 991
31	0	976 346	976 346	19 645 200	18 668 854	358 262
32	0	976 346	976 346	19 645 200	18 668 854	315 367
33	0	976 346	976 346	19 645 200	18 668 854	277 608
34	0	976 346	976 346	19 645 200	18 668 854	244 370
35	0	976 346	976 346	19 645 200	18 668 854	215 111
36	0	976 346	976 346	19 645 200	18 668 854	189 356
37	0	976 346	976 346	19 645 200	18 668 854	166 684
38	0	976 346	976 346	19 645 200	18 668 854	146 727
39	0	976 346	976 346	19 645 200	18 668 854	129 159
40	0	976 346	976 346	19 645 200	18 668 854	113 695
41	0	976 346	976 346	19 645 200	18 668 854	100 082
42	0	976 346	976 346	19 645 200	18 668 854	88 099
43	0	976 346	976 346	19 645 200	18 668 854	77 551
44	0	976 346	976 346	19 645 200	18 668 854	68 266
45	0	976 346	976 346	19 645 200	18 668 854	60 092
46	0	976 346	976 346	19 645 200	18 668 854	52 897
47	0	976 346	976 346	19 645 200	18 668 854	46 564
48	0	976 346	976 346	19 645 200	18 668 854	40 989
49	0	976 346	976 346	19 645 200	18 668 854	36 081
50	0	976 346	976 346	19 645 200	18 668 854	31 761
51	0	976 346	976 346	19 645 200	18 668 854	27 958
52	0	976 346	976 346	19 645 200	18 668 854	24 611
53	0	976 346	976 346	19 645 200	18 668 854	21 664

APPENDIX C

Roller Compacted Concrete formulation is composed of the dam body, spillway, cofferdams, diversion tunnel, and valve chamber. General layout of the project and the main cross-section of the dam are given in Figure C.1 and Figure C.2, respectively.

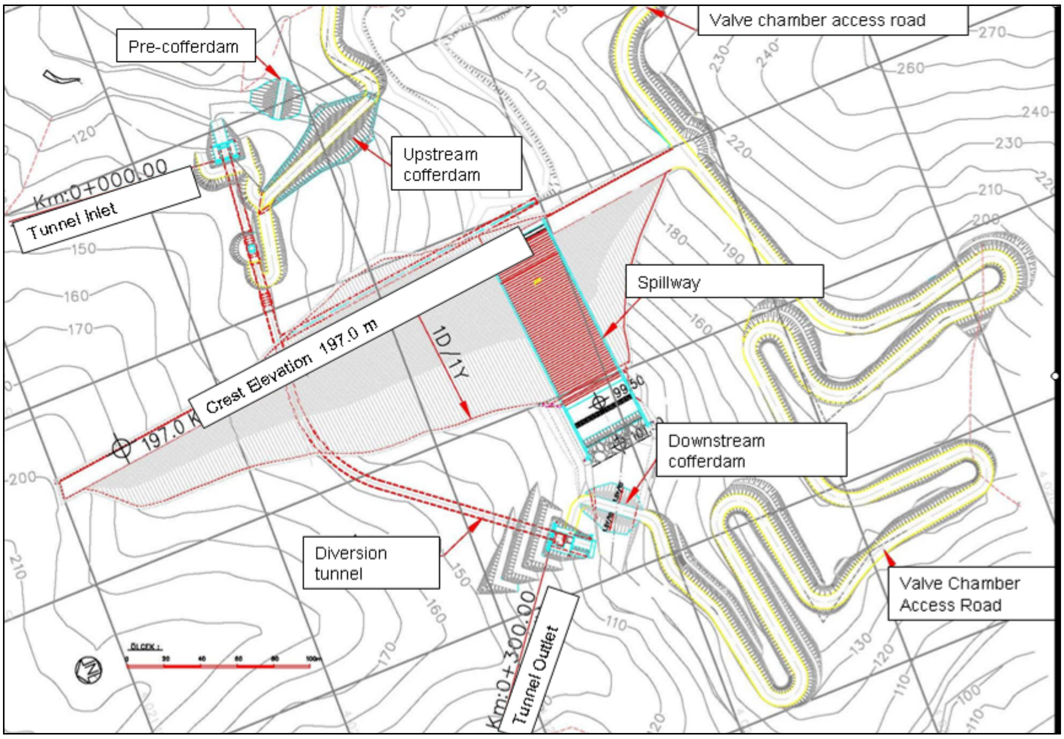

Figure C.1 General layout of RCC formulation (Hidro Dizayn, 2007-b)

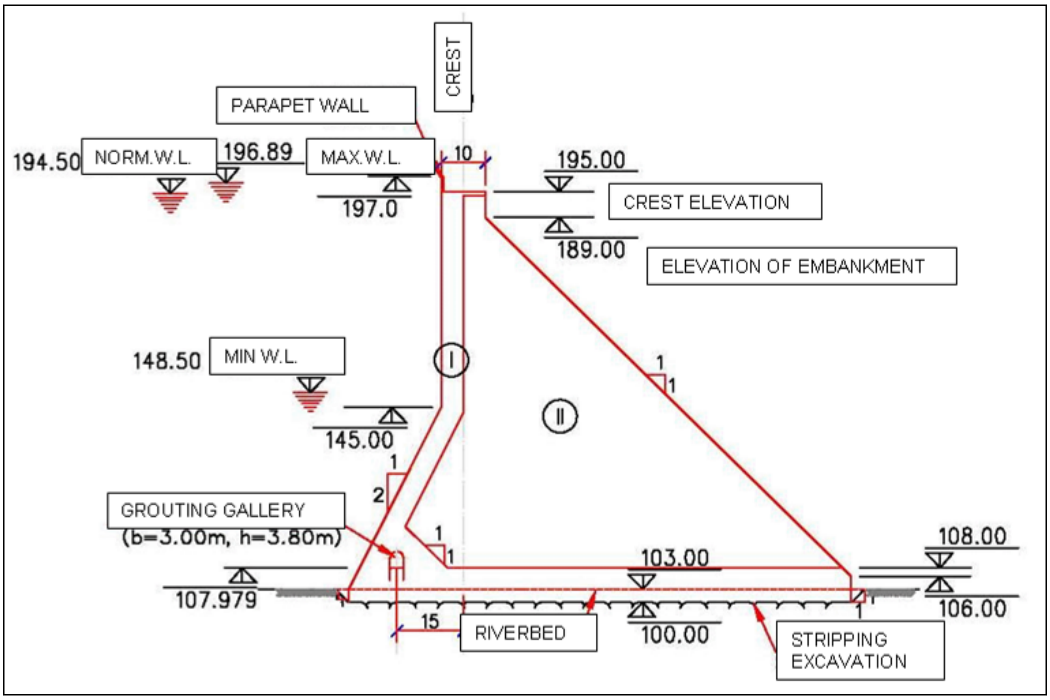

Figure C.2 Main cross-section of RCC type (Hidro Dizayn, 2007-b)

The most significant feature of RCC formulation is the spillway constructed on the dam body. This spillway is designed to transfer 10 000 years return period flood to the downstream safely. Its capacity is 315 m^3/s and maximum reservoir elevation is 197.00 m.

Estimated cost of facilities taking place within the Roller Compacted Concrete Dam for this formulation is given in detail in Tables C.1 ~ C.5.

Table C.1 Estimated cost analysis of cofferdams

COST ANALYSIS TABLE FOR RCC PRE-COFFERDAM, UPSTREAM AND DOWNSTREAM COFFERDAMS					
UNIT PRICE CODE	DEFINITION OF THE WORK	QUANTITY	UNIT	UNIT PRICE (DUC)	TOTAL (DUC)
GKL-08	Placement of excavated rock foundation within the rockfill and haulage for 1 km . (For pre-cofferdam)	1,170	m^3	2.66	3,118
GKL-08	Placement of excavated rock foundation within the rockfill and haulage for 1 km . (For downstream cofferdam)	1,089	m3	2.66	3,118
GKL-12	Sluicing and compaction of rockfill material.	2,259	m^3	0.58	1,312
GKL-19	Preparation and placement of roller compacted concrete	16,000	m^3	46.53	744,418
			SUB TOTAL =	751,965	
TOTAL COST OF COFFERDAMS (DUC) = 751,965					

Table C.2 Estimated cost analysis of diversion tunnel

COST ANALYSIS TABLE FOR RCC DIVERSION TUNNEL (L=120 m)					
UNIT PRICE CODE	DEFINITION OF THE WORK	QUANTITY	UNIT	UNIT PRICE (DUC)	TOTAL (DUC)
GKL-02	Excavation of rock foundation and haulage of rock for 1 km.	2,030	m^3	11.25	22,843
	Increase of excavation cost by 1% per 100 m of tunnel length exceeding 300 m	0		0.01	0
GKL-14	Preparation of aggregate mixed in concrete mortar and haulage for 23 km .	975	m^3	9.94	9,689
GKL-15	Supply of cement mixed in concrete mortar and haulage for 199 km.	234	ton	182.70	42,752
GKL-16	Supply of construction steel and haulage for 499 km.	62	ton	1,507.49	94,067
GKL-17	Preparation and placement of concrete	780	m^3	294.11	229,406
GKL-19	Preparation and placement of roller compacted concrete	610	m	161.04	98,234
			SUB TOTAL =	496,992	
TOTAL COST OF DIVERSION TUNNEL (DUC) = 621,240					

Table C.3 Estimated cost analysis of spillway

COST ANALYSIS TABLE FOR RCC SPILLWAY					
UNIT PRICE CODE	DEFINITION OF THE WORK	QUANTITY	UNIT	UNIT PRICE (DUC)	TOTAL (DUC)
GKL-05	Placement of surface protection from rockfill and haulage for 2.5 km.	2,450	m^3	16.19	39,653
GKL-14	Preparation of aggregate mixed in concrete mortar and haulage for 23 km .	16,563	m^3	9.94	164,591
GKL-15	Supply of cement mixed in concrete mortar and haulage for 199 km.	3,975	ton	182.70	726,244
GKL-16	Supply of construction steel and haulage for 499 km.	795	ton	1,507.49	1,198,451
GKL-17	Preparation and placement of concrete	13,250	m^3	294.11	3,896,958
		SUB TOTAL =	5,986,244		
TOTAL COST OF SPILLWAY (DUC) = 5,986,244					

Table C.4 Estimated cost analysis of dam body

COST ANALYSIS TABLE FOR RCC DAM BODY					
UNIT PRICE CODE	DEFINITION OF THE WORK	QUANTITY	UNIT	UNIT PRICE (DUC)	TOTAL (DUC)
GKL-03	Preparation of embankment foundation for placement of fill material.	81,750	m^3	7.93	648,415
GKL-14	Preparation of aggregate mixed in concrete mortar and haulage for 23 km. (For upstream covering and handrail)	2,138	m^3	9.94	21,242
GKL-15	Supply of cement mixed in concrete mortar and haulage for 199 km.	513	ton	182.70	93,727
GKL-16	Supply of construction steel and haulage for 499 km. (For handrail on the crest)	2	ton	1,507.49	2,261
GKL-17	Preparation and placement of concrete for upstream covering. (For upstream covering and handrail)	1,710	m^3	294.11	502,928
GKL-19	Preparation and placement of roller compacted concrete	840,000	m3	46.53	39,081,943
		SUB TOTAL =		40,350,515	
TOTAL COST OF DAM BODY(DUC) = 40,350,515					

Table C.5 Estimated cost analysis of grouting

COST ANALYSIS TABLE FOR RCC GROUTING					
UNIT PRICE CODE	DEFINITION OF THE WORK	QUANTITY	UNIT	UNIT PRICE (DUC)	TOTAL (DUC)
GKL-18	Grouting of every kinds and classes of formation	8,700	m	161.04	1,401,045
		SUB TOTAL =	1,401,045		
TOTAL COST OF GROUTING (DUC) = 1,401,045					

Table C.6 Total investment cost of RCC formulation

		Toplam	1. "6 Ay"	2. "6 Ay"	3. "6 Ay"	4. "6 Ay"
COFFERDAMS	(1)	751 965	751 965	-	-	-
DIVERSION TUNNEL	(2)	621 240	621 240	-	-	-
SPILLWAY	(3)	5 986 244	-	2 394 497	3 591 746	-
DAM BODY	(4)	40 350 515	-	17 293 078	17 293 078	5 764 359
GROUTING	(5)	1 401 045	700 523	700 523	-	-
IRRIGATION AND DRAINAGE FACILITIES	(6)	29 966 247	8 989 874	8 989 874	8 989 874	2 996 625
ESTIMATED COST	**(7)=(1)+...+(6)**	**79 077 256**	**11 063 602**	**29 377 972**	**29 874 698**	**8 760 984**
CONTINGENCY	(8)=(7)*0.15	11 861 588	1 659 540	4 406 696	4 481 205	1 314 148
CONSTRUCTION COST	**(9)=(7)+(8)**	**90 938 845**	**12 723 142**	**33 784 668**	**34 355 903**	**10 075 132**
PROJECT CONTROL	(10)=(9)*0.15	13 640 827	1 908 471	5 067 700	5 153 385	1 511 270
EXPROPRIATING	(11)	408 750	408,750			
PROJECT COST	**(12)=(9)+(10)+(11)**	**104 988 422**	**15 040 363**	**38 852 368**	**39 509 289**	**11 586 401**
INTEREST DURING CONSTRUCTION	(13)(*)	6 753 282	1 541 637	2 950 054	1 975 464	286 127
PERIODICAL CUMMULATIVE DEBT	(14)		16 582 000	58 384 423	99 869 176	111 741 704
TOTAL INVESTMENT COST	**(15)=(12)+(13)**	**111 741 704**	**16 582 000**	**41 802 422**	**41 484 753**	**11 872 528**

Table C.7 Annual project expense of RCC formulation

INTEREST RATE : %5.00

ALL UNITS ARE IN DUC

No.	TYPE OF WORK	CONST. COST	CONTINGENCY	COST	INVESTMENT COST	INTEREST & AMORT. FACTOR	RENEWAL FACTOR	OPER. & MAINT. FACTOR	ANNUAL COST			
									INTEREST & AMORT.	RENEWAL	OPER. & MAINT.	TOTAL
		(DUC)							(DUC)			
1	COFFERDAMS	751 965	112 795	864 760	920 385	0.05478	0.0001300	0.002000	50 419	112	1 730	52 261
2	DIVERSION TUNNEL	621 240	93 186	714 426	760 380	0.05478	0.0001300	0.005000	41 654	93	3 572	45 319
3	SPILLWAY	5 986 244	897 937	6 884 180	7 326 999	0.05478	0.0001300	0.010000	401 373	895	68 842	471 110
4	DAM BODY	40 350 515	6 052 577	46 403 092	49 387 928	0.05478	0.0001300	0.002000	2 705 471	6 032	92 806	2 804 309
5	GROUTING	1 401 045	210 157	1 611 202	1 714 841	0.05478	0.0001300	0.020000	93 939	209	32 224	126 373
6	IRRIGATION AND DRAINAGE FACILITIES	29 966 247	4 494 937	34 461 184	36 677 868	0.05478	0.0032300	0.019900	2 009 214	111 310	685 778	2 806 301
7	**CONSTRUCTION COST**			**90 938 845**					**5 302 069**	**118 652**	**884 951**	**6 305 672**
8	PROJECT CONTROL			13 640 827	14 518 260	0.05478	-		795 310			795 310
9	EXPROPRIATING			408 750	435 042	0.05478	-		23 832			23 832
10	**PROJECT COST**			**104 988 422**					**6 121 211**		**884 951**	**7 124 814**
11	INTEREST DURING CONSTRUCTION			6 753 282			-					
12	**TOTAL INVESTMENT COST**			**111 741 704**	**111 741 704**				**6 121 211**	**118 652**	**884 951**	**7 124 814**

Table C.8 Rantability of RCC formulation

YEAR	EXPENSE			INCOME			PRESENT VALUE	
	PROJECT COST	OPERATION MAINT., RENEW	TOTAL	EXISTING SITUATION	FUTURE SITUATION	INCOME INCREASE	EXPENSE	INCOME
(n)	(DUC)							
	(1)	**(2)**	**(3)=(1)+(2)**	**(4)**	**(5)**	**(6)**	**(7)=(3)/1.05^n**	**(8)=(6)/1.05^n**
1	53 892 732		53 892 732				51 326 411	
2	51 095 690	334 534	51 430 224	4 883 000	6 802 789	1 919 789	46 648 730	1 741 305
3	0	1 003 603	1 003 603	14 649 000	20 408 367	5 759 367	866 950	4 975 158
4	0	1 003 603	1 003 603	14 649 000	27 011 432	12 362 432	825 667	10 170 603
5	0	1 003 603	1 003 603	14 649 000	29 533 498	14 884 498	786 349	11 662 394
6	0	1 003 603	1 003 603	14 649 000	31 784 128	17 135 128	748 904	12 786 496
7	0	1 003 603	1 003 603	14 649 000	32 199 195	17 550 195	713 242	12 472 596
8	0	1 003 603	1 003 603	14 649 000	32 588 462	17 939 462	679 278	12 142 134
9	0	1 003 603	1 003 603	14 649 000	33 273 369	18 624 369	646 931	12 005 434
10	0	1 003 603	1 003 603	14 649 000	33 813 559	19 164 559	616 125	11 765 377
11	0	1 003 603	1 003 603	14 649 000	34 158 292	19 509 292	586 786	11 406 679
12	0	1 003 603	1 003 603	14 649 000	34 294 200	19 645 200	558 844	10 939 182
13	0	1 003 603	1 003 603	14 649 000	34 294 200	19 645 200	532 232	10 418 269
14	0	1 003 603	1 003 603	14 649 000	34 294 200	19 645 200	506 888	9 922 161
15	0	1 003 603	1 003 603	14 649 000	34 294 200	19 645 200	482 750	9 449 677
16	0	1 003 603	1 003 603	14 649 000	34 294 200	19 645 200	459 762	8 999 692
17	0	1 003 603	1 003 603	14 649 000	34 294 200	19 645 200	437 869	8 571 136
18	0	1 003 603	1 003 603	14 649 000	34 294 200	19 645 200	417 018	8 162 986
19	0	1 003 603	1 003 603	14 649 000	34 294 200	19 645 200	397 160	7 774 273
20	0	1 003 603	1 003 603	14 649 000	34 294 200	19 645 200	378 247	7 404 069
21	0	1 003 603	1 003 603	14 649 000	34 294 200	19 645 200	360 236	7 051 495
22	0	1 003 603	1 003 603	14 649 000	34 294 200	19 645 200	343 082	6 715 709
23	0	1 003 603	1 003 603	14 649 000	34 294 200	19 645 200	326 744	6 395 913
24	0	1 003 603	1 003 603	14 649 000	34 294 200	19 645 200	311 185	6 091 346
25	0	1 003 603	1 003 603	14 649 000	34 294 200	19 645 200	296 367	5 801 282
26	0	1 003 603	1 003 603	14 649 000	34 294 200	19 645 200	282 254	5 525 030
27	0	1 003 603	1 003 603	14 649 000	34 294 200	19 645 200	268 813	5 261 934
28	0	1 003 603	1 003 603	14 649 000	34 294 200	19 645 200	256 013	5 011 365
29	0	1 003 603	1 003 603	14 649 000	34 294 200	19 645 200	243 822	4 772 729
30	0	1 003 603	1 003 603	14 649 000	34 294 200	19 645 200	232 211	4 545 456
31	0	1 003 603	1 003 603	14 649 000	34 294 200	19 645 200	221 153	4 329 006
32	0	1 003 603	1 003 603	14 649 000	34 294 200	19 645 200	210 622	4 122 863
33	0	1 003 603	1 003 603	14 649 000	34 294 200	19 645 200	200 593	3 926 536
34	0	1 003 603	1 003 603	14 649 000	34 294 200	19 645 200	191 041	3 739 558
35	0	1 003 603	1 003 603	14 649 000	34 294 200	19 645 200	181 943	3 561 484
36	0	1 003 603	1 003 603	14 649 000	34 294 200	19 645 200	173 279	3 391 889
37	0	1 003 603	1 003 603	14 649 000	34 294 200	19 645 200	165 028	3 230 371
38	0	1 003 603	1 003 603	14 649 000	34 294 200	19 645 200	157 170	3 076 544
39	0	1 003 603	1 003 603	14 649 000	34 294 200	19 645 200	149 685	2 930 042
40	0	1 003 603	1 003 603	14 649 000	34 294 200	19 645 200	142 557	2 790 516
41	0	1 003 603	1 003 603	14 649 000	34 294 200	19 645 200	135 769	2 657 634
42	0	1 003 603	1 003 603	14 649 000	34 294 200	19 645 200	129 304	2 531 080
43	0	1 003 603	1 003 603	14 649 000	34 294 200	19 645 200	123 147	2 410 552
44	0	1 003 603	1 003 603	14 649 000	34 294 200	19 645 200	117 282	2 295 764
45	0	1 003 603	1 003 603	14 649 000	34 294 200	19 645 200	111 698	2 186 442
46	0	1 003 603	1 003 603	14 649 000	34 294 200	19 645 200	106 379	2 082 326
47	0	1 003 603	1 003 603	14 649 000	34 294 200	19 645 200	101 313	1 983 167
48	0	1 003 603	1 003 603	14 649 000	34 294 200	19 645 200	96 489	1 888 731
49	0	1 003 603	1 003 603	14 649 000	34 294 200	19 645 200	91 894	1 798 791
50	0	1 003 603	1 003 603	14 649 000	34 294 200	19 645 200	87 518	1 713 135
51	0	1 003 603	1 003 603	14 649 000	34 294 200	19 645 200	83 350	1 631 557
52	0	669 069	669 069	9 766 000	22 862 800	13 096 800	52 921	1 035 909
	104,988,422						**114 567 003**	**299 255 779**

TOTAL INCOME / TOTAL EXPENSE : 2.612

Table C.9 Internal rate of return for RCC formulation

YEAR	EXPENSE			INCOME INCREASE	DIFFERENCE	PRESENT VALUE
	PROJECT COST	OPREATION & MAINTENANCE	TOTAL			**IRR = 13.69%**
	(DUC)			(DUC)		
	(1)	**(2)**	**(3)=(1)+(2)**	**(4)**	**(5)=(4)-(3)**	**(6)**
1	53 892 732		53 892 732		-53 892 732	-47 401 490
2	51 095 690	334 534	51 430 224	1 919 789	-49 510 435	-38 301 902
3	0	1 003 603	1 003 603	5 759 367	4 755 764	3 235 979
4	0	1 003 603	1 003 603	12 362 432	11 358 829	6 797 993
5	0	1 003 603	1 003 603	14 884 498	13 880 895	7 306 787
6	0	1 003 603	1 003 603	17 135 128	16 131 525	7 468 720
7	0	1 003 603	1 003 603	17 550 195	16 546 592	6 738 157
8	0	1 003 603	1 003 603	17 939 462	16 935 859	6 065 988
9	0	1 003 603	1 003 603	18 624 369	17 620 766	5 551 124
10	0	1 003 603	1 003 603	19 164 559	18 160 956	5 032 186
11	0	1 003 603	1 003 603	19 509 292	18 505 689	4 510 088
12	0	1 003 603	1 003 603	19 645 200	18 641 597	3 995 992
13	0	1 003 603	1 003 603	19 645 200	18 641 597	3 514 685
14	0	1 003 603	1 003 603	19 645 200	18 641 597	3 091 351
15	0	1 003 603	1 003 603	19 645 200	18 641 597	2 719 005
16	0	1 003 603	1 003 603	19 645 200	18 641 597	2 391 508
17	0	1 003 603	1 003 603	19 645 200	18 641 597	2 103 457
18	0	1 003 603	1 003 603	19 645 200	18 641 597	1 850 101
19	0	1 003 603	1 003 603	19 645 200	18 641 597	1 627 261
20	0	1 003 603	1 003 603	19 645 200	18 641 597	1 431 262
21	0	1 003 603	1 003 603	19 645 200	18 641 597	1 258 870
22	0	1 003 603	1 003 603	19 645 200	18 641 597	1 107 242
23	0	1 003 603	1 003 603	19 645 200	18 641 597	973 878
24	0	1 003 603	1 003 603	19 645 200	18 641 597	856 577
25	0	1 003 603	1 003 603	19 645 200	18 641 597	753 404
26	0	1 003 603	1 003 603	19 645 200	18 641 597	662 659
27	0	1 003 603	1 003 603	19 645 200	18 641 597	582 843
28	0	1 003 603	1 003 603	19 645 200	18 641 597	512 641
29	0	1 003 603	1 003 603	19 645 200	18 641 597	450 895
30	0	1 003 603	1 003 603	19 645 200	18 641 597	396 586
31	0	1 003 603	1 003 603	19 645 200	18 641 597	348 818
32	0	1 003 603	1 003 603	19 645 200	18 641 597	306 804
33	0	1 003 603	1 003 603	19 645 200	18 641 597	269 850
34	0	1 003 603	1 003 603	19 645 200	18 641 597	237 347
35	0	1 003 603	1 003 603	19 645 200	18 641 597	208 759
36	0	1 003 603	1 003 603	19 645 200	18 641 597	183 615
37	0	1 003 603	1 003 603	19 645 200	18 641 597	161 499
38	0	1 003 603	1 003 603	19 645 200	18 641 597	142 047
39	0	1 003 603	1 003 603	19 645 200	18 641 597	124 938
40	0	1 003 603	1 003 603	19 645 200	18 641 597	109 889
41	0	1 003 603	1 003 603	19 645 200	18 641 597	96 653
42	0	1 003 603	1 003 603	19 645 200	18 641 597	85 012
43	0	1 003 603	1 003 603	19 645 200	18 641 597	74 772
44	0	1 003 603	1 003 603	19 645 200	18 641 597	65 766
45	0	1 003 603	1 003 603	19 645 200	18 641 597	57 845
46	0	1 003 603	1 003 603	19 645 200	18 641 597	50 877
47	0	1 003 603	1 003 603	19 645 200	18 641 597	44 749
48	0	1 003 603	1 003 603	19 645 200	18 641 597	39 359
49	0	1 003 603	1 003 603	19 645 200	18 641 597	34 619
50	0	1 003 603	1 003 603	19 645 200	18 641 597	30 449
51	0	1 003 603	1 003 603	19 645 200	18 641 597	26 781
52	0	669 069	669 069	13 096 800	12 427 731	15 704
TOPLAM						**0**