Bulletin XXX

Selection of Dam Type

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SOMMAIRE	**CONTENTS**

	Foreword
1	Introduction
2	Factors required for outline designs
3	Concrete dams
4	Embankment dams
5	Spillways
6	Diversions
7	Evaluation of Technically Feasible Dam Types

ACRONYMS AND ABBREVIATIONS

ABL	Asphaltic Binder Layer
ACRD	Asphaltic Core Rockfill Dam
AFRD	Asphalt Faced Rockfill Dam
CFRD	Concrete Faced Rockfill Dam
CVC	Conventionally Compacted (Vibrated) Concrete
DAC	Dense Asphaltic Concrete
ECRD	Earth Core Rockfill Dam
GLOF	Glacial Lake Outburst Food
ICOLD	International Commission on Large Dams
ISRM	International Society for Rock Mechanics
MCE	Maximum Credible Earthquake
PMF	Probable Maximum Flood
RCC	Roller Compacted Concrete
RTS	Reservoir Triggered Seismicity
WMO	World Meteorological Organisation
UCS	Uniaxial compressive strength

FOREWORD

TO BE PREPARED LATER (2019 03 08)

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This Bulletin was made by the ICOLD Committee for Selection of Dam type under the Chairmanship of O J Berthelsen (Norway). The first Vice-chairman was K Ferguson (USA) who was succeeded by G Tarbox (USA and then A Marulanda (Colombia).

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TABLE OF CONTENTS

FOREWORD

ACKNOWLEDGEMENTS

1 INTRODUCTION 1

2 INFORMATION REQUIRED FOR OUTLINE DESIGNS 3

2.1 INTRODUCTION 3

2.2 TOPOGRAPHY 3

2.3 CLIMATE 4

2.4 GEOLOGY AND FOUNDATION CONDITIONS 4

2.4.1 PRIMARY CONSIDERATIONS 4

2.4.2 INVESTIGATIONS 5

2.4.3 GEOLOGICAL HAZARDS 7

2.4.4 CONSTRUCTION EXCAVATIONS 7

2.5 CONSTRUCTION MATERIALS AVAILABILITY 7

2.6 OPERATIONAL REQUIREMENTS 8

2.6.1 INTRODUCTION 8

2.6.2 FREEBOARDS 8

2.6.3 FLOOD SURCHARGE 8

2.6.4 RESERVOIR OPERATION RANGE 8

2.6.5 BOTTOM (LOW LEVEL) OUTLETS 9

2.6.6 ENVIRONMENTAL FLOW REQUIREMENTS 9

2.6.7 RESERVOIR SEDIMENTATION 10

2.6.8 STAGED FILLING 10

2.7 FLOOD HAZARDS 10

2.7.1 INTRODUCTION 10

2.7.2 REPORT ON HYDROLOGY 10

2.7.3 SELECTION OF SPILLWAY TYPE 11

2.8 EARTHQUAKE AND OTHER EXTERNAL GEOLOGIC HAZARDS 11

2.8.1 INTRODUCTION 11

2.8.2 EARTHQUAKE RISK ESTIMATES 11

2.8.3 DESIGN METHODS FOR THE DAMS 11

2.8.4 ACTIVE FAULTS IN THE FOUNDATION....12
2.8.5 RESERVOIR TRIGGERED SEISMICITY....12
2.8.6 LANDSLIDES AND RESERVOIRS....12
2.8.7 RESERVOIR LEAKAGE....12
2.9 ENVIRONMENTAL AND SOCIO-ECONOMIC IMPACTS....12
3 CONCRETE DAMS....13
3.1 INTRODUCTION....13
3.2 GRAVITY DAMS....14
3.2.1 INTRODUCTION....14
3.2.2 DAM CROSS-SECTION....15
3.2.3 STABILITY AND STRESS ANALYSIS....16
3.2.4 FOUNDATION STIFFNESS AND STRENGTH....17
3.2.5 ACCEPTANCE CRITERIA....17
3.2.6 HORIZONTAL ALIGNMENT....18
3.2.7 FOUNDATION TREATMENT....18
3.2.8 UPLIFT CONTROL....18
3.2.9 GALLERIES....19
3.2.10 CONCRETE PROPERTIES....20
3.2.11 SPILLWAYS....21
3.2.12 DIVERSION....21
3.3 CEMENTED MATERIALS DAMS (CMD)....21
3.3.1 INTRODUCTION....21
3.3.2 TYPICAL CROSS-SECTION....22
3.3.3 STABILITY AND STRESS ANALYSIS....22
3.3.4 MATERIALS....23
3.3.5 OVERTOPPING....23
3.3.6 THERMAL EFFECTS....23
3.3.7 EFFECTS OF SEISMICITY....23
3.3.8 SPILLWAY....24

3.4 GRAVITY DAMS AND CMD ON DRIFT 24
3.5 ARCH DAMS 25
3.5.1 INTRODUCTION 25
3.5.2 VALLEY SHAPE AND DAM CURVATURE IN PLAN 26
3.5.3 FOUNDATION REQUIREMENTS 28
3.5.4 THE DAM IN CROSS-SECTION 28
3.5.5 OUTLINE DESIGN 29
3.5.6 INTERNAL DRAINAGE AND UPLIFT CONTROL 34
3.5.7 FOUNDATION TREATMENT 35
3.5.8 COOLING CONCRETE 35
3.5.9 JOINTS AND JOINT GROUTING 35
3.5.10 CONCRETE PROPERTIES 35
3.5.11 SPILLWAYS AND OUTLETS 36
3.5.12 DIVERSION 36
3.5.13 ACCESS 36
3.6 RUBBLE MASONRY CONCRETE DAMS 37
3.7 METHODS OF CONSTRUCTION AND MATERIALS REQUIREMENTS 39
3.7.1 FOUNDATION SURFACE TREATMENT 39
3.7.2 CVC 39
3.7.3 RCC 39
3.7.4 JOINTS 40
3.7.5 MATERIALS 41
3.7.6 RCC PRODUCTION AND MATERIAL STORAGE 41
3.8 CONSTRUCTION SCHEDULE 43
3.9 MAJOR COST ELEMENTS 44
3.10 REFERENCES 45
4 EMBANKMENT DAMS 47
4.1 INTRODUCTION 47
4.2 COMPOSITE DAMS 48
4.3 EARTHFILL DAMS 50
4.3.1 OVERVIEW 50

4.3.2 DESIGN PRINCIPLES....51
4.3.3 MAKING A PRACTICAL OUTLINE DESIGN....54
4.4 EARTH CORE ROCKFILL DAMS, ECRD....59
4.4.1 INTRODUCTION....59
4.4.2 MATERIAL ZONES....59
4.4.3 DETERMINING THE EXTERIOR SLOPES OF THE DAM....59
4.5 CONCRETE FACED ROCKFILL DAMS, CFRD....59
4.5.1 INTRODUCTION....59
4.5.2 EMBANKMENT SLOPES....60
4.5.3 EMBANKMENT MATERIALS AND ZONES....60
4.5.4 THE PLINTH....61
4.5.5 CONCRETE FACING....62
4.5.6 FOUNDATION....62
4.6 ASPHALTIC CORE EMBANKMENT DAMS, ACED....62
4.6.1 INTRODUCTION....62
4.6.2 TYPICAL DAM CROSS-SECTION....63
4.6.3 ASPHALTIC CORE PLACEMENT....64
4.6.4 MATERIALS FOR CORE AND FILTERS....64
4.7 ASPHALT FACED ROCKFILL DAM, AFRD....64
4.7.1 DAM CROSS-SECTION....64
4.7.2 THE PLINTH....65
4.7.3 THE BITUMINOUS LINING....65
4.8 GEOMEMBRANE FACE OR CORE DAMS....66
4.8.1 GENERAL CONSIDERATION....66
4.8.2 TYPICAL CONSTRUCTION DETAILS....67
4.9 ASPECTS COMMON TO EMBANKMENT DAMS....69
4.9.1 FREEBOARDS....69
4.9.2 EARTHQUAKE SETTLEMENTS....70
4.9.3 CAMBER: ESTIMATING SETTLEMENTS....71

4.9.4 MATERIALS 71
4.9.5 SPILLWAYS 72
4.9.6 SLOPE PROTECTION 72
4.9.7 EFFECT OF FOUNDATION STRENGTH ON EMBANKMENT SLOPES 72
4.9.8 DETERMINATION OF EMBANKMENT SLOPES 72
4.9.9 FILTERS, DRAINS AND TRANSITIONS 73
4.9.10 FOUNDATION REQUIREMENTS AND TREATMENT 73
4.9.11 EFFECTS OF CLIMATE 74
4.9.12 RISKS 75
4.9.13 RATE OF CONSTRUCTION, SCHEDULE 76
4.10 COST-DRIVING ELEMENTS 77
4.11 REFERENCES 78
5 SPILLWAYS 80
5.1 GENERAL/INTRODUCTION 80
5.2 ICOLD BULLETINS 80
5.3 ECONOMICS, RISK AND SAFETY ISSUES IN SPILLWAY DESIGN 81
5.4 SELECTING SPILLWAY FLOOD CAPACITY 81
5.5 SPILLWAY CLASSIFICATION 81
5.6 SPILLWAY COMPONENTS AND TYPES 82
5.6.1 COMPONENTS 82
5.6.2 SPILLWAY COMPONENT TYPES 82
5.6.3 DESIGN CONSIDERATIONS 83
5.6.4 STEPPED SPILLWAY CHUTES 84
5.7 ENERGY DISSIPATION 85
5.8 FACTORS INFLUENCING SPILLWAY SELECTION 86
5.8.1 BACKGROUND 86
5.8.2 SPILLWAYS FOR EMBANKMENT DAMS 86
5.8.3 SPILLWAYS FOR CONCRETE DAMS 88
5.8.4 GATED OR UNGATED SPILLWAYS 88
5.8.5 SPILLWAY CAPACITY OPTIMISATION 89
5.9 COST CONSIDERATIONS 89

5.10 REFERENCES 90
6 DIVERSIONS 92
6.1 INTRODUCTION 92
6.2 ROLE OF RIVER DIVERSION IN SELECTION OF DAM TYPE 92
6.3 TYPES OF DIVERSION 93
6.3.1 SINGLE STAGE DIVERSION 93
6.3.2 MULTI STAGE DIVERSION 93
6.3.3 CONSTRUCTION ABOVE RIVER LEVEL 93
6.4 APPLICABILITY TO DAM TYPE 94
6.4.1 FACTORS TO BE CONSIDERED 94
6.4.2 EMBANMENT DAMS 94
6.4.3 CONCRETE DAMS 94
6.4.4 INTEGRATED COFFERDAMS 94
6.5 INCLUSION IN ANALYSIS OF TYPE OF DAM 94
7 EVALUATION OF TECHNICALLY FEASIBLE DAM TYPES 96
7.1 INTRODUCTION 96
7.2 QUANTITIES 96
7.3 ALLOWANCES FOR CONSTRUCTION ITEMS NOT EXPLICITLY QUANTIFIED 97
7.4 UNIT RATES 97
7.4.1 DATABASES 97
7.4.2 CONTRACTOR TYPE ESTIMATES 97
7.5 ITEMS NOT INCLUDED IN THE ESTIMATES 98
7.6 CONTINGENCIES 98
7.7 COST ASSOCIATED WITH CONSTRUCTION SCHEDULE 98
7.8 ACCURACY OF ESTIMATED COSTS 98
7.9 METHODS OF COST COMPARISON 99
7.9.1 DETERMINISTIC METHODS 99
7.9.2 PROBABILISTIC METHODS 99
7.10 MULTI-CRITERIA EVALUATION 103

FIGURES

Figure 2-1: Elements of a reservoir (after Bulletin 31) ... 9

Figure 3-1 Gravity dam of RCC (Rialb, Spain: Height 99 m, Volume 1.02 Mm^3) ... 15

Figure 3-2 Schematic cross-section of a gravity dam ... 16

Figure 3-3 Schematic cross-section for seismic loads ... 16

Figure 3-4 Long section of dam with horizontal galleries ... 19

Figure 3-5 Long section of dam with foundation gallery ... 20

Figure 3-6 Typical gravity dam cross-section showing galleries ... 20

Figure 3-7 Non-overflow cross-section of Contraembalse de Monció ... 22

Figure 3-8 Distribution of vertical stress over bottom of dam ... 23

Figure 3-9 Capillucas Dam on deep alluvium (Platanal HEP in Peru) ... 24

Figure 3-10: Gravity dam on dense silt ... 25

Figure 3-11 Double curvature arch dam in CVC (Atazar, Spain, 130 m high) ... 26

Figure 3-12 Definitions for valley shape ... 27

Figure 3-13 Definition of arch dam with circular axis ... 28

Figure 3-14 Concrete dam cross-sections ... 29

Figure 3-15 Definition of crown cantilever ... 31

Figure 3-16 Schematic plan of dam at an elevation, a) constant thickness dam and b) variable thickness dam ... 32

Figure 3-17 Reference plane with lines of centres for extrados and intrados for a single centre and variable thickness dam ... 33

Figure 3-18 Schematic plan of dam at an elevation for an asymmetrical dam site ... 33

Figure 3-19 Slenderness coefficient versus height ... 34

Figure 3-20 Section of an arch dam showing crest spillway (gated or un-gated) and orifice spillway ... 37

Figure 3-21 Rubble Masonry Concrete Arch dam, 42 m high (Lucilia Poort, Zimbabwe) ... 38

Figure 3-22 Possible internal zoning of an RCC gravity dam ... 40

Figure 4-1 Composite dam: embankment wrapped around concrete gravity dam ... 48

Figure 4-2 Wrap-around junction between embankment and gravity dam ... 49

Figure 4-3 Connection between gravity and embankment dam with retaining walls ... 50

Figure 4-4 Homogeneous dam with internal drainage ... 51

Figure 4-5 Core locations in zoned dams ... 52

Figure 4-6 Earthfill dam with integrated cofferdam ... 54

Figure 4-7 A zoned embankment dam, central core ... 55

Figure 4-8: Zoned embankment dam with sloping core ... 56
Figure 4-9: Zones for CFRD of sound rock fill and sound rock foundation ... 61
Figure 4-10: Typical Internal Toe Slab Cross-section ... 62
Figure 4-11. Typical cross-section of ACRD ... 63
Figure 4-12 Plinth for ACRD ... 63
Figure 4-13 Cross-Section of an AFRD ... 66
Figure 4-14 Typical membrane installations (from Bull. 135) ... 67
Figure 4-15 Typical composition of membrane system for an earthfill dam ... 68
Figure 4-16 Typical composition of membrane system for a rockfill dam ... 68
Figure 4-17 Function and type of the various layers ... 69
Figure 4-18: Chart for estimating earthquake-induced settlements ... 70
Figure 5-1 Spillway components ... 82
Figure 5-2 Common spillway components ... 83
Figure 5-3: Embankment dam with spillway in adjacent valley ... 87
Figure 5-4 Harding Dam; unlined spillway was used as a source of rockfill ... 90
Figure 7-1 Probability density function for triangular distribution ... 99
Figure 7-2 Histogram of values from Monte Carlo simulation ... 102

TABLES

Table 2-1 Recommended map scales ... 3
Table 2-2: Geological and geotechnical data collection for a dam site ... 6
Table 3-1: Concrete dam types ... 14
Table 3-2 Typical acceptance criteria for gravity dam stability and stress analyses ... 18
Table 3-3 Rocha (1964) Structural Influence of E_c/E_m Ratio for Arch Dams ... 28
Table 3-4 RCC placement rates ... 42
Table 3-5 Major cost elements ... 44
Table 4-1 Average and maximum rockfill placement rates for several dams ... 76
Table 4-2 Average and maximum placement rates for several earthfill dams ... 77
Table 5-1 ICOLD Bulletins dealing with the design of spillways ... 80
Table 5-2 Energy dissipated for a range of step heights, unit discharges and heads ... 85
Table 7-1 Bill of quantities for Monte Carlo simulations ... 101

1 INTRODUCTION

A dam site is normally suited to more than one dam type and a choice of dam type has to be made. This bulletin provides guidance on the selection process. The purpose of this process is to ensure that the most economic, safe and serviceable dam type is constructed where risk factors and environmental impacts are taken into account.

The principal dam types currently in use for large dams are included.

The preparation of this Bulletin was prompted by the many instances of inappropriate choice of dam type caused by misconceptions about technical suitability, a tendency to employ previously used concepts uncritically, by out-of-date design criteria or regulations and designers who have imperfect knowledge of the merits of alternatives. Lack of rigour in the selection process is a further factor that can lead to an inappropriate choice.

ICOLD Bulletin 144, Cost Savings in Dams contains much information on most dam types related to cost savings by adopting suitable designs and construction methods. Bulletin 144 indirectly provides information which relates to the selection of dam type. This Bulletin develops the themes in Bulletin 144 by giving more detail about the suitability of each dam type.

The choice of dam type for any one particular site should be based on the evaluation of a range of dam types with selection of the optimal dam type based on risk factors, costs and environmental impacts. The risk factors may include hydrological risk (uncertainties in the spillway design flood estimate), geological risks (imperfect knowledge of conditions at the time of dam type selection) and cost risks (unusually large uncertainties in expected tender prices). Spillways, diversions and other structures associated with the dam as well as quantity and suitability of locally available construction materials may have a significant impact on the choice and have to be included in the development of each of the options. The optimal dam type may vary with required reservoir elevation, alternatively this may be a factor in the optimisation of a scheme.

The Bulletin contains a section on information required to develop dam designs of a dam such as topography, climate, hydrology, geology and materials availability. Indications are given of the minimum information required to make a preliminary choice for pre-feasibility design and definitive choices at later stages of project development. This is followed by descriptions of each dam type with indications of conditions where it is suitable and the implications of conditions that deviate from this ideal. Diversions and spillways are covered in separate chapters. The final section gives an outline of how to resolve the choice between technically feasible dam types. The inclusion of various uncertainties in the evaluations is considered. Advice is given on how to treat cases where the risk and cost data are such that it is not possible to rank the dam types in order of preference.

The text gives guidance on how to make an outline design based on limited data and the minimal design effort compatible with the overall objective of dam type selection. Design aspects that do not have a significant impact on costs are not discussed in detail but references to sources of design information are given. There will be cases where the minimum design effort will not suffice and these sources will have to be consulted.

Dam type selection is commonly made early in project development, such as in a pre-feasibility study, when limited data is available. As the quantity and quality of data improves, this initial selection should be revisited and revised as necessary. This should be done in any new phase of project development: feasibility study, tender design and maybe also around the time of tender evaluation.

There are a number of dam types that have become rare in modern construction such as multiple arch dams, buttress dams and others that have been displaced by RCC gravity dams because of their lower cost. Although such dams may have their place in particular circumstances, they will commonly be technically suitable where a RCC gravity dam is suitable. Many decades ago there was a shift from puddle clay core dams to embankment dams with much stiffer core materials with the advent of heavy vibrating roller compactors that allowed their compaction. Masonry gravity dams are still constructed in some countries where labour costs are low.

This Bulletin does not include tailings-dams or dams associated with flood defences (levees).

Cofferdams are dealt with only as components of river diversions. Their type, height and cost may be affected by main dam type but the selection of cofferdam type beyond this is not included.

2 INFORMATION REQUIRED FOR OUTLINE DESIGNS

2.1 INTRODUCTION

The development of a water resources project such as a large reservoir and its dam and appurtenant structures requires consideration of a wide range of issues including environmental, social, economic and technical aspects.

This chapter covers the information required for the selection of an appropriate dam type and appurtenant structure arrangement and not on the wider effects of the reservoir and damming of the waterway. For example, the choice of dam type does not have a primary effect on the deposition of sediment in a reservoir. Sediment issues, however, may have a major impact on the type of dam and the appurtenant structures necessary to enable its passage past the dam if this is a requirement of the project. Similarly, the major socio-economic and environmental aspects of the project will be assessed for the reservoir. These will result in requirements for such features as fish passage and environmental flow releases which will have to be provided within the dam and appurtenant structures.

2.2 TOPOGRAPHY

Topography, to a large measure, dictates the choice of site and type of dam. Accurate topographic maps are essential for dam type selection, structure layouts and cost estimating.

The topography, in conjunction with the geology, determines the physical limits on the reservoir volume and dam height. It forms the base on which geomorphic, geological and geotechnical information will be developed and displayed. Topographic information is required for developing the fundamental relationship between reservoir elevation and reservoir area and volume.

Topographic information is typically obtained from either aerial photography where there is minimal vegetation cover, bathymetric survey for underwater topography, remote sensing technologies such as Lidar which can provide accurate topographic data where there is dense vegetation cover and ground-based surveys. Laser-beam based surveying instruments have been use with success where the site topography lent itself to it. All methods require supplementary ground-based survey and ground proofing.

The topographical maps should include the contours of the river bottom which is normally the province of hydrography.

Different scale maps are used in the course of different phases. It depends upon the project phase and sometimes upon the importance of the dam. In some cases there may be limitations in time or in site accessibility (related technological availability of sophisticated – laser beam based-surveying instruments in inaccessible gorges). Recommended requirements for topographical maps as shown in Table 2-1.

Table 2-1 Recommended map scales

	Range	Recommended minimum	
	Scale	Scale	Contour interval
Reconnaissance:	1:50,000 to 1:10,000	10,000	5 m
Pre-feasibility:	1:10,000 to 1:1,000	2,000	2 m
Feasibility:	1:2,000 to 1:500	1,000	1 m

The topographical survey must cover not only the potential dam site but also a considerable area upstream and downstream of it. Allowance should be made for adjustments to the dam axis, possibly by several hundred metres. The survey must include areas that might be affected by the works such as the diversion, spillways, plunge pools or stilling basins, access roads, construction facilities (batch plants, storage areas, waste storage, borrow areas quarries and camps). A common failing is a survey which is insufficient in extent.

2.3 CLIMATE

Many different types of dams can be built in a wide variety of climates. Extremes of either temperature or rainfall may significantly influence the choice of type of dam. The primary effect of rainfall is on the construction schedule. For concrete dams the ambient temperatures have a major effect on contraction joint spacing, concrete placing temperatures, joint grouting schedule, all influencing cooling demands. Sub-zero temperatures can affect the construction schedule for all dam types.

The meteorological information required for the selection process is in order of importance:

1. Rainfall

Daily maximum and minimum rainfalls for as many years as possible

Hourly records for several years. The latter is not always available and anecdotal evidence may have to be used to give a first estimate of diurnal variations.

2. Temperature

Daily data of maximum and minimum temperatures for as many years as possible. Representative hourly data should be obtained to give the diurnal variation in different seasons.

Such data may be obtainable from meteorological stations which may be in the proximity to the site or remote from it. Where the stations are remote from the site, the applicability to the site can be improved by compensating for topographical effects including elevation. This requires the expertise of a meteorologist.

Consideration might be given to supplementing and calibrating the available information by installing a meteorological station at the site. This may be either a full WMO station or, as a minimum, a temperature recording device which can be set to give hourly temperatures. The data can then be downloaded into a computer at convenient intervals.

The susceptibility of the various dam types to climate effects is described in the relevant sections of Chapters 3 and 4.

2.4 GEOLOGY AND FOUNDATION CONDITIONS

This section makes extensive use of information from ICOLD Bulletin 129 'Dam Foundations: Geologic Considerations. Investigation Methods. Treatment. Monitoring.' This bulletin should be consulted for more detailed information.

2.4.1 PRIMARY CONSIDERATIONS

The dam site will have been selected using topography as the main criterion with geology as a supplement. However, caution is needed as there are dam sites of equal topography (valley profile) but different suitable dam type due to geology.

The dam site selection process is aimed at finding a narrow section of a valley where dam volumes could be expected to be small and where the reservoir volume or extent is optimal. A narrow valley section is commonly associated with the rock being harder and closer to the surface than elsewhere. At sites where alluvium dominates the landscape, there may be a considerable depth to rock and other factors will indicate the best site.

Dam type selection is normally made in a pre-feasibility study and re-assessed at the feasibility stage and possibly later stages. The geological information available increases with time over the development of the project.

Budgets for investigations are commonly restricted at the pre-feasibility stage and decisions have to be made with limited information. Nevertheless, sufficient investigations have to be made at these early stages to characterise the site and allow a sensible determination of dam type. Inadequate investigations are those where ambiguities are present which prevent a rational determination.

Broad regional geological studies form the basis for understanding the geologic processes that have led to the current geological setting and topography together with the regional seismotectonics, geologic structure, stratigraphy, geomorphology and groundwater. These studies may point to the existence of important features such as ancient landslides, faults or folds that may be present but are not otherwise recognised at the dam site during the early stages on investigation.

When combined with the early field reconnaissance and preliminary exploration of the dam site, these studies form the basis of identifying geological hazards that may be present at the site. The higher (the more uncertain) these hazards are the more investigations are required. It is general much more expensive changing a dam type or dam site at a later design stage as a consequence of geological surprises.

Geologic hazards may be a major factor in determining both the initial selection of dam type and the planning of the more detailed site investigations, see Section 2.4.2. The correct identification of rock types present or likely to be present at the dam site and in the reservoir is an essential prerequisite for identifying the potential geologic hazards.

The three most important factors that have to be established are:

1. The depths to a suitable foundation stratum or strata.
2. The presence of material or geological structures which might adversely affect dam stability. This will include any weak or deformable materials in the prospective foundation as well as general joint surveys to discover potentially unstable rock masses.
3. The presence of pervious strata which might give high seepage losses or adverse foundation pore pressures that could be hard to mitigate.

Until such time as the dam type has been selected, the investigations should be focused on revealing the geology of the site rather than properties at locations specific to a particular dam type. Furthermore, at the time of dam type selection there may be adjustments to the location of the dam by a few tens of metres and up to hundreds of metres, all according to the results of the investigations. Such adjustments are made to minimise dam volumes and avoid unfavourable geological features.

2.4.2 INVESTIGATIONS

Geotechnical investigations of dam sites are a staged process as described in ICOLD Bulletin 129 starting with the broad regional and site reconnaissance studies already referred to. Table 2-2 shows the principal elements of the data collection.

Table 2-2: Geological and geotechnical data collection for a dam site

Regional geology	Site geology	Geotechnics/rock mechanics	Hydrogeology
Geological model	Overburden	Strength	Permeability
Stratigraphy	Bedrock	Deformability	Groundwater conditions
Seismic zoning	Weathering, faulting, jointing	Rock mass characterization	
obtained from geological mapping, geophysics, drillings, adits		obtained from laboratory and in-situ testing	

The investigation of a site will typically follow these steps:

1. Evaluation of available information on the regional geology and seismotectonics.
2. Evaluation of aerial photographs and satellite imagery.
3. Walkover surveys and inspections of the dam site and surrounding areas including the slopes above the dam. A prerequisite is a reliable topographic base (Section 2.2) and geological mapping where the latter is improved and elaborated in the walkover survey. Experienced engineers and geologists can often make a reasonable assessment of important aspects such as the likely depth to an adequate foundation. Rock exposures will give the rock type and allow joint measurements and thus give valuable information on likely foundation characteristics.
4. Refraction seismic surveys can be used to give improved estimates of the depth to a suitable foundation, large areas can be covered but verification by drillholes will ultimately need to be provided. Other geophysical methods may be considered. Such methods may also locate weakness zones (often associated with an increased depth of weathering). Foundation level contours drawn from the seismic survey results can be used for a first stage of optimising the dam location.
5. Test pits in alluvial and other soft deposits to reveal the material characteristics at shallow depth.
6. Drillholes in the abutments to confirm the depths to suitable foundation strata and the characteristics of the foundation.
7. Field and laboratory testing including rock strengths tests, Lugeon tests, grain size distributions of loose materials, characterisation of fine-grained soils, etc.

The definition of a suitable foundation stratum is dependent on dam type and, in many cases, also on the location within the dam. For example:

- Arch dams require strong and stable rock for the whole of their foundation
- Gravity dams are less demanding of their foundation than arch dams (less demanding with low dam sections, may also be founded on gravels)
- CFRD, AFRD and ACRD will require a plinth founded on sound rock whereas the rockfill may be placed on a less strong and less rigid foundation. For small dams the required rock quality is less. Plinth foundations on alluvial or residual soils may be feasible but require additional design considerations.
- ECRD will require the foundation of its core to be founded typically on moderately weathered rock (ISRM weathering grade III or better)

The surveys should provide pertinent data for all reasonably feasible dam types, noting that weak rocks typically require more investigation than competent rock sites. The surveys should be a foundation for dam type selection and not a result of pre-selection.

The characteristics of the future reservoir including watertightness (where evidence of karst is important) will have been made at the time of dam site selection. These issues should be reviewed at the time of dam type selection.

The extent of grouting for dams founded on rock will normally be similar for all dam types and is unlikely to affect dam type selection. Where a dam type is to be founded on alluvium, the cost of cut-offs and other ground treatment may be an important factor in the selection process and grouting costs for all dam types under consideration should be considered.

The foundation requirements for each dam type are discussed in Chapter 3.

2.4.3 GEOLOGICAL HAZARDS

Various geological hazards, which includes adverse conditions, may have an impact on dam type selection. Reference should be made to ICOLD Bulletin 129 which describes geologic hazards as “difficult’ or troublesome geologic conditions in dam foundations including: low strength ground and seams, master joints, adversely dipping rock beds, highly fractured and sheared ground, heterogeneous and highly permeable alluvial deposits, karst, soluble evaporates such as gypsum, fault zones, unusual in situ stresses (i.e. high or low compared with geostatic), volcanic sequences and lahars (volcanic mudflows), high groundwater tables during construction and unusually low natural groundwater tables to be changed by the impoundment.

2.4.4 CONSTRUCTION EXCAVATIONS

The foundation excavation required for a dam is intrinsically linked with the selection of the dam type. The requirements for each dam type are given in Chapter 3. To allow a meaningful comparison between different types of dam, excavation plans have to be developed in the early stages of a project. The volume of the excavation as well as the volume of the dam will depend on this plan. Moreover, the excavation plan has to take into account the layout and scheme for river diversion when this is located within the dam footprint.

The excavation plans also have to allow for any stabilisation measures for the abutments and slopes above the dam, both temporarily and permanent.

2.5 CONSTRUCTION MATERIALS AVAILABILITY

All dams require large amounts of quarried rock or alluvial material to make up the bulk of their mass. These materials should be obtained as far as possible from sources close to the dam as haulage costs can be a substantial cost element. The area to be examined may be extensive if adequate material cannot be found in proximity to the dam. There may be a quality aspect to this. For example, it may be cheaper to haul aggregate from a distant good source than to use a local poor source with the additional processing that might be required. It is thus important that the cost of materials including quarrying, processing and haulage be addressed early in the survey of material sources.

Bulletin 165, The Selection of Materials for Concrete in Dams should be consulted. This bulletin gives guidance on the suitability of various rock types as concrete aggregate, testing and the time required for this. All potential sources of aggregate should be tested, as early as possible, for alkali-silica reaction, or alkali-carbonate reaction, or other potential deleterious phenomena that could develop for the identified rock type.

An inventory of available sources of material should be made at the time of dam type selection. This will include sources of alluvial materials, both sand and gravel, weathered rock, residual soils and hard rock. The quantities available for each source and their engineering properties may have an important effect on the choice of dam type.

Initial assessments will be made from available geological maps, walkover surveys and setting preliminary properties using typical values. The most promising sources of material in the vicinity of the dam site would be investigated at the same time as the dam site itself using similar techniques. This may include detailed geological mapping, sampling and testing of specimens taken from the surface, refraction seismic surveys (and possibly other geophysical methods), test pits and drill-holes.

The work might be phased and become more focused with time. Ultimately, the investigations of the local construction materials will be directed at sources required for the chosen dam type.

Cement and commonly also pozzolan, is required for all dam projects. This will be for the whole dam or mostly for hydraulic structures according to the dam type. Sources of these materials have to be found for the dam type selection study. A check has to be made that there is adequate manufacturing capacity to supply the project and that the material properties are adequate. Haulage costs can be a substantial proportion of the cost of the material delivered to site. Consideration should be given to a potential need for upgrading access roads to accommodate the traffic generated by delivery of large volumes of cementitious materials.

2.6 OPERATIONAL REQUIREMENTS

2.6.1 INTRODUCTION

Dams are constructed for a number of purposes which typically include power generation, irrigation, water supply, flood control or a combination of these. The operational requirements for dams vary significantly for each project site and should be considered as part of dam type selection. Below are set out the various requirements that should be considered for each dam type under consideration.

2.6.2 FREEBOARDS

Freeboards vary with dam type and spillway characteristics. Early in the dam type selection process the minimum freeboards for each dam type should be evaluated and set. The freeboards are set above the maximum flood surcharge level as defined in Section 2.6.3 and include consideration of wave run-up and seismic seiches. The major difference between dam types is between those that can be safely overtopped (concrete dams) and those that cannot (embankment dams).

2.6.3 FLOOD SURCHARGE

The flood surcharge plus the required freeboard gives the required crest elevation above the normal water level (c.f. Section 2.6.4). (In addition, camber will be required for embankment dams). The flood surcharge is estimated from the incoming flood hydrograph, the reservoir area – elevation relationship and the spillway discharge characteristics. The required crest elevation affects the dam volume and is dependent on the spillway design. Therefore, the spillway has to be optimised for each dam type to give a reasonably balanced design for each. This can have a marked influence on dam type selection.

2.6.4 RESERVOIR OPERATION RANGE

The operation range varies for each project site and generally consists of three impounding levels:

- *Minimum Operating Level* [1]*:* the lowest level to which the reservoir is drawn down under normal operating conditions
- *Normal Water Level* [1]*:* the maximum water level at the dam to which water may rise under normal operating conditions, exclusive of any provision for flood surcharge
- *Maximum Water Level* [1]*:* the maximum water level, including flood surcharge, which the dam has been designed to withstand (Figure 2-1).

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[1] Definitions and Figure 2-1 from ICOLD Bulletin 31

Some reservoirs are operated with small reservoir fluctuations (e.g. for some run-of-river hydropower generation schemes) while others are designed for large reservoir level fluctuations (e.g. flood mitigation, irrigation, some hydropower schemes designed for peaking generation). Reservoirs designed to operate frequently with large fluctuations in reservoir level will require dam designs appropriate to withstand rapid drawdown on the upstream face; this is an issue only for embankment dams. This may be more easily achieved with some dam types than with others. Such reservoir fluctuations can also have an effect on reservoir rim stability, but this is the same for all dam types and should be addressed only in the context of the overall feasibility of the scheme.

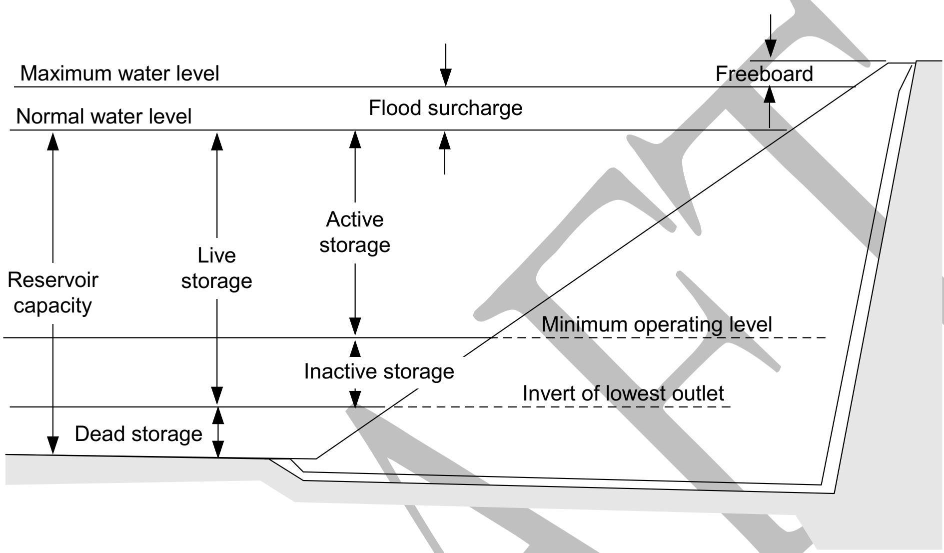

Figure 2-1: Elements of a reservoir (after Bulletin 31)

2.6.5 BOTTOM (LOW LEVEL) OUTLETS

Operational or jurisdictional requirements may require a bottom outlet [2] for emergency dewatering, sediment flushing, environmental flows or other purposes. Bottom outlets may either utilize construction diversion facilities such as diversion culverts, diversion tunnels and gated diversion sluices, or be purpose-built outlets incorporated into concrete gravity dams, tunnels and, rarely, hydropower penstocks. For very high dams and large reservoir volumes there are practical limits to the size of bottom outlets. This may be due to gate size and head limitations or the difficulty and cost of providing suitable energy dissipation. Locating bottom outlets within the body of an embankment dam is discouraged for reasons of dam safety. The physical accommodation of the outlet structure within an embankment dam body creates stress concentrations and potential leakage and internal erosion paths (c.f. Bulletin 164).

2.6.6 ENVIRONMENTAL FLOW REQUIREMENTS

Environmental flow requirements at a project site are discussed in ICOLD Bulletins 159 and 96. Environmental flow releases may require constraints on the volume, rate, temperature and timing of the release (ICOLD Bulletin 159). This may require an outlet able to draw water from different levels within the reservoir. It may be more economical to provide such an outlet in a concrete dam

[2] Bottom outlets are defined in ICOLD Bulletin 31 as "an opening at a low level in the reservoir generally used for emptying or for scouring sediment and sometimes, in addition, for irrigation releases".

than for an embankment dam which may require a separate structure outside the dam body. The design of any fish passage structures will be affected by the choice of dam type.

2.6.7 RESERVOIR SEDIMENTATION

Management of reservoir sedimentation may be an operational requirement for a project site to maintain reservoir storage, for environmental purposes or to minimize damage to turbines and loss of hydropower production. ICOLD Bulletins 147 and 115 provide detailed discussion on reservoir sedimentation management. Concrete dam types which allow bottom outlets with sufficient capacity at the original riverbed level are typically required for flushing[3] or sluicing[4] of sediments. There are other techniques to get rid of sediments such as diversion, dredging, depositing in settling basins. It is a fundamental design requirement at early project stages to verify which type of sediment management is the most efficient one, because this may substantially influence dam type selection.

2.6.8 STAGED FILLING

If staged filling of the reservoir is required for commissioning, there will be a need to plan outlet requirements (e.g. outlet tunnels, bottom outlets) of sufficient capacity to maintain the reservoir at a particular level. Outlets may be located in the body of concrete dams but embankment dams which typically require outlets located outside the dam body.

Although staged filling requirements rarely influence concrete dam type selection, they can influence the choice of embankment dam type as they vary with respect to ease of construction and risk of damage due to overtopping with a partially impounded reservoir.

2.7 FLOOD HAZARDS

2.7.1 INTRODUCTION

All impounding dams require spillways to convey floodwater safely past the dam. The river has to be conveyed past the site to allow construction of the dam. Both these require knowledge of the hydrology of the river and the physical characteristics of the future reservoir.

2.7.2 REPORT ON HYDROLOGY

A report on hydrology has to be made available as early in the dam type selection process as possible. The report should contain:

1. Statistical analysis of floods giving the flood peaks and hydrographs for flood with return periods of 2 year to 1,000 years and maybe 10,000 years.
2. Estimates of the PMF which may be made deterministically. Noting that the PMF may be the result of precipitation, GLOFs or natural dam breaches, flood studies in addition to those based on meteorology and river flows may be required in certain environments.
3. Statistical analysis of flows for each month of the year are required for design of diversion flow capacity and river closure arrangements including timing. The analyses should give the flood risks for each month with emphasis on recurrence intervals commensurate with the

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[3] Paraphrased from ICOLD Bulletin 115: Flushing is an operational technique where flow velocities in a reservoir are increased so that deposited sediments are remobilised and transported through bottom outlets.

[4] Paraphrased from ICOLD Bulletin 115: Sluicing is an operational technique where sediment-laden flows are released through a dam before the sediment particles can settle. This is accomplished in most cases by operating the reservoir at a lower water level during the flood season.

duration of the diversion, typically 5 to 50 years. As a rule-of-thumb, the maximum recurrence interval of interest is about five times the expected duration of the diversion.
4. Statistical analysis of river flows to derive water yields, i.e. flow duration curves. With the planned extractions, these are used to model the reservoir storage volumes and elevations over time.
5. The volume of the reservoir as a function of water elevation has to be calculated from topographic maps.

Items 1, 2 and 5 will be used in designing the spillway. As stated in Section 2.6.3, the floods are routed through the reservoir and the flood surcharge estimated.

Item 3 is used for diversion design including river closure. The nature of the diversion may depend on dam type, topography, geology and hydrology. Chapter 3 provides guidance on diversion design.

2.7.3 SELECTION OF SPILLWAY TYPE

Chapter 5 gives guidance on selection of spillway type for the different dam types. ICOLD Bulletin 58, *Spillways for dams* provides guidance on spillway type and capacity selection. A suitable spillway type will depend on the dam type and topography.

2.8 EARTHQUAKE AND OTHER EXTERNAL GEOLOGIC HAZARDS

2.8.1 INTRODUCTION

A number of ICOLD bulletins contain guidance on design of dams for seismic regions:

- Bulletin 112. Neotectonics and Dams
- Bulletin 123. Design Features of Dams to resist Earthquakes
- Bulletin 123. Seismic Design and Evaluation of Structures Appurtenant to Dams
- Bulletin 137. Reservoirs and Seismicity – State of Knowledge
- Bulletin 148 (2010 revision of Bulletin 72). Selecting Seismic Parameters for Large Dams

Damage to dams and their appurtenant facilities may result from ground motion induced at the site by an earthquake remote from the dam (Bulletins 123 and 148) or, less commonly, from either direct fault movement within the dam foundation (Bulletin 112) or reservoir triggered seismicity (Bulletin 137).

2.8.2 EARTHQUAKE RISK ESTIMATES

For the purposes of dam type selection, a preliminary assessment of seismic risk is required. The design ground motions may be based on code requirements, values derived for projects in a similar seismic zone or from a site-specific assessment. Whichever approach is used, design values should be available early.

2.8.3 DESIGN METHODS FOR THE DAMS

The design of the dams should be based on pseudo-static or pseudo-dynamic methods. More sophisticated methods are normally beyond the scope of a dam type selection study and are not required. Dams perform well in earthquakes, even dams that have been designed to lesser criteria than those used in current practice. Some damage has occurred during severe shaking, but nothing that endangered a dam. Dams founded on, or constructed from, liquefiable materials such as fine sand and silt may be seriously damaged or fail during an earthquake.

2.8.4 ACTIVE FAULTS IN THE FOUNDATION

A dam site will have been selected on the basis of various criteria where consideration of potential fault activity would have been an element. Sites with recent fault movement would mostly not be considered as suitable. Such a site would most probably have been abandoned in favour of a tectonically more stable one.

A conservatively designed embankment dam or concrete gravity dam should be able to survive fault break effects. Such an event would be classified as extreme and substantial damage to the dam would be acceptable but without catastrophic release of water. Thus, assuming the dam site selection process has been sound, no particular measures are required for dams in tectonically active areas except for avoidance of arch dams where a fault break in the foundation could occur.

2.8.5 RESERVOIR TRIGGERED SEISMICITY

The state of knowledge of Reservoir Triggered Seismicity (RTS) is the subject of ICOLD Bulletin 137; *Reservoirs and Seismicity – State of Knowledge*. RTS is not a significant factor in the choice of dam type and need not be addressed at the early stages of project development.

2.8.6 LANDSLIDES AND RESERVOIRS

While the problems associated with landslides and reservoirs do not directly influence the choice of dam type, their presence can influence freeboard provisions. If a potential for impulse waves due to a rockfall is identified, additional freeboard to contain them might have to be provided for embankment dams and less likely also for concrete dams.

2.8.7 RESERVOIR LEAKAGE

Leakage can occur through the rims and floors of reservoirs but is the same for all dam types and does not affect the choice.

2.9 ENVIRONMENTAL AND SOCIO-ECONOMIC IMPACTS

Dams with their reservoirs always have significant impacts by virtue of their size and flooding of usually arable land in valley floors and cross river access. Many of these impacts are permanent changes to the environment where the reservoir is the major component. Beyond the reservoir, there are permanent changes to the river flows and flood regime downstream of the dam. None of this is significantly affected by dam type.

Various construction activities can have a temporary or permanent impact. These include the presence and effect of access roads, quarries and borrow pits, works areas, spoil tips and more. Where these are located within the future reservoir, there is no significant long-term impact. Embankment dams and concrete dams may require different sources of local materials which can have different impacts on the environment and the people in the vicinity. It is therefore necessary assess the impacts for the sources of natural materials identified for each of the dam types under consideration. This assessment may be an important factor in the selection of dam type. Other aspects remain approximately the same for all dam types and need not be considered in the selection process.

3 CONCRETE DAMS

3.1 INTRODUCTION

Concrete dams can be classed into four types:

- Gravity dams that resist the water load from the reservoir by dead weight alone. They have a trapezoidal section and commonly a vertical or steep upstream face. This dam type is suited to most sites where rock is present at the surface or at moderate depth. Gravity dams less than 30 m high may also be constructed on gravelly alluvium.
- Arch-gravity dams where the water load is transferred partly by the deadweight of the dam and partly by arch action to the abutments for some or all load cases. They have also a trapezoidal section, similar to but slimmer than gravity dams. When designed to carry load in arch action for all load cases, this dam type is better described as a single curvature thick arch dam. A relatively narrow valley is required with a dam width to height ratio not exceeding about 5.
- Arch dams which may have double curvature, curvature in both vertical and horizontal directions, or single curvature (barrel vault) dams. Arch dams require a valley shape which will allow an arch to be constructed with a width to height ratio of less than about 5, but ideally 3 or less.
- Cemented Materials Dams (CMD) (or Faced Symmetrical Hard-fill Dams) are constructed from low strength concrete and commonly have a symmetrical profile. Conceptually, they are between gravity and embankments dams. These dams exert lower stresses on the foundation than other concrete dam types and are suited to sites with weak foundation strata where gravity dams would be unsuitable.

Table 3-1 illustrates these options.

Gravity and arch dams in the region of 300 m high have been constructed and operated successfully (Jinping-I, China, a 305 m high arch dam; Grande Dixence, Switzerland, a 285 m high gravity dam) but this is not a height limit.

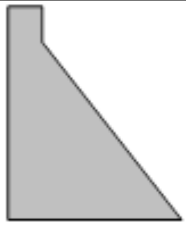
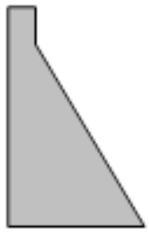
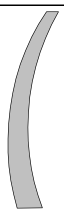
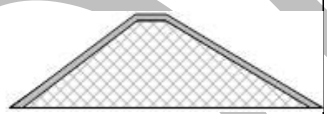
Most gravity dams are now constructed with RCC. Arch dams have until recently been made exclusively with conventionally vibrated concrete (CVC), but RCC has been used in some dams with success and this trend is likely to continue.

Gravity and arch dams are constructed in masonry in some countries where labour costs are low or where this labour-intensive construction method is desired for employment reasons. Although the construction methods are different from the mechanised methods more commonly adopted, the safety requirements and methods of design are the same.

Section 3.7 describes construction methods.

Concrete dams have an excellent record of behaviour under earthquake loads and are a good candidate for a new dam subject to all other factors being satisfied. For designs for dam type selection, seismic aspects do not have to be considered except sites with known or likely recent fault breaks. For a site with an active fault within the footprint of a proposed dam and the potential for movement, an arch dam should not be considered.

Table 3-1: Concrete dam types

Dam type	Load transfer	Foundation requirements
 Gravity	Water loads are resisted by deadweight alone; stresses transferred to the foundation are moderate	Any rock foundation, dams up to around 30 m high can be on gravelly alluvium provided cut-offs and drainage are included in the foundation to control seepage and uplift forces.
 Arch-gravity	Water loads are resisted partly by gravity, but with some loads transferred to the abutments by arch action. Stresses applied to the abutments are moderate.	Foundation requirements between those for gravity and double-arch dams.
 Double curvature arch	Water loads are transferred to the abutments by arch action. Stresses applied to the abutments can be large.	High foundation strength requirements. Requires sound and competent rock.
 CMD (hardfill)	Water loads are resisted by dead weight alone, but with low compressive and shear stress in the foundation.	Suited to weak foundations that cannot support a gravity dam.

3.2 GRAVITY DAMS

3.2.1 INTRODUCTION

As described in the introduction, gravity dams resist the water load from the reservoir by dead weight alone. They have a trapezoidal section and commonly a vertical or steep upstream face. Such dams are constructed from concrete which today is normally placed as RCC (c.f. Section 3.7.3). Small dams will be constructed from CVC where the dam volume is too small to make RCC economical. Such dams are mostly founded on rock while dams less than 30 m high may be constructed on gravelly alluvium. Figure 3-1 shows a typical large gravity dam.

Figure 3-1 Gravity dam of RCC (Rialb, Spain: Height 99 m, Volume 1.02 Mm³)

3.2.2 DAM CROSS-SECTION

The dam section is defined by the upstream face slope, the downstream face slope and their intersection (resistance apex), Figure 3-2. As a guide we have:

- Where the foundation is sufficiently strong so as not to affect dam stability, the sum of the upstream and downstream slopes may be set at 0.72:1 (h:v) for low seismicity regions, 0.80:1 for medium seismicity regions and 0.85:1 for highly seismic regions. Here medium seismicity is defined as regions with an MCE of 0.2 g and high seismicity at of the order of 0.4g
- For preliminary designs, the upstream face is normally set vertical. In zones with medium to high seismicity, the upstream face may be inclined, Figure 3-3.
- The top width of the crest may be in the range 7 to 10 m with 8 m being common. Varying the top width has no significant effect on the volume of concrete in the dam. For the purposes of dam type selection, a crest width of 8 m may be adopted.
- In seismic regions the upstream face may be inclined and a fillet between the vertical face of the top block and the downstream slope should be used to reduce tensile stresses, Figure 3-3.
- The foundation contact will normally be rough as it reflects the rock excavation process.
- Internal drainage is required to control uplift force, see section 3.2.8.

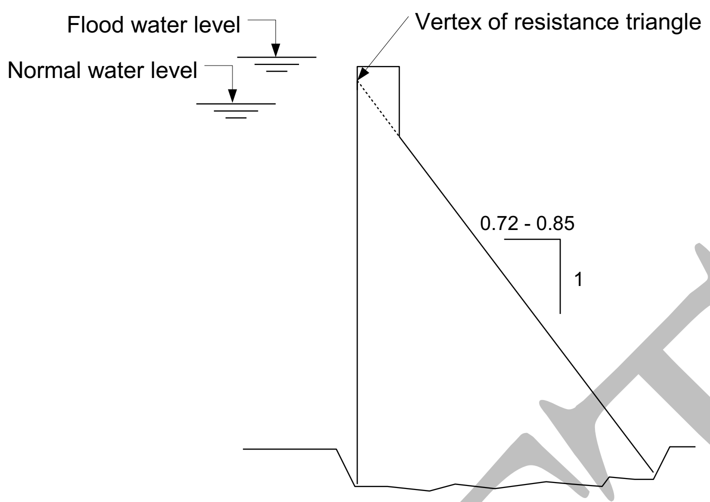


Figure 3-2 Schematic cross-section of a gravity dam

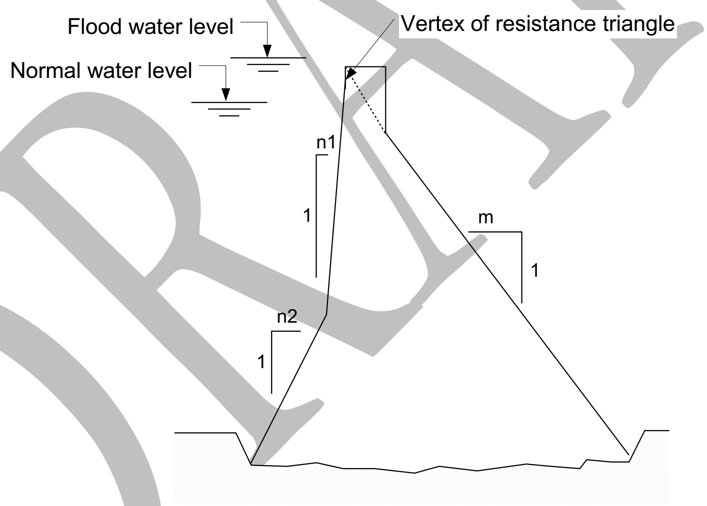


Figure 3-3 Schematic cross-section for seismic loads

3.2.3 STABILITY AND STRESS ANALYSIS

A calculation of the stability of the dam may be required to demonstrate that the acceptance criteria (Section 3.2.5) are satisfied. Adoption of a dam cross-section that complies with the listed recommendations in Section 0 will lead to the resultant of force falling within the prescribed distances from the dam centre. The sliding resistance has to be checked for the applied loads along the body of the dam (concrete lifts) and along the dam-foundation interface, considering the foundation strength recommendations presented in Section 3.2.4. The applied loads will be the mass of the dam (including the weight of piers, gates and other mechanical equipment, if present), the reservoir load and uplift forces due to pore pressures on the base of the dam (Section 3.2.8), tailwater forces, hydraulic forces from flood over the ogee and nape of spillway sections, earthquake loading, and external forces such as post-tensioning, if present. The freeware CADAM can be used to analyse gravity dams with simple cross-sections which also permits pseudo-static and pseudo-dynamic analysis of earthquake stresses and stability.

3.2.4 FOUNDATION STIFFNESS AND STRENGTH

The relevant mechanical features of the rock mass are the shear strength, bearing capacity and the stiffness. All three properties may be estimated using the Hoek and Brown method (Hoek 2007). If rock joints are expected to dominate shear strength, the Barton-Bandis method may be used (Barton and Bandis 1990).

The dam foundation must have adequate stiffness to limit differential settlements and resulting stresses in the dam. The stiffness is expressed as the deformation modulus which relates strain resulting from an applied stress. Foundation deformation modulus is primarily governed by structure (the degree of jointing of the rock) and the condition of the joint surfaces. The joint surface condition is related primarily to degree of weathering. A common foundation criterion is for ISRM weathering grade II to III, slightly to moderately weathered. This will confer adequate modulus for all dams where the rock structure can be characterised as Very Blocky or better (see RocLab). The underlying assumption is that the surface exposure reflects the stiffness of the rock at depth. Where this does not apply, the stiffness of underlying rock masses should be considered.

The required foundation modulus can range from 0.3 to 6 GPa depending on the dam height. A modulus of ¼ of the concrete modulus, about 4 to 6 GPa, is required for very high dams. Dams of medium height (80 to 100 m) have been constructed on weak rock with modulus around 2 GPa. With dam heights of 30 m or less, the required foundation modulus is less than this, maybe 0.3 to 1 GPa.

Reference is made above to a foundation being sufficiently strong so as not to affect dam stability. The strength can be evaluated using RocLab where the UCS, geological strength index GSI and intact rock parameter m_i all have a marked effect on the estimate. The adequacy of the foundation so evaluated has to be checked by applying the acceptance criteria set out below. Where the sliding resistance is shown to be inadequate for the adopted cross-section, the foundation shear stresses should be reduced and the frictional strength increased by making the cross-section wider. A further measure is to slope the upstream face of the dam so as to apply vertical water load to the foundation. In addition or alternatively, the foundation should be lowered to reach stronger rock.

Soil foundations have significantly lower deformation moduli than rock. Sandy gavel foundations might have a modulus of the order of 50 MPa and glacial till might have moduli in the range 30 to 100 MPa depending on till density. Dams up to 30 m high have been constructed on such foundations.

The strength of soil foundations should be assessed using a soil mechanics approach with visual assessments and testing as the basis.

Note should be taken of features that might adversely affect dam stability, such as sub-horizontal clay seams, shear planes, or other low-strength and continuous features with unfavourable orientation. Weak horizons such as clay seams should be characterised in soil mechanics terms.

The depth of excavation required to achieve an adequate foundation for the dam has to be estimated, initially from the topography and rock exposures at the site, and later from refraction seismic surveys and drillholes. Examination of eroded gullies and cuttings can provide valuable information about depths of weathering.

3.2.5 ACCEPTANCE CRITERIA

The stability of the dam should conform to established acceptance criteria. Typical criteria for factors of safety are shown in Table 3-2. Note that the factors for sliding stability apply to the dam-foundation interface or the foundation material itself and reflect the typically poor accuracy in determining foundation strengths in early project phases. Lower factors may be applicable when the strength assessments are based on comprehensive field and laboratory testing.

The calculation of the maximum stresses transmitted to the foundation should be carried out to verify that the foundation can bear those stress levels.

Table 3-2 Typical acceptance criteria for gravity dam stability and stress analyses

Loads	Sliding stability	Bearing capacity	UCS	Location of resultant
Normal	3	3	3	Middle 1/3
Unusual	2	2	2	Middle 1/2
Extreme	1.5	1.5	1.3	Within foundation
MCE			1	No requirement

UCS = uniaxial compressive strength, MCE = maximum credible earthquake. *USBR, 1995*

US Army Corps of Engineers 1995 and BC Hydro 1995 are useful references for gravity dam design. CADAM is useful freeware for stability analysis of gravity dams of trapezoidal cross-section.

3.2.6 HORIZONTAL ALIGNMENT

Gravity dams may have straight crests or may be curved in plan for economy (minimising dam volume if the topography is suitable), for aesthetic reasons, to accommodate a spillway with converging sidewalls or a combination of these factors. Although curved, no arch action is taken into account in the design. The axis of the dam may also be strategically aligned to allow expansion or raising of the dam in the future, if so desired.

3.2.7 FOUNDATION TREATMENT

The whole footprint of the dam is commonly subject to consolidation grouting with the purpose of increasing the foundation stiffness and reduction of permeability. Gross blast damage will also be repaired. The depth of grout holes and their spacing will depend on the rock properties and dam height and may range in depth from 5 to 10 m with hole spacings of 2 to 7 m for rock ranging from heavily fractured to massive.

Particular note should be taken of soluble rocks and minerals in the dam foundation where in particular karstic voids can cause major problems. Such features are problematic for all dam types and should be discovered at the time of dam site selection. They may affect the technical feasibility for one or all dam types. The treatment of karst may entail cut-off blankets and walls, concrete infills and grouting. The cost of such works, to the extent that they may be different for different dam types, has to be included in the cost estimates for dam type selection.

Some rock types such as porous sandstones cannot be easily grouted. Consideration should be given to alternatives such as clay blankets, positive cut-offs or drainage curtains.

3.2.8 UPLIFT CONTROL

Hydrostatic uplift in the dam and its foundation should be controlled by drainage comprising galleries and drainage holes drilled between them and from the crest to the top gallery. The maximum vertical spacing between galleries is normally about 35 m due to limitations on the accuracy of production drilling the 75 to 100 mm dia. drainage holes. A drain efficiency of 2/3 is commonly adopted but in some countries 4/5 is used. The spacing of the holes along the galleries is conventionally adopted as 3 m but can also be set as half the distance between the upstream face of the dam and the curtain and will still achieve required drain efficiencies. The pore pressures in the foundation are controlled by extending the curtain holes in the dam body into the foundation.

3.2.9 GALLERIES

The galleries should be large enough to allow drilling of drainage holes and for foundation grouting. Gallery sizes ranging from 2.0 x 2.7 m (w x h) to 3.0 x 4.0 m can be considered with 2.4 x 3.0 m being common. The galleries can be constructed with RCC (commonly grout enriched) against formwork with the roof being made of precast concrete elements. This is typically the lowest cost option but other methods have been used. The cost of galleries is small as the cost of the concrete volume saved offsets some of the cost of facing with its formwork and the pre-cast roof, however their negative impact on RCC placing rates needs to be considered when determining their layout. Similar galleries are used with CVC dams.

For large dams (height > 30 m) and unless the abutments are very steep, it is common to provide a foundation gallery along the whole length of the dam close to the foundation contact. If only horizontal galleries are used, they should be extended into the abutments, such that a lower gallery terminates vertically below the point where the next higher gallery exits from the RCC into the abutment rock.

Galleries should be placed as far upstream as possible to give maximum benefit of reduced uplift pressures. However, the space between the gallery and the upstream face of the dam should allow efficient construction. In the case of RCC dams, this space should be a minimum of 7 m and preferably 8 m.

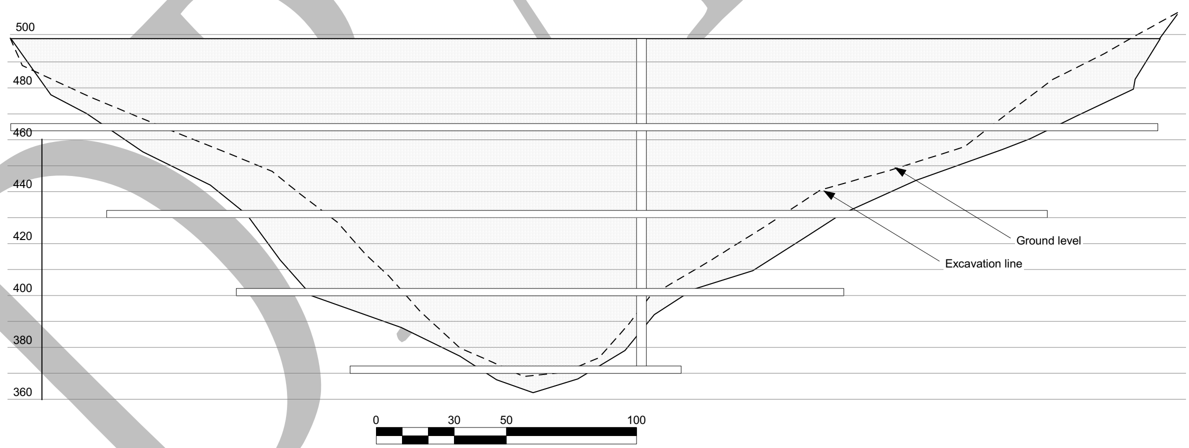


Figure 3-4 Long section of dam with horizontal galleries

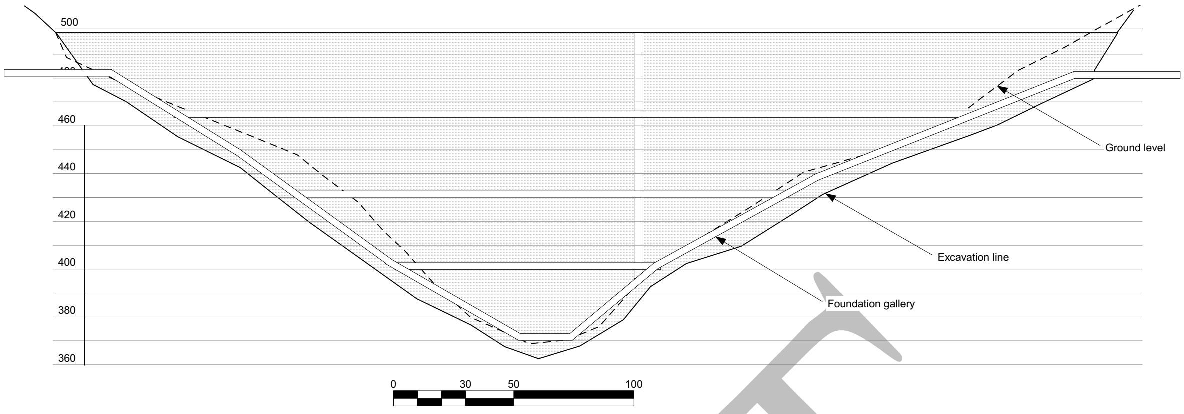


Figure 3-5 Long section of dam with foundation gallery

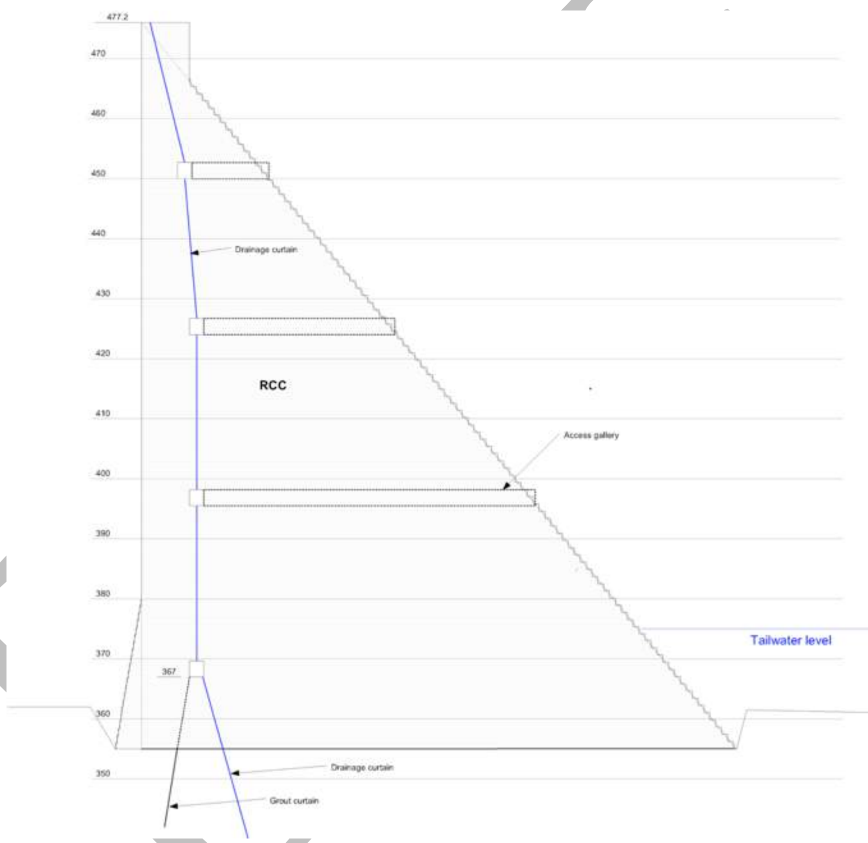


Figure 3-6 Typical gravity dam cross-section showing galleries

3.2.10 CONCRETE PROPERTIES

For initial dimensioning, the concrete used for a gravity dam should meet the following requirements:

1. A density of more than 2.35 t/m^3. A lower density would cause the dam to have flatter external slopes than those indicated above.
3. A compressive strength of more than 10 MPa at 90 to 365 days maturity. In general, tensional aspects in the dam body are not critical in the feasibility study of a gravity dam.

4. Low permeability concrete (less than 10^{-8} cm/s). This need not be the subject of a separate consideration as any well-designed medium or high cementitious content concrete mix will yield this permeability or better.

For the purposes of this bulletin, the concrete, including RCC, may be assumed to have a cohesive strength of 1 MPa and a frictional strength of 45°. This will apply also to the lift joints of RCC.

3.2.11 SPILLWAYS

Chapter 5 gives information on spillway design. The text below gives the most important considerations in locating and configuring spillways and outlets.

There are four locations where spillways can be placed:

- On the crest, as controlled (gated) or uncontrolled overflows where the water is evacuated either via a chute and flip-bucket with energy dissipation in a plunge-pool, or via a chute with energy dissipation in a stilling basin.
- As orifice spillways in the dam body with evacuation and energy dissipation as indicated above.
- Independent of the dam body as one or more open channels along the flanks crossing the dam abutments or at a remote location
- Tunnel spillways

Factors to be considered when evaluating alternatives are:

- Flood size (peak discharge)
- Head difference between reservoir and tailwater
- Width of river at plunge-pool or stilling basin
- Shape of dam allowing free overflow or chute flow
- Potential for erosion at plunge-pools
- Possible spillway locations: on dam, on abutments, remote location, tunnels

There may be two or more technically viable options to be used separately or in combination. The cost of these should be estimated and added to the cost of the dam.

3.2.12 DIVERSION

Chapter 6 provided guidance on diversion strategies and design.

3.3 CEMENTED MATERIALS DAMS (CMD)

3.3.1 INTRODUCTION

This dam type is also known as a Faced Symmetrical Hard-fill dams (FSHFD) or simply hard-fill dams. They are typically symmetrical with slopes of 0.6:1 to 0.8:1 (h:v) with 0.7:1 being common.

The dam body comprises natural sandy gravel or excavated rock material bound with cement. It can be porous depending on the grading of the aggregates and cementitious content, in which case an upstream face membrane usually concrete is required. The materials however, are mostly not classified, with only large stone removed.

Cementitious contents are typically 60 to 100 kg/m³. Strength at 180 days is typically required to be 5 MPa. If nothing else is known, a cement content of 80 kg/m³ may be adopted for a preliminary design.

The water barrier is on the upstream face and is commonly made of concrete. As with CFRDs, most FSHDs have featured reinforced concrete slabs and with either variable or constant thicknesses typically in the range of 300 to 500 mm although one dam has been completed with a geo-membrane face seal. The steepness of the face demands fixing of the membrane.

This dam type is suitable for weak rock foundations, also gravel where the dam is less than 50 m high.

Dam heights are commonly 30 to 60 m but one dam of 107 m height has been built, see ICOLD Bulletin 135.

3.3.2 TYPICAL CROSS-SECTION

Figure 3-7 shows a typical cross-section. For preliminary design purposes a top width of 6 m can be adopted with side slopes of 0.7:1 (h:v). A drainage gallery should be incorporated for dam heights exceeding 15 m.

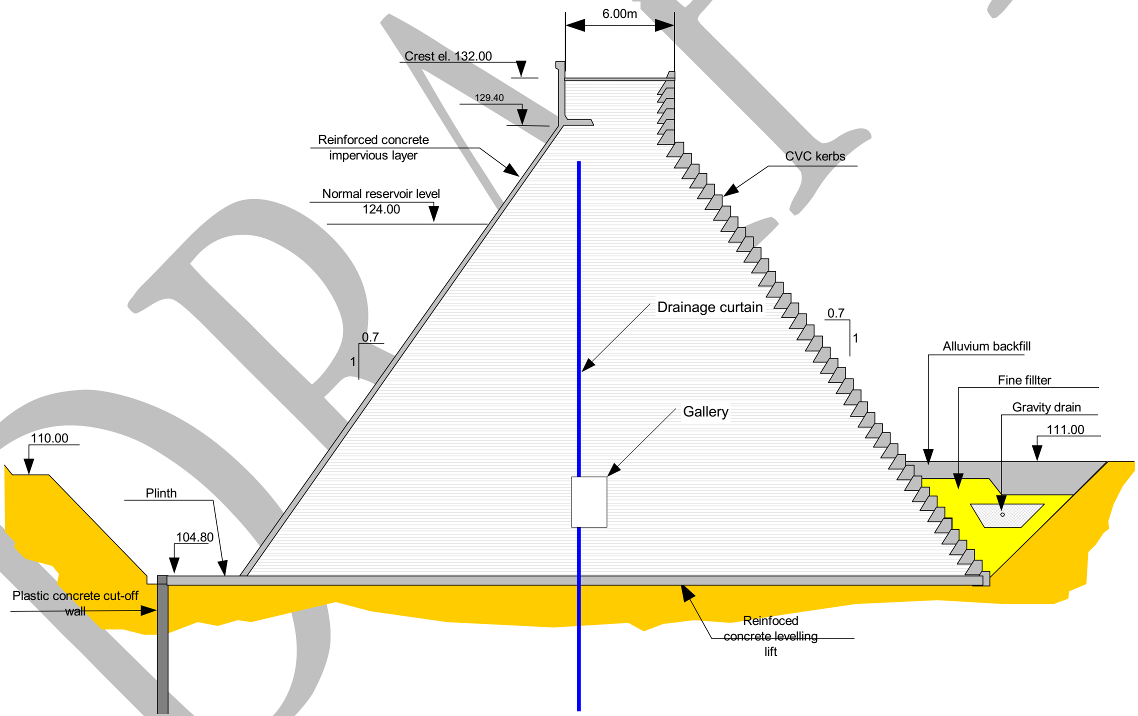


Figure 3-7 Non-overflow cross-section of Contraembalse de Monció

3.3.3 STABILITY AND STRESS ANALYSIS

Acceptance criteria the same as for gravity dams.

Vertical stresses on the foundation are of the order half those of a gravity dam, see Figure 3-8

Most FSHDs feature drainage systems beneath the facing but may be omitted if the natural porosity of the hardfill renders this unnecessary.

3.3.4 MATERIALS

In order to realise the benefits of hardfill natural, as-dug material is used as much as possible and with minimum processing.

Maximum grain sizes of 80mm are commonly used however in China this has increased to 150mm with 300mm being acceptable for cofferdams

3.3.5 OVERTOPPING

Raised upstream water levels due to over-topping may also raise internal pressures on lift joints. However successful cases of FSHDs being over-topped, for example in the case of cofferdams, indicates that this does not require modification of design assumptions about required internal shear-friction characteristics.

3.3.6 THERMAL EFFECTS

As far as thermal behaviour is concerned, the low cement content means only a small temperature rise, making contraction joints in the body of the dam generally unnecessary.

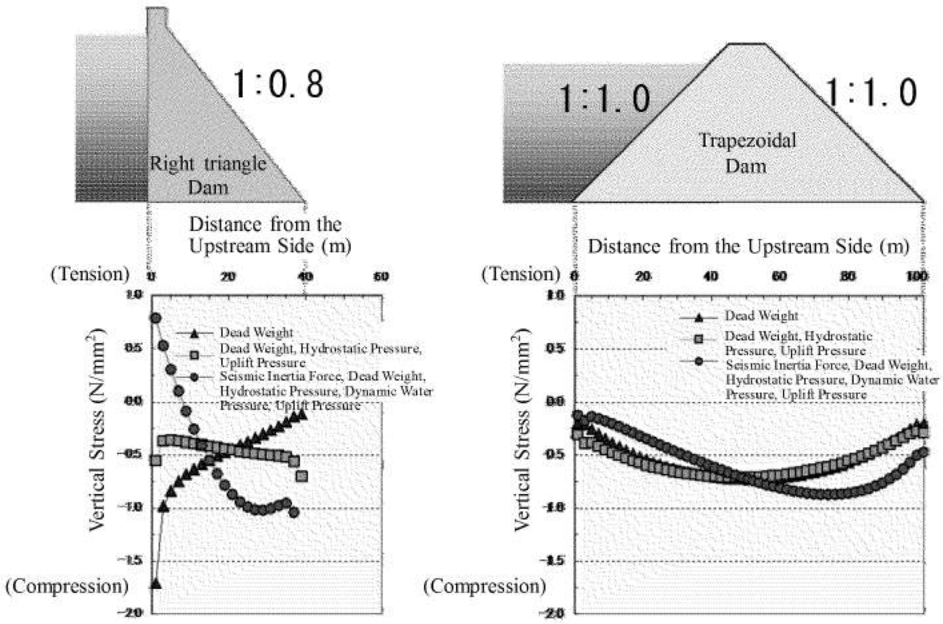


Figure 3-8 Distribution of vertical stress over bottom of dam

3.3.7 EFFECTS OF SEISMICITY

A FSHD has a large factor of safety under seismic condition. Even strong earthquakes do not generate tensile stresses of significance.

3.3.8 SPILLWAY

Spillways can be located on the dam as for gravity dam. The control structure, chute and energy dissipater have to be made of CVC.

3.4 GRAVITY DAMS AND CMD ON DRIFT

Concrete dams can be constructed on alluvium or other drift where there is no rock at a reasonable depth. They are typical of wide river valleys (barrage projects) and comprise the flood discharge (spillway) sections. Non-overflow sections would mostly be embankment dams in this context (c.f. section 4.2). Dams with an impounded depth of 20 m have been successfully implemented.

Drift foundations will have a low modulus and in the case of alluvium, high permeability and susceptibility to piping. Designs can be developed to accommodate such foundations, but the low modulus will place limits on dam height. Settlement mitigation can include increasing the base width and thus reduce the foundation pressures and lower the foundation elevation to stiffer strata.

Traditional gravity dams, possibly with a widened base, and CMD can be considered. CMD may in many cases be optimal due to lower foundation pressures and lower modulus of the dam body than concrete dams. Joints in the dam may have to be designed to accommodate differential settlement between structural blocks while maintaining watertightness.

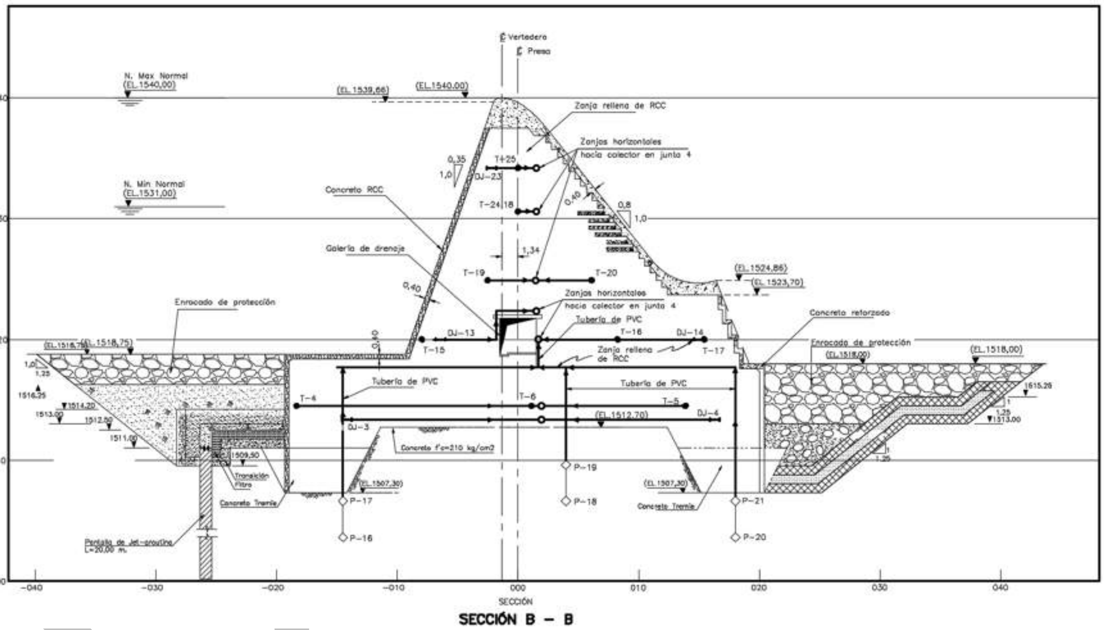


Figure 3-9 Capillucas Dam on deep alluvium (Platanal HEP in Peru)

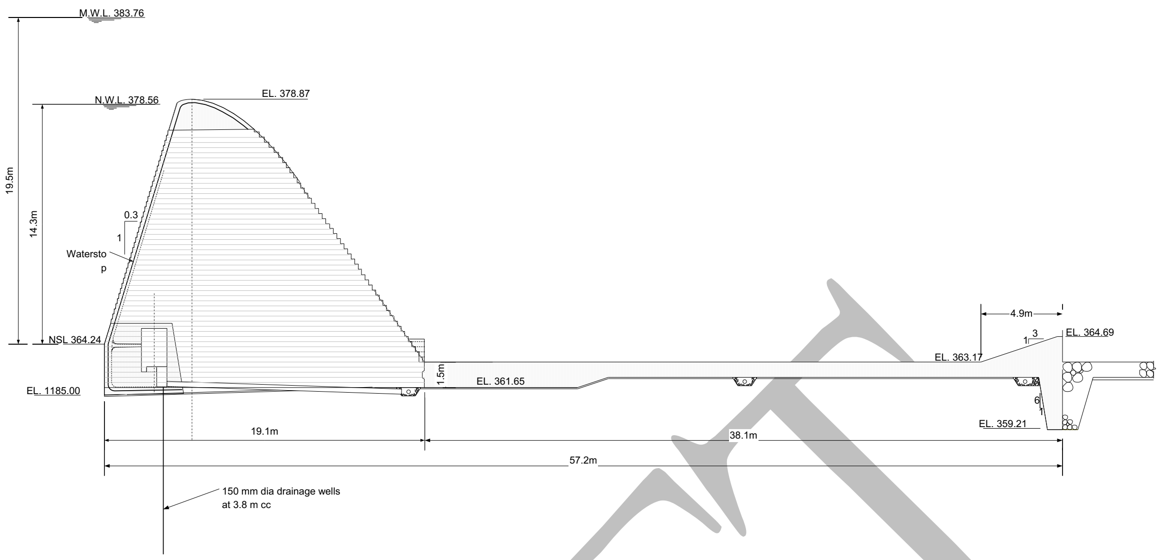


Figure 3-10: Gravity dam on dense silt

Mangla dam emergency spillway, Pakistan, shaded are is CVC, hatched is RCC

Measures to reduce seepage losses and to control hydraulic gradients will normally have to be included. This may comprise cut-off walls as for Capillucas Dam (Figure 3-9), aprons and grouting. Filters may be required against the downstream part of the foundation to prevent piping.

If this type of dam is selected for these foundation conditions, careful assessment is required of the deformation parameters and pattern. The deformation pattern may show that the construction joints between the blocks forming the dam should allow movement between them. The seals for these joints require careful detailing. Much of this work may not be required for a dam type selection study as the associated construction costs are small.

The foundation modulus should be determined with in situ investigation methods such as SPT, CPT, pressure meter, dilatometer and plate load tests (Lutenegger & DeGroot, 1995).

For uncemented deposits the foundation modulus may be in the following ranges:

	Modulus, MPa
Sand	10 to 80
Gravel	50 to 200
Till	100 to 1500

3.5 ARCH DAMS

3.5.1 INTRODUCTION

An arch dam carries the water load partly as horizontal arches spanning from abutment to abutment and to a lesser extent by cantilever action where the cantilevers are notional vertical slices of the dam anchored to the foundation. The proportion of load carried by arch action depends primarily on the shape of the valley. Arch action dominates in narrow valleys with cantilever action being more important in broad valleys. Dam cross-sections tend to be thicker in the latter case.

This section gives a method of making an outline design of an arch dam sufficient for a dam type selection study. Structural analysis is not required for this development stage. Only the principal quantities are needed: dam volume, face area, foundation excavation volume and foundation area. In later stages of project development detailed engineering analysis and design is required. USACE (1994), USBR (1977) and USBR (2013) provide guidance on the detailed design of arch dams. There are a number of finite element software packages that are suited to their structural analysis. Such analysis might give a better definition of the structural shape and enhance credibility of the design but cannot be expected make cost estimates more accurate in the context of dam type selection.

Arch dams can be divided into two groups:

- Double curvature arch dams, which are curved both in plan and in section.
- Single curvature arch dams, which are curved only in plan. This group includes arch-gravity dams which are intermediate in thickness between double curvature arch and gravity dams.

Figure 3-11 Double curvature arch dam in CVC (Atazar, Spain, 130 m high)

3.5.2 VALLEY SHAPE AND DAM CURVATURE IN PLAN

The valley shape can be characterized by its width to height ratio (L1/H in Figure 3-12). A ratio is below 3 gives very good economy and it should not exceed 5 or 6 or most of the arch action will be lost.

The axis of the dam is defined as a line curved in plan that follows the upstream edge of the dam crest.

The valley shape influences the type of curvature to be used in plan. The following are commonly used:

- Narrow V profiles are suited to circular arches and of uniform thickness with the crown section having moderate curvature.
- Broad V profile favour an axis shape with variable curvature (having a maximum curvature in the centre and a minimum one on the sides defined by three arches, parabolas, ellipses or logarithmic spirals.

- Narrow U profiles have steep valley sides in the upper part of the profile with a stream bed that might be half the width at the crest. The horizontal arches will have a relatively uniform thickness in the upper part of the dam but of variable thickness lower down. The crown cantilever will have more curvature than the crown cantilever in a narrow V site.
- Broad U profiles typically have dams with arches with uniform thickness in the upper part of the dam and variable lower down with the transition at 1/3 point down from the crest. In the higher part of the dam, a high proportion of the load is carried by cantilever action which requires a thick crown section with strong curvature.

For the purposes of dam type selection, an outline design will typically be based on a circle segment, Figure 3-13, or may be asymmetrical with each half of the dam having different curvatures, Figure 3-18. With little more effort, non-circular axes can be adopted.

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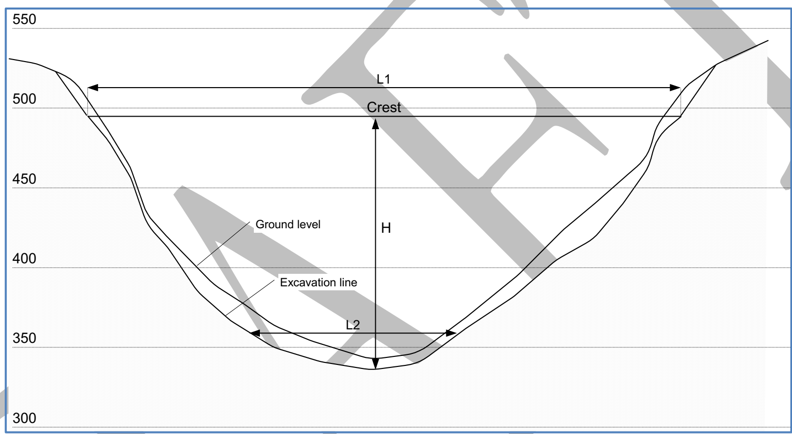


Figure 3-12 Definitions for valley shape

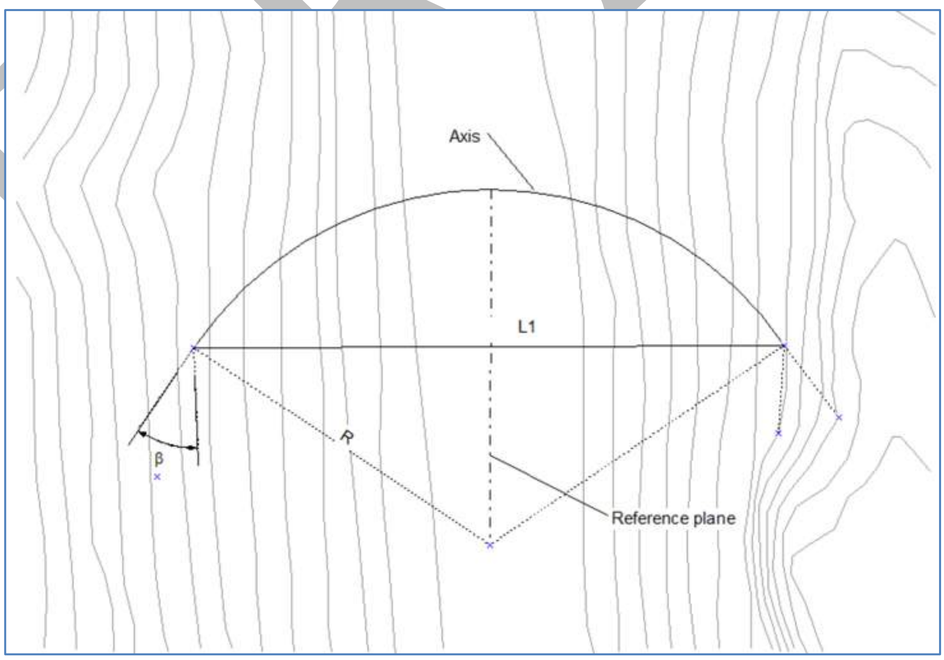


Figure 3-13 Definition of arch dam with circular axis

3.5.3 FOUNDATION REQUIREMENTS

The foundation profile across the valley should be made reasonably smooth. Abrupt changes in rock profile should be removed by excavation and concrete infill

All arch dams require a sound and competent rock foundation.

The maximum stresses applied by the dam to the footprint are typically in the range of 8 to 12 MPa at the base of the downstream face. Higher values may occur locally under specific conditions (thermal extreme, and earthquake loadings). The bearing capacity of foundation rock should generally be three times the applied stress. The Hoek & Brown method (RocLab) yields the global rockmass strength as a function of basic rock characteristics which equates to bearing capacity.

The foundation modulus (E_m) should not be less than ¼ of the concrete modulus (E_c), see Table 3-3, or about 5 GPa. The modulus can be obtained from RocLab, c.f. Section 3.2.4. Alternatively, a refraction seismic survey p-wave velocity of 3,000 to 4,500 m/s will indicate an adequate modulus.

Table 3-3 Rocha (1964) Structural Influence of E_c/E_m Ratio for Arch Dams

E_c/E_m	Influence on Dam	Problems
< 1	Negligible	None
1 – 4	Low Importance	None
4 – 8	Important	Some
8 - 16	Very Important	Serious
> 16	Special Measures	Very Dangerous

The depth of excavation required to find the required rock quality will have to be assessed. Excavation depths can be large to find strong rock and for valley side shaping. If the depth of excavation exceeds some 20% of the height of the dam, an arch dam might not be cost-effective.

A number of issues may have to be addressed to ensure technical feasibility but that do not significantly affect the cost estimate:

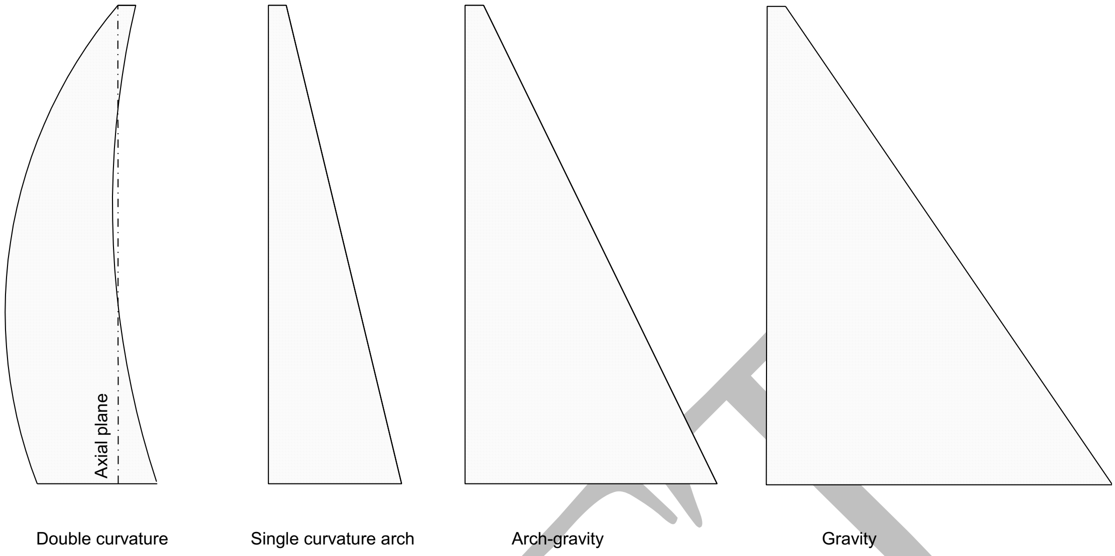
The stability of the foundation should be checked and any kinematically possible failure mechanisms identified. Where such exist, stability of the dam on its foundation should be checked using appropriate joint strengths and applied loads including pore pressures.

The width of the foundation is narrow with correspondingly high hydraulic gradients. Extensive grouting may be required to stiffen and strengthen the abutments and control water seepages and uplift pressures. Uplift pressures in the abutments may have to be controlled with drainage galleries and drainage holes. Uplift at the base of arches in wide valley may be critical.

Performing block stability analyses for rock mass in the abutments is also required. The analyses may result in additional treatment or other mitigation measures in design

3.5.4 THE DAM IN CROSS-SECTION

The dam cross-section is defined by the crown cantilever, the section at the highest point of the dam. In double curvature arch dams both faces are curved Figure 3-14a, whereas with single curvature (barrel vault) dams the faces are planar, Figure 3-14b.



a b c d

Figure 3-14 Concrete dam cross-sections

Where abutments have adequate strength and stiffness, double curvature arch dams will have the smallest concrete volume. However, smaller dam (less than 40 m high) may be more economically constructed as single curvature dams where the complications of providing a double curvature are not cost-optimal. Any savings in concrete volume achieved by the double curvature might be offset by additional formwork and other costs.

Arch-gravity dams (Figure 3-14c) may be designed to act as gravity dam under normal and possibly unusual loads with arch action being invoked for more severe load cases. Where such dams are designed to carry a proportion of normal loads in arch action, they are better considered and named as thick arch dams.

The stresses in the dam are mostly in compression remote from the abutments but at the abutments there are moments which have to be transferred to the foundation. When viewed on a horizontal section, the dam thickness typically increases towards the abutments.

3.5.5 OUTLINE DESIGN

3.5.5.1 INTRODUCTION

This section gives a method of making an outline design of an arch dam sufficient for a dam type selection study. It is aimed at providing estimates of concrete and excavation volumes, the principal cost-driving elements. Face areas and foundation areas follow as a natural result.

The method is based on USBR EM 36 and USACE EM 1110-2-2201. Only the essence of these publications is presented here.

3.5.5.2 ARCH DAM GEOMETRY

An arch dam is defined by three elements:

5. A surface curved in plan that follows the upstream edge of the crest (the dam axis), the axial plane, Figure 3-13. This plane may be one or more circle segments, an ellipse or other function depending on the shape of the valley. Initial estimates are commonly made on the basis of one or two circle segments and this is followed here.

6. The crown cantilever which is the section of the dam at its maximum height,
7. A reference plane running at right angles to dam axis and containing the crown cantilever.

To achieve effective arch action, the included angle in plan should be 100° to 120°. EM36 suggests a radius of 0.6 L1 where L1 is the distance between the abutments at crest level including foundation excavation. This gives an included angle of a little less than 120°.

The curved surface has to be fitted to the terrain such that the angle the tangents to the circle make to the foundation contact is about 40°, see Figure 3-13. The foundation contact alignment is the average rock contour. This should apply to the top 2/3 of the dam. This angle is required to ensure that there is sufficient rock downstream of the arch to carry the applied load.

A developed section of the contact of the reference plane and ground level should be drawn and the estimated foundation level shown. Foundation strength requirements and shaping requirements are discussed in Section 3.5.3.

3.5.5.3 THE CROWN CANTILEVER

The crown cantilever is defined by the crest width and the foundation width according to the following formulas (from EM36):

$$T_C = 0.01\,[H + 1.2\,L_1] \qquad \text{Eq. 1}$$

$$T_B = k\sqrt[3]{0.0012 \cdot H \cdot L_1 \cdot L_2 \left(\frac{H}{400}\right)^{\left(\frac{H}{400}\right)}} \qquad \text{Eq. 2}$$

$$T_{0.45H} = 0.95\,T_B \qquad \text{Eq. 3}$$

Here H is the maximum height of the dam and L_2 is the width between the abutments to the excavation line at a height of 0.15H above the lowest part of the dam. At the crest the dam extends the distance T_C to the downstream of the reference plane, at a height of 0.45H it extends the distance 0.95 T_B upstream of the reference plane and at the foundation a distance of $^1/_3$ T_B upstream of this plane, see Figure 3-15.

The formula for T_B is for foot (imperial) units and conversion is required for metric units. This formula contains a factor k >1 which is introduced to increase the base width of the section for cases where the volume of the dam is too small, see Section 3.5.5.5.

The upstream face is known as the extrados and the downstream face the intrados of the dam. The faces of the crown cantilever are defined by circle segments. The width of the dam and its upstream projection from the axis as a function of height must be calculated or obtained graphically.

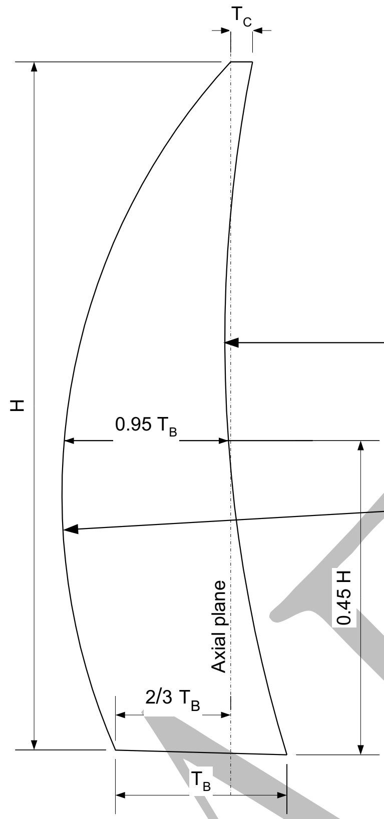


Figure 3-15 Definition of crown cantilever

3.5.5.4 DEFINING THE FACES OF THE DAM

The upstream face (extrados) of the dam can be defined by a series of contour lines as follows:

1. Divide the dam into a series of vertical (contour) intervals, often taken as 10 m.
2. Starting with the first contour below the crest, identify the plan locations at which the dam axis intercepts the foundation level. From the crown cantilever geometry, find the plan location of the extrados at the crown section. A circle segment drawn through these three points defines the first contour, see Figure 3-16a.
3. Repeat the above until the bottom of the dam is reached.
4. The downstream face (intrados) may be found as follows:
5. The radius of the intrados will have its origin at the centre of the extrados radius for a single centre and constant thickness dam, Figure 3-16a. A variable thickness dam will have shorter intrados radii, Figure 3-16b.
6. For each elevation and at the intersections with the foundation as found above, draw radial lines. The intercepts with the intrados defines a point on the downstream edge of the foundation.
7. The thickness of the crown cantilever at the same elevation defines a third point of a circular arc which gives the contour of the intrados at that elevation.
8. Repeat the above until the bottom of the dam is reached.

The upstream and downstream contour with the foundation excavation line defines an area. From the areas of successive elevations, the volume of the dam can be calculated.

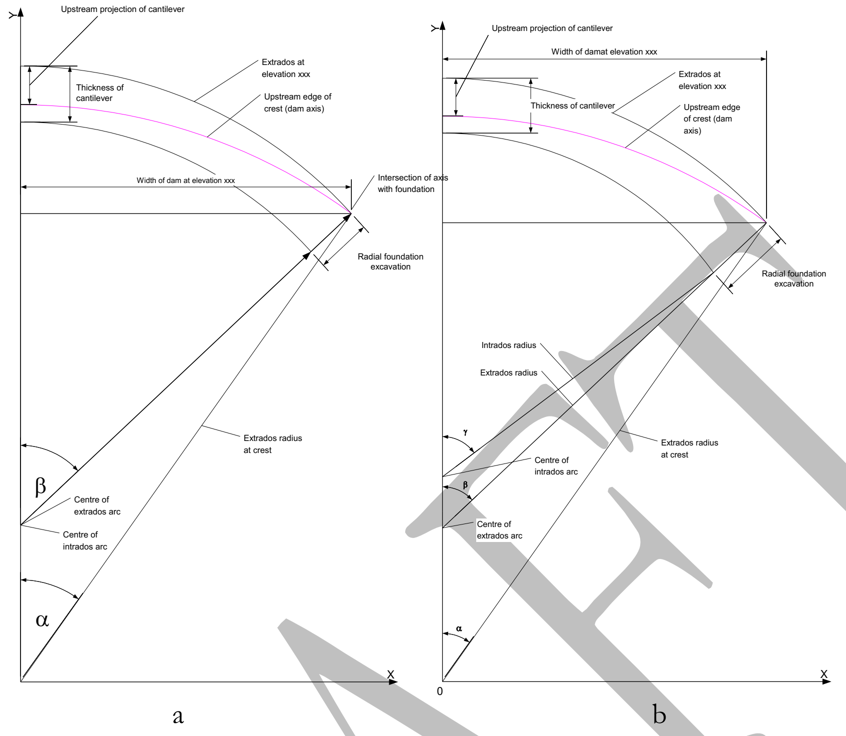


Figure 3-16 Schematic plan of dam at an elevation, a) constant thickness dam and b) variable thickness dam

As indicated above, the dam may have a uniform thickness at any elevation in which case the centres of the horizontal arcs defining the contours of the extrados and intrados will have the same centres, Figure 3-16a. Giving the radius to the intrados a smaller value will thicken the cross-section towards the abutments, Figure 3-16b. The latter is commonly adopted but for the purposes of estimating the quantities for dam type selection, a single centre dam is sufficient. Figure 3-17 shows schematically a reference plane with lines of centres for both faces of the dam.

Asymmetric sites may require a dam where each side of the reference plane is treated separately. The reference plane will be the same, but the radii will be different for the left and right sides, see Figure 3-18.

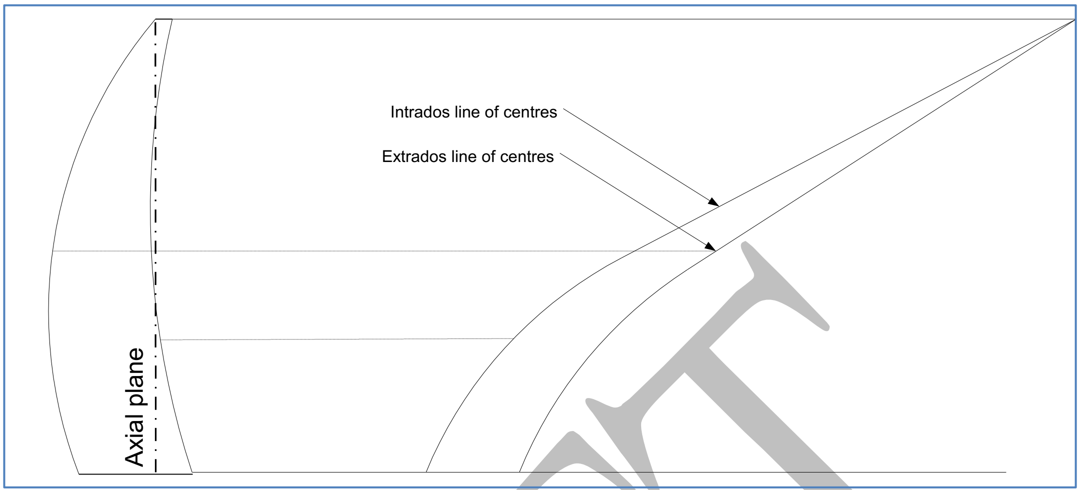


Figure 3-17 Reference plane with lines of centres for extrados and intrados for a single centre and variable thickness dam

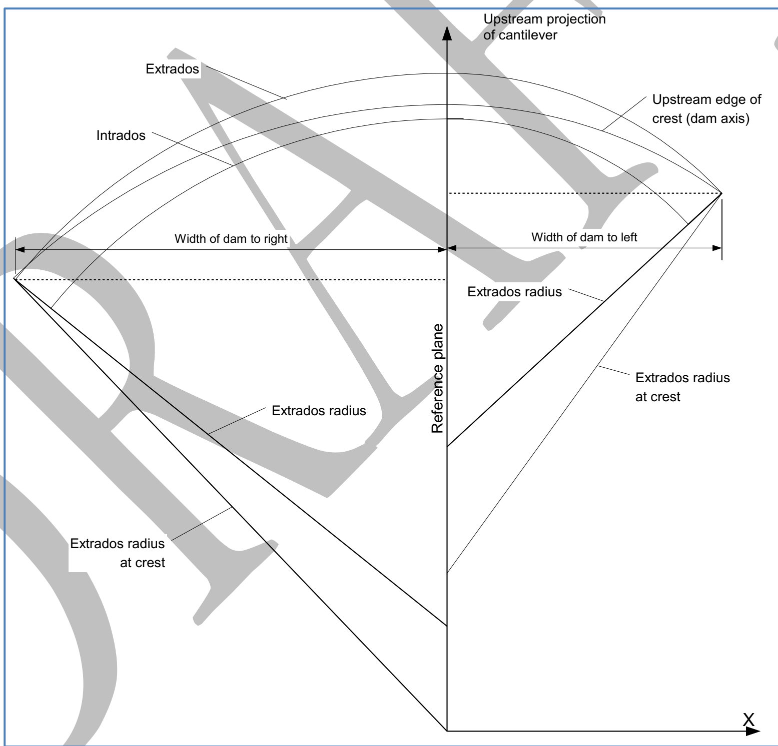


Figure 3-18 Schematic plan of dam at an elevation for an asymmetrical dam site

3.5.5.5 LOMBARDI SLENDERNESS COEFFICIENT AND SUFFICIENCY OF CONCRETE

The definition of the dam faces as shown above will yield the following:

- H, the maximum height (m)
- V, the volume from the areas of the horizontal slices and their spacing (m^3)
- F, the face area from the excavation profile and its length (m^2)

From this the Lombardi slenderness coefficient (Lombardi, 1991 and Lombardi, 2005) can be calculated:

$$C = \frac{F^2}{VH}$$

Figure 3-19 shows the basis for the above. The graph gives the slenderness coefficient and the height of arch dams across the world. A line is defined where H·C =3000 m. Dams with generally good performance plot below this line and this should be used as a design guide.

H·C < 3000 should be maintained to ensure that there is adequate concrete in the dam to give satisfactory performance. If H·C exceeds 3000, then the thickness of the crown cantilever should be increased by making the coefficient k in Eq. 2 greater than unity.

It is a relatively simple task to perform the tasks set out above using linked spreadsheets such that a change in the k-value will give a new dam volume with minimal effort.

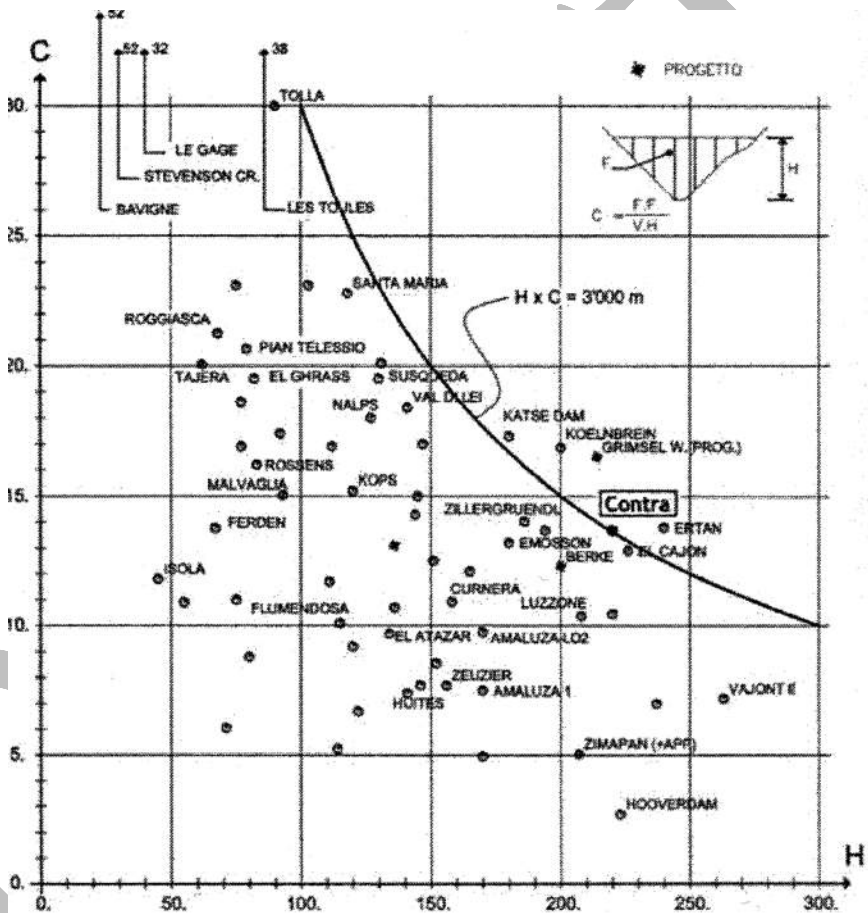


Figure 3-19 Slenderness coefficient versus height
Lombardi (2005)

3.5.6 INTERNAL DRAINAGE AND UPLIFT CONTROL

Arch dams require internal drainage to control pore pressures and seepage through the structure and its foundation. This comprises a system of galleries and drainage curtains similar those of a gravity dam (Section 3.2.9, Figure 3-5 and Figure 3-6). Galleries are not made in thin sections where the dam thickness is less than about five times the gallery width.

3.5.7 FOUNDATION TREATMENT

The whole footprint of the dam is subject to consolidation grouting with the purpose of increasing the foundation stiffness and reduction of permeability and is the same as for gravity dams (c.f. Section 3.2.7) but the treatment may be more intense given the higher stresses imposed by an arch dam.

3.5.8 COOLING CONCRETE

Concrete cooling will be required to reduce the peak temperatures due to heat of hydration and to open up joints between monoliths to allow these to be grouted. Cooling may be by pre-cooling concrete constituents prior to mixing or by post-cooling using circulation of chilled water through pipes embedded in the concrete, or both. Post-cooling pipes have traditionally been made of thin-walled steel tube for CVC construction. Post-cooling pipes for RCC have been made from HDPE which is less susceptible to damage during RCC placement and compaction. The installations required to achieve this need not be designed for the purposes of dam type selection, but an allowance for cost of so doing should be provided.

3.5.9 JOINTS AND JOINT GROUTING

3.5.9.1 CVC CONSTRUCTION

An arch dam constructed in CVC will be made as a series of contiguous monoliths which are continuous from the upstream to the downstream faces of the dam. Chapter 13 of USACE (1994) provides recommendations on the width of monoliths for CVC construction. For the purposes of this bulletin, monolith widths (joint spacings) of 15 m to 20 m may be adopted.

Waterstops and grout stops are required, the latter only if the joints in the dam are to be grouted.

Shear keys are installed in monolith joints to provide shearing resistance between monoliths.

Monolith joints may be grouted to provide a monolithic dam. Grouting is done when the temperature of the concrete has reduced to a pre-determined value, the closure temperature. The grout is injected through embedded pipes and the grout is confined to zones of a joint with grout stops. The grout stops are PVC water bars installed across the joints.

3.5.9.2 RCC CONSTRUCTION

An arch dam may be constructed as RCC with induced contraction joints (Section 3.7.3). As with CVC construction, waterstop are required for seepage control and grout stops are needed to divide the area of each joint into zones for the purposes of grouting. These waterstops and grout stops have to be formed on the induced contraction joints as RCC construction proceeds, see ICOLD Bulletin 175..

Shear keys are no commonly provided on the induced joints as the method of their construction makes these very rough.

As for CVC construction, joints may be grouted to provide a monolithic dam, c.f. Section 3.5.9.1.

3.5.10 CONCRETE PROPERTIES

The required concrete properties are similar to those of gravity dams:

1. Density greater than 2.35 t/m³.
2. A design compressive strength of 15 to 25 MPa at one-year maturity for major dams, as the concrete will be at least this old before reservoir loads are applied. It may be necessary to consider shorter maturities for temporary load conditions.

3. The risk of alkali - aggregate reaction or other potential deleterious phenomena should be analysed early, due to the time required for corresponding tests, ICOLD Bulletin 165, 2013. This risk is mitigated by the incorporation of adequate quantities of pozzolan in the concrete.
4. Low permeability concrete (less than 10^{-8} cm/s) is required, but this will follow if the above conditions are satisfied.

3.5.11 SPILLWAYS AND OUTLETS

Chapter 5 gives information on spillway design. The advice on spillways given for gravity dams (Section 3.2.11) applies also to arch dams. However, particular note should be taken of the risks associated with a plunge-pool at the toe of an arch dam which might have the potential for undermining it.

3.5.12 DIVERSION

Similar considerations used for gravity dams are used for the design of the diversion system of arch dams. Due to the narrowness of the valley, the use of tunnel diversions is often required.

A relatively low protection level against floods (10-25 years) is generally acceptable, given the low consequences of flooding the works during construction.

Chapter 6 provides further guidance on diversion strategies and design.

3.5.13 ACCESS

Access to the site and internal construction roads will differ somewhat depending on the dam type being considered but the cost difference will be small and need not be taken into account.

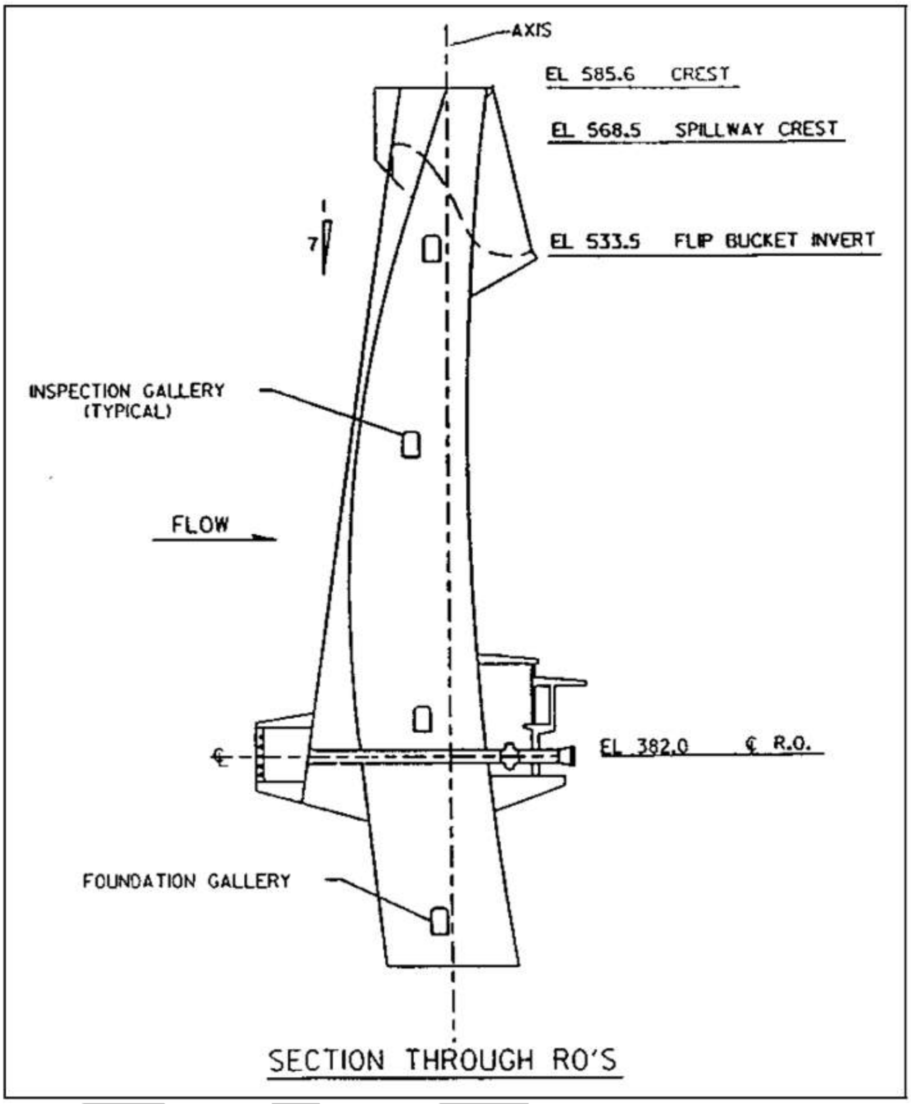


Figure 3-20 Section of an arch dam showing crest spillway (gated or un-gated) and orifice spillway
From USBR EM 1110-2-2201.

3.6 RUBBLE MASONRY CONCRETE DAMS

Masonry gravity and arch dams have in earlier times be constructed extensively. After the first quarter of the 20th century, the number of masonry dams diminished and were replaced by concrete and embankment dams. Masonry dam construction has remained common where low-cost labour is plentiful. In China and India for example, very large gravity masonry structures up to 95 m in height were constructed until the advent of Roller Compacted Concrete (RCC) in the early 1980s. In seismically active areas, masonry dams are not recommended.

Rubble Masonry Concrete (RMC) dams are constructed with stones set in mortar and faced with natural stone. The facing stones are set in a stiff mortar and the space between them is filed with stones set in a free-flowing mortar. The lift height is commonly 200 to 250 mm. The method is labour intensive with minimum use of construction machinery. This approach gives benefit to the community through employment and skill development.

Figure 3-21 Rubble Masonry Concrete Arch dam, 42 m high (Lucilia Poort, Zimbabwe)

While RMC is an appropriate material for gravity dam construction, the reduced material volumes inherent in an arch configuration imply that arch dams are likely to be the preferred solution for most RMC dams of heights greater than five to seven metres.

Where a competent foundation with minimal overburden exists, sources of suitable sand are available and river flows require a reasonable spillway size, an RMC arch dam will often be the lowest costs dam type option for heights under 20 to 25 m. While RMC arch dams have been constructed to a height of 42 m, their cost-effectiveness diminishes with height as construction at height requires cranes and more costly safety provisions. Furthermore, arch-buttress dam types lose competitiveness above 20 m height due to a rapidly increasing mass requirement for sliding stability.

The methods of design and acceptance criteria for stresses and stability are the same as for concrete dams. With hand placed materials there will be greater variabity in material properties than with concrete producd in a modern batch plant. Such varialbity has to be taken into account in analysis of stresses in the dam body.

The success of modern RMC dams lies in the design of the associated structures as arch, multiple arch-buttress and arch-gravity dam types; in each case deriving a substantial reduction in masonry volume compared to a gravity dam configuration. Impermeable sections have been constructed on arches with widths as small as 600 mm and the dams are constructed as continuous structures, without joints.

Where the site topography and geology are suitable, single-curvature arch structures have been successfully constructed, while multiple arch-buttress structures have generally been applied in valleys typically more suited to gravity dam configurations. The single limitation of the RMC dam technology lies in an inability to effectively construct as overhangs, which imposes limits on arch dam cross-sections and makes RMC arch-buttress structures unrealistic.

The jointless construction of RMC dams do not crack in temperate climates. In spite of the method of construction and the thin sections, RMC dams do not leak but seep on first filling before sealing themselves by autogenous healing and becoming watertight. In this process, efflorescence and calcite deposited in layers on the downstream face and a white streaking effect can detract from the overall visual appeal. As seepage diminishes, however, the calcite deposits weather and discolour, becoming less evident with time.

As indicated above, RMC is a matrix comprising large stones, or plums, embedded in a mortar binder. To minimize cost and optimize the structural properties, it is beneficial to maximize the stone content and 55% rock and 45% mortar is typically achieved. Two cement contents are generally used for the mortar component, with one part cement to four parts sand giving a 14 MPa strength and one part cement to six parts sand giving 9 MPa. Rock particles will generally range in size from 50 mm to 300 mm, the largest size being dependent on the width of the member under construction and the physical restriction of manageable weight.

The surface finish may be plastering to create an impression similar to concrete, or recess pointing to achieve a finish similar to stone-pitching.

3.7 METHODS OF CONSTRUCTION AND MATERIALS REQUIREMENTS

3.7.1 FOUNDATION SURFACE TREATMENT

For all concrete gravity and arch dams, the foundation preparation will typically involve both mechanical and hand finishing of the rock excavation. This will comprise clean-up of fissures, joints, seams and crevasses, their subsequent filling with dental concrete and possibly stitching grouting. This is a very labour-intensive activity with a high cost which should be included in the cost estimates.

3.7.2 CVC

Conventionally vibrated concrete will be used on all dams in spillways and other ancillary works. Smaller concrete dams may be made economically with CVC rather than RCC.

Various methods of mix design are used with those of the ACI (2017) being in common use internationally. Mass concrete mix designs may be made following 211.1-91: Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete in ACI (2017).

CVC in concrete dams is placed in vertical blocks, see Section 3.7.4, as a succession of vertical lifts. Various methods of transporting the concrete to the dam and into the formwork are used with cranes and buckets being common.

3.7.3 RCC

Most concrete gravity type dams are now constructed using the Roller Compacted Concrete (RCC) method where the concrete is dumped on the dam, spread and compacted by heavy vibrating rollers in 300 mm thick layers (ICOLD Bulletin 177). This technology allows rapid construction compared to CVC and significantly lower costs for all but the smallest dams. The setting time of the RCC is commonly delayed, so that successive layers are placed on layers which have not set. Where this is not possible, there are methods for treating the lift joints to ensure that an adequate bond is obtained. The density and strength of RCC is commonly the same or better than CVC for the same cementitious content. Most commonly the faces of the dam are formed with RCC against formwork, where the RCC has been enriched with grout to obtain a mix that can be consolidated with internal vibrators. In frost-prone areas the facing has to be frost resistant which is achieved by air entrainment of the facing concrete, be it enriched RCC or CVC. If the RCC has a low cementitious content, say less than 180 kg/m^3, adequate air entrainment may not be possible and a higher cost CVC facing may be required. To control thermal cracking, the dam body is split into blocks by joints running upstream to downstream for the full height of the dam. The space between joints is typically in the range of 15 to 30 m and depends on placing temperature, adiabatic heat gain, the long-term environmental temperatures and the tensile strength of the concrete. A joint spacing of 20 m may be adopted for preliminary designs. The state-of-the-art use of RCC in dam construction is described in ICOLD Bulletin 177.

RCC typically contains about 2.2 t/m³ of aggregate of which 40% may be fine aggregate (sand). The RCC will contain cement and pozzolan, commonly flyash, although also natural pozzolan can be used. Most dams are designed with high cementitious contents (150 to 220 kg/m³). Low cementitious content dams (< 150 kg/m³ and down to 60 kg/m³) may not contain sufficient paste to make a watertight concrete and watertightness is then obtained with an upstream membrane or adding bedding mortar between lifts, or both. Paste content may be increased with added inert fines which may make RCC with cementitious contents lower than 150 kg/m³ watertight. Cost estimates can be made on the basis of such a design, but unless there are compelling reasons to do otherwise, a cementitious content of 180 kg/m³ (40% cement, 60% pozzolan) can be assumed for cost estimating purposes.

In high gravity dams, internal zoning of the RCC is may be used. In internal zones that have lower stress levels, it is possible to use a lower cementitious content in the RCC, see Figure 3-22.

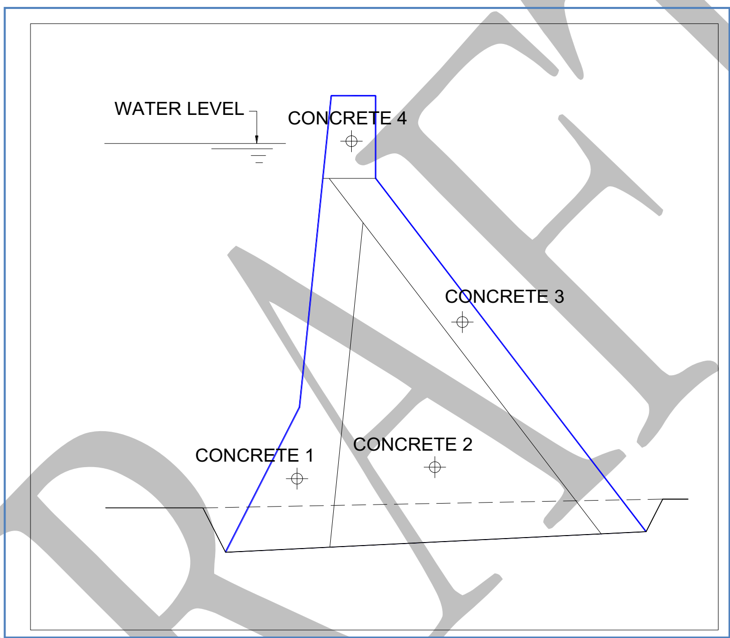


Figure 3-22 Possible internal zoning of an RCC gravity dam

3.7.4 JOINTS

Due to thermal and construction conditions concrete dams comprise a series of structurally independent blocks delimited by formwork joints spaced typically every 15 to 25 m that are set perpendicular to the dam axis. This applies to gravity and arch dams. No joints are provided in the body of FSHD.

Dams that rely on the arch action for stability typically require formwork joints in the dam to be grouted in order to maintain the arch action, design shape and stresses of the dam. Thus, in arch dams, blocks are not independent.

In roller compacted concrete (RCC) dams, transverse joints are typically not formed by formwork and induced by inserting a fabric or metal bond-breaker.

In arch RCC dams, when the transverse joints need to be post-grouted, systems have been developed to allow for this while disturbing as little as possible the RCC placing operation during construction (ICOLD Bulletin 177).

3.7.5 MATERIALS

Advice on the selection of materials for concrete in dams is given in ICOLD Bulletin 165.

3.7.5.1 AGGREGATE

Concrete aggregate has to be obtained either from a quarry or a borrow pit in alluvium within reasonable distance from the dam.

At early project stages, the estimated quantity of source rock or natural gravels available for concrete aggregate should be not less than three times the estimated requirement. As the project progresses towards a tender design, this may be reduced to a factor of two. The reasons for these factors are uncertainties in the estimates of available quantities and wastage in the quarry and of processed aggregate.

Quarries inside the future reservoir to limit environmental impact are generally preferred. A distance of a quarry from the dam of one to two kilometres may be a suitable compromise between limiting transport distances and preventing blasting from having an impact on dam construction. The quarry location should include consideration of space for aggregate processing and storage.

The aggregate may be all-crushed or partly crushed if the source is alluvial. Fine aggregate will typically require crushing or milling, whichever source is used. Transport distances should be considered in establishing a unit rate for aggregate.

The cost of quarry development and restoration of the quarry or borrow areas should be considered and added to the cost of the aggregates.

3.7.5.2 CEMENT

ASTM Type I or Type II are commonly used with Type I, being the most readily available and cheaper, often preferred when used with a pozzolan. Cement will be required in substantial quantities and sources should be determined to establish costs. The cost at site will comprise the cost ex-works and transport costs. A long transport distance is not a fundamental impediment but does affect costs.

3.7.5.3 POZZOLAN

Pozzolanic materials are cementitious materials activated by the products of hydration of cement. Their use has several benefits and the major ones are:

1. Lowering the cost of cementitious materials.
2. Lowering the heat of the concrete.
3. Increasing the paste content
4. Improving workability
5. Prohibiting alkali reaction of the aggregates

Pozzolan is preferably low-lime flyash (ASTM Type F) but can be natural pozzolan. High lime flyash (ASTM Type C) can be used if not too far removed chemically from the Type F boundary. ICOLD Bulletin 165 gives details on sources of pozzolan and its use.

There is increasing scarcity of flyash due to reduction of coal-fired power generation and natural pozzolans are likely to be more dominant in future dam construction.

3.7.6 RCC PRODUCTION AND MATERIAL STORAGE

RCC is produced in high capacity batch plants. The factory may include means of cooling the concrete (wet-belts, ice, chilled water, chilled air). The cost of providing this facility should be included in the unit rate for RCC.

Cementitious materials are stored in silos. The quantity required will depend on the maximum RCC production rate and the security (reliability) of the supply. With high reliability the quantities might be equal to 2 weeks of production but for remote sites with difficult transport this might have to be much larger.

Areas need to be identified for the aggregate plant and storage. Wherever possible, there should be space for processed aggregate sufficient of 3 months of peak production of RCC or one third of the total aggregate demand, whichever is the smaller. Successful dams have been constructed with storages equal to 10 to 14 days of peak RCC production as long as the weekly production of the crushing plant is at least equal to the weekly consumption of aggregates by the RCC placed.

Any major costs resulting from these requirements should be identified and included in the estimates.

3.7.6.1 RCC TRANSPORT AND PLACEMENT

For most large dams, conveyors are used to transport RCC from the batch plant to the dam. On the dam the RCC is usually transported on trucks but conveyors can be used also for this purpose. The cost of transport, placement and compaction will be included in the unit rate for RCC.

Table 3-4 RCC placement rates

	Dam - Country	Max Height (m)	RCC Volume (m^3)	Max m^3/hr	Max m^3/day	Average m^3/day	Max m^3/ month	Average m^3/ month	Plant utilisation %
1	Grand Renaissance - Ethiopia	160	10,099,770	966	23,197				
2	Gibe III - Ethiopia	246	6,200,540	771	18,518		250,889		23
3	Ta Dan, Thailand	95	4,900,000	700	13,280	4,009	201,490	122,266	24
4	Longtan - China	178	4,623,263		18,475	4,757	400,754	142,758	18
5	Son La – Vietnam	138	2,676,707		9,919	2,797	200,355		
6	Yeywa – Myanmar	134	2,471,806		7,555	2,373	125,009	72,264	
7	Yukari Kalekoy, Turkey	150	2,448,000	500	11,330	3,319	173,600	100,965	28
8	Beydag – Turkey	96	2,350,242			3,700	164,999	112,566	33
9	Nam Ngiep 1	167	2,288,080	400	9,426	3,360	188,209	102,196	35
10	Taum Sauk - USA (Missouri)	49	2,179,746	500	9,426	3,053	209,068	98,492	27
11	Lai Chau - Myanmar	131	1,883,863					73,023	
12	Beni Haroun - Algeria	118	1,689,666	382	9,152	3,382	174,624	102,833	37
13	Miel I - Columbia	188	1,669,023		7,141		118,735	62,908	
14	Ralco - Chile	155	1,595,626	550	7,791	2,515	142,742	127,298	32
15	Miyagase - Japan	155	1,556,634		5,153		116,977		
16	La Brena II – Spain	119	1,441,186			2,323	130,529		

17	Al Wehdah – Jordan/Syria	103	1,426,659		6,185	2,473	123,858	75,171	
18	Porce II - Columbia	123	1,299,743		6,347		87,731		
19	Upper Stillwater - USA (Utah)	90	1,124,660	838	8,410		204,442	125,311	20
20	Three Gorges - China (Longitudinal Cofferdam)	94	1,106,311	1365	21,063	9,068	476,012	274,399	28
21	Jiangya - China	131	1,100,194		7,183		121,029		
22	Olivenhain - USA (California)	94	1,036,736	825	12,271	4,008	225,131	121,870	21
	Saluda – USA (South Carolina)	63	1,003,861		14,135		76,991		
23	Upper Paung Laung - Myanmar	101	935,815					26,675	
24	Pirris - Costa Rica	113	750,028					51,684	
25	Elk Creek - USA (Oregon)	25	266,065	774	9,473			55,736	10

Note: Plant utilisation = (average monthly placement) / (maximum theoretical monthly placement), max theoretical monthly placement = plant capacity (m^3/h) x (number of hours in month = 24 x 30.4)

3.8 CONSTRUCTION SCHEDULE

The construction schedule has to be estimated as it may significantly impact the choice of dam type. The principal elements are:

1. Access and site contractor's facilities
2. Development of quarries and borrow areas
3. Diversion
4. Foundation excavation and preparation
5. Grouting
6. Dam body
7. Spillway and other water outlets

Climate can have an effect on the schedule as certain operations can be delayed or take longer because of rainfall or extreme temperatures. These effects must be assessed and taken into account.

CVC placements can be made under freezing temperatures. Formwork and concrete may have to be heated and the completed pour thermally insulated. RCC placements may be made during daytime when the air temperature is above zero and then protected with insulating mats during sub-zero night-time temperatures.

High air temperatures and solar radiation can lead to undesirable heat gain of RCC. Under these circumstances placements are commonly made during night with evaporative cooling of curing water controlling temperatures during daytime.

Placement of CVC is normally made intermittently and may not be significantly affected by rainfall. Covers can be used.

RCC has been placed in climates with very high annual rainfall with only moderate effects on productivity. The timing of placements should take into account the rainfall patterns. Placements can be stopped at short notice and the works are can be covered.

3.9 MAJOR COST ELEMENTS

The major cost items required for dam type selection are shown in Table 3-5.

Table 3-5 Major cost elements

	RCC gravity dam		**Arch dam**
1	Foundation excavation	1	Foundation excavation
2	Foundation preparation	2	Foundation preparation
3	Concrete volume (dam body)	3	Concrete volume (dam body)
4	Mass of cement and pozzolan	4	Mass of cement and pozzolan
5	Facing area of the dam (formwork area for CVC dams)	5	Formwork area
6	Internal galleries and drainage	6	Internal galleries and drainage
7	Foundation drainage and grouting	7	Foundation drainage and grouting
8	CVC volume for RCC dams: (levelling concrete, spillways, outlets)	8	CVC volume for levelling concrete, spillways, outlets
9	Joints and waterstops	9	Waterstops and grout stops
10	Instrumentation	10	Provisions for grouting joints
11	Outlet works and control gates/valves	11	Outlet works and control gates/valves
	Concrete cooling or heating included	12	Instrumentation
			Concrete cooling or heating included
	Cemented Materials Dam		
1	Foundation excavation	7	Internal galleries and drainage
2	Foundation preparation	8	Foundation drainage and grouting
3	Concrete volume (dam body)	9	CVC volume for spillways, outlets
4	Facing concrete	10	Waterstops
5	Mass of cement and pozzolan	11	Outlet works and control gates/valves
6	Formwork area		

The amount of cement and pozzolan is estimated from the assumed quantities of each per cubic metre. The cost of the facing will be the unit rate for formwork and grout enriched RCC times the face area or CVC where this is a requirement to confer frost resistance. For CVC dams the formwork for the transverse joints will have to be included, not just the face areas.

Item 7 should be based on an assumed depth of grout and drainage curtain. The grout curtain may be assumed to be 60% of the local dam height subject to a minimum of 10 m. The drainage curtain can be taken as $^{2}/_{3}$ of this.

Some of the above items would be estimated as percentages of the cost of the mass concrete (CVC or RCC, items 3 and 4 above). Rates vary but might be:

- Waterstops in gravity dams: 1%,
- Waterstops and grout stops in arch dams: 2%.
- Post-cooling of concrete: 4 to 6%.
- Galleries and drainage curtains: 1.5%.
- Instrumentation: 0.5% can be ignored

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4 EMBANKMENT DAMS

4.1 INTRODUCTION

Embankment dams may be constructed predominantly from rockfill or from earthfill.

Earthfill dams are constructed from native materials containing fine particles that dominate the stress-strain and permeability properties. A typical criterion for a dam being of earthfill and not rockfill is that the average particle size should not exceed 50 mm with 50 to 60% of the material passing a 25 mm screen (inverted from Lepps, 1988). Earthfill dams may be homogeneous but are more commonly zoned embankments.

Thus rockfill, being other than earthfill, is normally free draining and pore pressures are not a major factor in the design. The slopes of the rockfill embankments reflect the strength of the fill and are relatively steep. The fill is most commonly derived from quarried rock but can also be natural coarse gravel.

Rockfill dams comprise a number of sub-types according to the nature of the impervious element. The choice of sub-type will depend on a range of factors where cost may be the major one.

8. ECRD: Central Core Rockfill Dams

This dam type comprises almost exclusively local materials. It demands a sufficiency of low permeability material for its core. It is less sensitive to foundation settlements than other types of rockfill dam. Clayey core materials are typically weather-sensitive which makes for slow progress in rainy weather. The principles of design for an ECRD are the same as for an earthfill dam and the earthfill dam section of this bulletin applies also to this dam type.

9. CFRD: Concrete Face Rockfill Dams

This type of dam is sensitive to settlements along the junction of the facing and the foundation. It is well suited where a rock foundation is present but has been used on coarse alluvium and other strong soil.

10. ACRD: Asphalt Core Rockfill Dams

The asphalt core is normally central in the dam cross-section and is founded on rock.

11. AFRD: Asphalt Facing Rockfill Dams

The asphaltic concrete facing can deform without cracking which makes this dam type suitable where substantial foundation settlements can be expected.

12. Geomembrane face or core dams

Facings (or less commonly cores) using plastic membranes are normally employed only on small dams (< 15 m high) but higher dams have been made.

The choice of dam type will depend on material availability, foundation strength and dam height. As a general rule, rockfill dams require a stronger foundation than earthfill dams.

Embankment dams are normally designed with spillways outside the dam body. Spillways on dam embankments should be avoided as any damage to the spillway could lead to catastrophic erosion of the dam.

A number of design aspects apply to more than one type of embankment dam:

- Freeboards and zone heights
- Crest widths and core widths
- Dimensioning filters and drains
- Earthquake settlement and camber

- Foundation requirements
- Slope protection
- Spillways
- Integrated cofferdams

These aspects are treated under separate headings.

In this Bulletin the following terms are used:

- Core: the water-retaining element.
- Shells: fill material forming the exterior sections of the embankment
- Filters: a zone or layer that that is incorporated into a dam and is graded to prevent the migration and erosion of soil materials into adjacent foundation or fill materials as seepage water flows to the drain. The filter should accept seepage without the build-up of excessive pore water pressures.
- Drain: layer of permeable material to relive pore pressure by facilitating drainage.
- Riprap: A layer of durable material for bank protection and stabilization usually consisting of rock or stone.

4.2 COMPOSITE DAMS

Composite dams comprise two or more adjoining dam types. Common to all are varying foundation conditions along the length of the dam axis. These may comprise rock in some sections with deeply weathered rock or drift elsewhere. A common configuration shows a gravity dam on rock in the river flanked by embankment dams. Junctions between embankment dams and concrete gravity dams may be formed with wrap-arounds (Figure 4-2) or with retaining walls (Figure 4-3).

A number of embankment dams have spillway structures on an abutment which form part of the dam. These are not commonly considered as composite dams.

Figure 4-1 Composite dam: embankment wrapped around concrete gravity dam

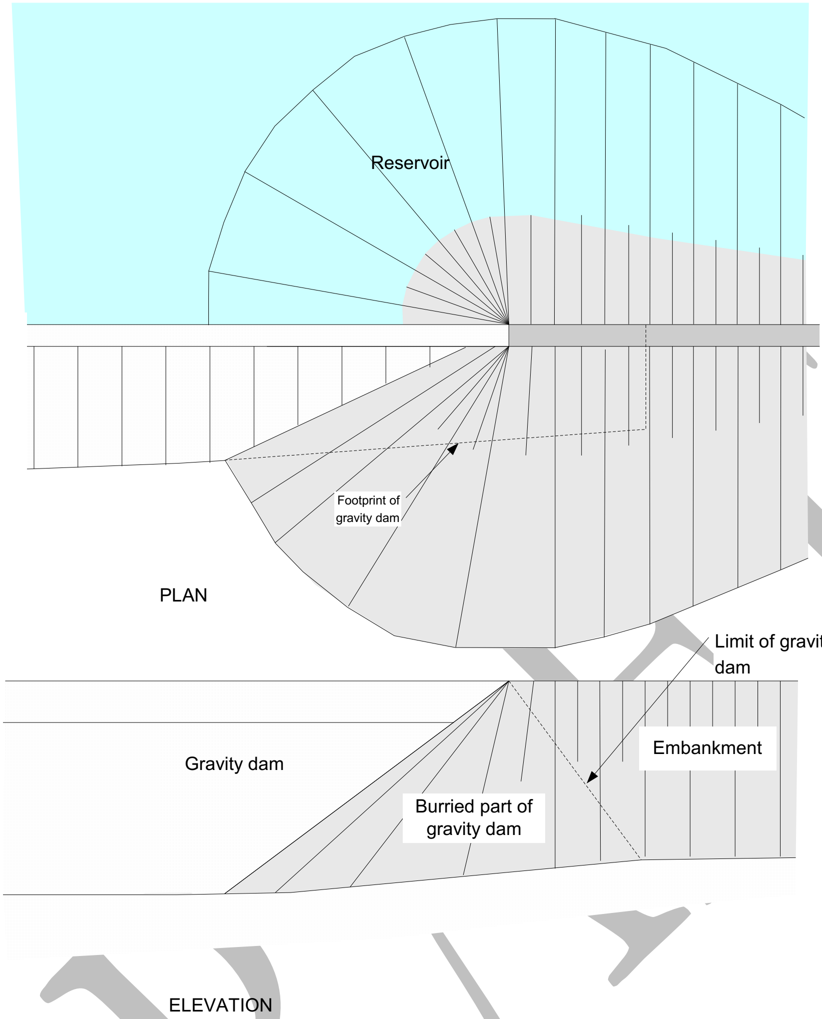


Figure 4-2 Wrap-around junction between embankment and gravity dam

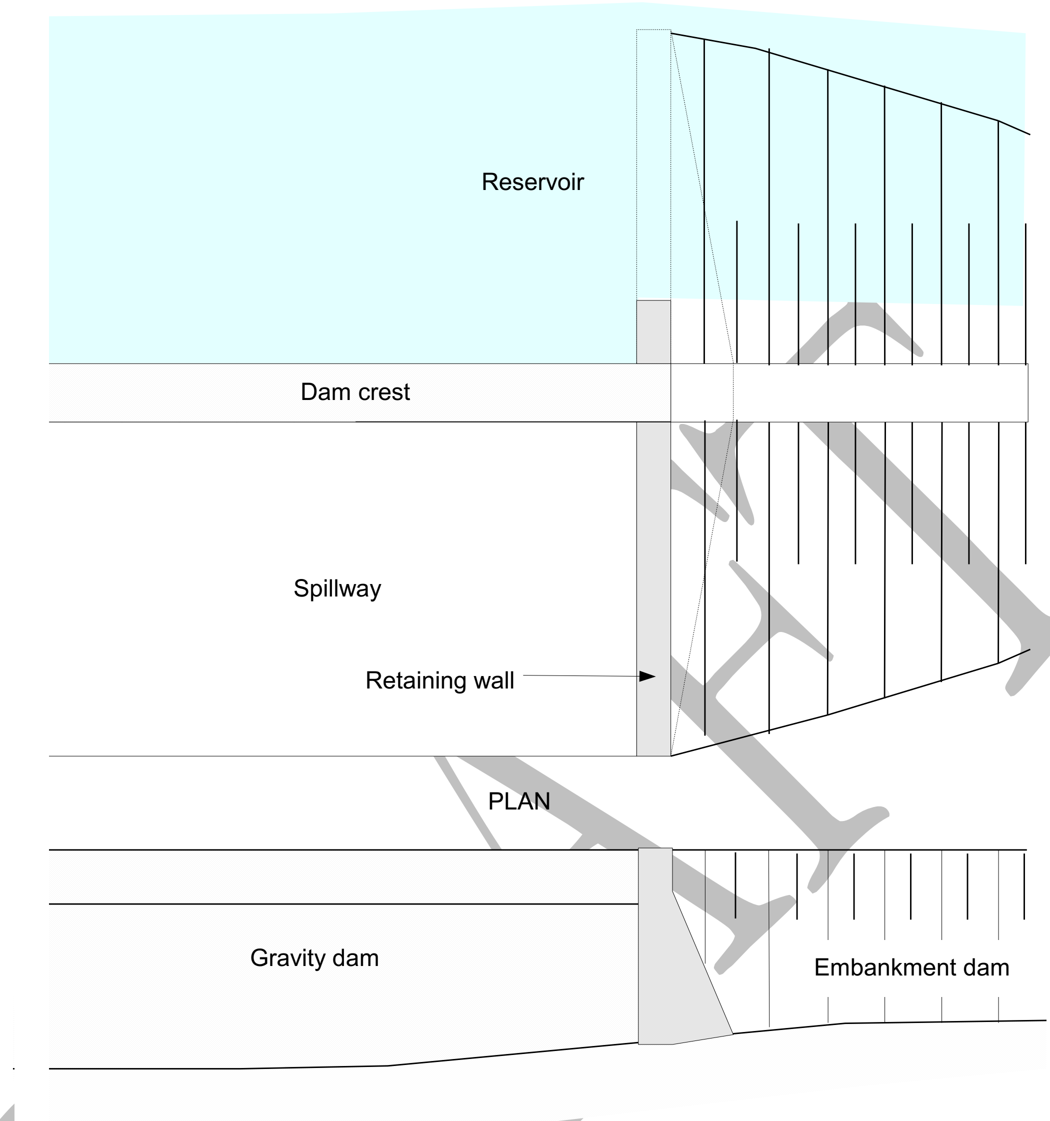


Figure 4-3 Connection between gravity and embankment dam with retaining walls

4.3 EARTHFILL DAMS

4.3.1 OVERVIEW

A number of publications provide detailed guidance on the design of earthfill dams (USBR, USACE) and these may be consulted when an elaborated design is required. The USBR is producing a series of *Design Standards for Embankment Dams*, which is a partial update of Small Dams but the latter is still a useful reference. A number of ICOLD Bulletins may be consulted on various aspects of design and construction as shown in the References.

The native superficial materials may be derived from diverse sources such as alluvial and lacustrine deposits, loess, glacial till and deeply weather rock including residual soils. There is no blasting involved in obtaining these materials and they are not processed, with the possible exception of filter and drain materials. The riprap protection to the upstream slope is commonly obtained from quarried rock. In the interest of economy, material excavated from the dam foundation and other project components should be incorporated in the dam to the maximum extent possible.

Central to the design of such dams is the control of seepage and pore pressures.

Earthfill dams are of two main types:

- Homogeneous dam comprising one material type. Modern dams include internal drainage to control pore pressures, internal erosion and phreatic surfaces.
- Zoned dams containing two or more material zones with different permeabilities and strength.

The choice of dam type will depend on material availability, foundation strength and dam height.

Most foundation materials are in principal suitable for earthfill dams. The nature of the foundation (strength, permeability) can have major influence on the design and cost of the dam. Liquefiable soils such as fine sand, dispersive soils and soft clays may not be suitable and would require special study.

In the following text certain terms are used:

- Core: the water-retaining element wherever it be located
- Shells: fill material forming the exterior slopes of the dam

4.3.2 DESIGN PRINCIPLES

4.3.2.1 THE EMBANKMENT

Homogeneous dams require incorporation of internal drainage in the downstream shell to control pore pressures, see Figure 4-4. The chimney drain (B) should extend to the level of the PMF. This prevents uncontrolled seepage and possible failure at exceptionally high reservoir elevations as the upper part of the embankment fill may dry out and crack as it is normally above reservoir level.

Dam with a height of less than 10 m may be constructed with a blanket drain only and dams less than 5 m high with only a toe drain (Bulletin 157).

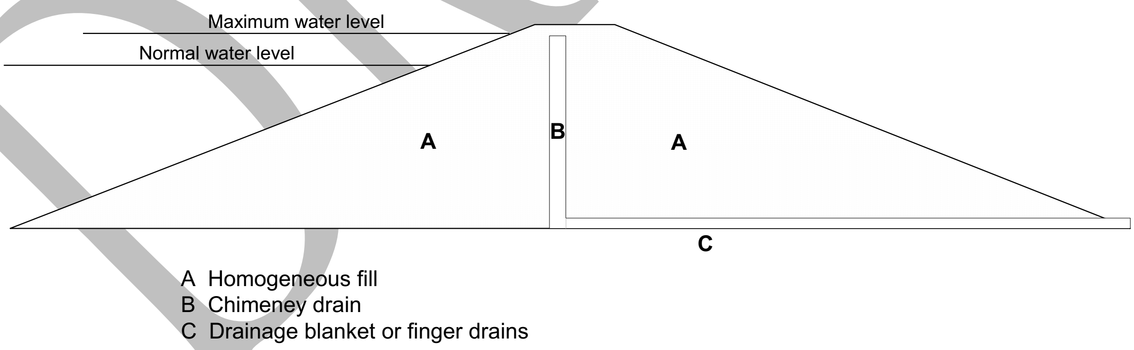


Figure 4-4 Homogeneous dam with internal drainage

Zoned dams contain a fine-grained soil core as the water barrier. The core may be in the centre part of the dam or inclined near the upstream face, Figure 4-5.

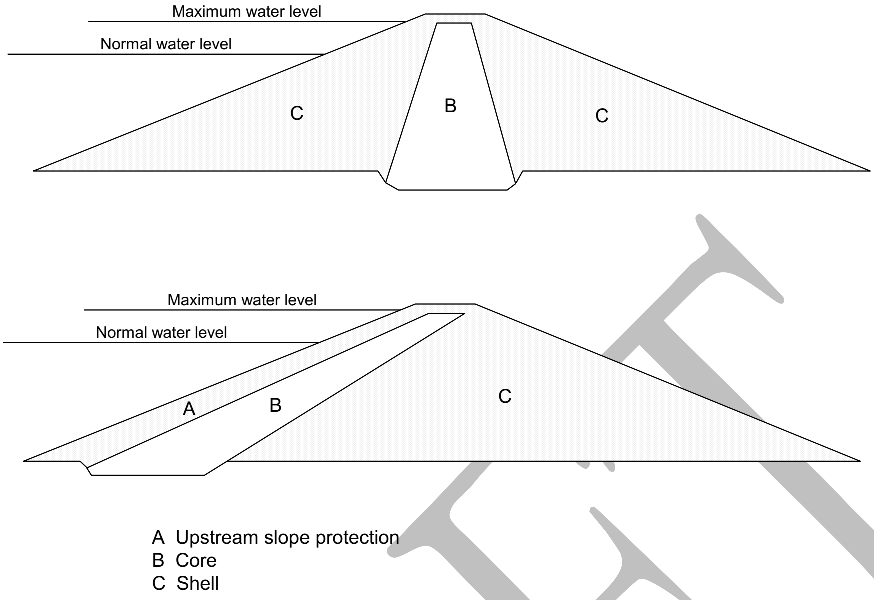


Figure 4-5 Core locations in zoned dams

The shells may comprise several different materials depending on the quantities and nature of the available soils.

The following principles may be used in configuring the dam:

- Stronger materials are used in the outer part of the shells where the shear stresses are highest
- With a central core the zones next to the lower part of the core are subject to low shear stresses and low strength material can be placed here. However, such material should have a compressibility (stiffness, consolidation characteristics) that is not inferior to that of the core.
- The upstream face has to be stable during rapid drawdown of the reservoir and the zone so affected should comprise strong and free-draining material. There is a relationship between the permeability of this zone and the slope required for stability.
- The material under an inclined core should comprise materials which give rise to only moderate settlement
- The upstream face of the dam requires riprap to prevent wave erosion. Cast slabs or blocks may also be used but are far less common. The downstream face also requires protection which may range from grass cover for small dams in wet climates to coarse gravel or boulders (small riprap) for large dams
- The permeability of the materials should increase from the core and outwards.

The core material may range from high plasticity clay to silty sand which may be gravelly. Suitable glacial till will fall into the latter category.

Rockfill is occasionally used in dams which are predominantly earthfill, such as in downstream part of the downstream shell, in order to steepen the slope and reduce fill volumes. This feature may be incorporated in response to a shortfall of suitable earthfill, a need to reduce the footprint of the dam or to use rockfill arising from various excavations including the spillway. Thus, there is no sharp boundary between a dam described as earthfill or rockfill.

In order to make an outline design, certain minimum knowledge of material properties is required. This includes gradings of all materials as well as Atterberg limits for fine-grained soils. Strength and consolidation properties can be assessed by various correlations from these characteristics. In the absence of further testing, the range of strengths can be characterised by assignment of the most likely, the probable maximum and the probable minimum values. Where the strength of the fill materials dominates stability, the range of strengths will yield different embankment slopes and thus different costs. Different consolidation properties of the core material and the foundation will give different settlements, and thus camber, with corresponding differences in dam volume as placed. However, camber due to core settlement accounts for typically not more than 1% of the dam volume and is thus a minor contributor to cost.

Zoned dams will contain granular filters and drains. Filters are required to satisfy filter criteria and thus prevent migration of fine soil particles from a fine to a coarse zone under a hydraulic gradient. Drainage will be required where there is a risk of pore pressure build-up, for example where the downstream shell has a low permeability. The filter may also function as a drain where the relative permeabilities so allow.

The width of the crest must be such that vehicles and equipment can obtain access for maintenance. The number of zones near the crest of the dam may also affect the width.

Regulations in some countries require the crest to be covered with very large boulders (1.5 to 2 m diameter) so as to prevent damage due to acts of sabotage or war, or at least make such damage difficult to inflict.

4.3.2.2 FOUNDATION

A low strength foundation may dominate the stability of the shells as potential sliding may be along the embankment to foundation interface or may be deeper. Conversely, a strong foundation will not influence the required slopes of the shells: potential slip surfaces will be entirely within the embankment fill. This will apply to all rock foundations.

To avoid cracking of the core, abrupt changes of its foundation geometry with adjacent areas should be avoided. The foundation surface should have a smooth profile, avoiding abrupt slope changes or steps.

Foundation permeability will affect seepage losses and possibly pore pressures within the dam. Most dams will contain some form of seepage cut-off below the foundation of the core. The core itself will normally be founded below the foundation level of the shells. A core trench is excavated, preferably to the full width of the core as seen in cross-section. Its depth will depend on the geology but should be brought down to a low permeability horizon where reasonably feasible. A seepage cut-off is commonly provided below this, as described below.

With weak foundations that affect dam stability, pore pressures in the foundation below the downstream shell have to be controlled.

As an alternative or supplement to a cut-off under the core, an impervious or low permeability blanket can be constructed upstream of the dam to lengthen the seepage paths. The correspondingly reduced hydraulic gradient gives reduced water pressures under the dam and reduced seepage losses. Pore pressures in the foundation may also be controlled with drainage wells located at the toe of the dam. To prevent a rise in pore pressures in a downstream shell due to foundation seepage, a blanket drain extending from the core to the dam toe is commonly incorporated. Filter criteria have to be satisfied between the blanket and underlying strata.

Seepage cut-offs below the core trench (see Bulletin 150) may comprise:

- A grout curtain, most commonly used with rock foundations but alluvial grouting can be done
- A concrete wall (diaphragm wall or slurry trench), used with alluvial foundations
- Sheet piles, secant piles or other driven or bored continuous walls used with alluvial foundations

The grout curtain may be made from the bottom of the core trench but if the foundation is weak, it may be made after the first few metres of core fill have been placed. Concrete walls and piles will be stiff relative to the foundation strata. Differential settlements induced by the weight of the embankment may cause these stiff elements to protrude into the core fill and disrupt it. The junction between rigid cut-offs and the dam core have to be made such that piping along the interface cannot occur. In general, grout curtains are preferred.

4.3.2.3 FREEBOARDS AND ZONE HEIGHTS

Outline freeboard requirements are shown in Sections 2.6.2 and 2.6.3 and further detailed in Section 4.9.1.

The elevation of the core should be no lower than the design flood level. Above this it should also extend to the PMF reservoir elevation or this upper zone might comprise filter material backed up by a drain on its downstream side. This is done to reduce the number of zones at the top of the dam. The various zones have to be of certain minimum widths to allow effective construction. In cold climates the top of the core must be below the frost penetration zone with material above it not susceptible to frost damage. Insulation sheets are commonly employed in this context.

4.3.2.4 INTEGRATED COFFERDAMS

The upstream cofferdam required to direct the river flow into the diversion conduit (tunnel or channel) is commonly large. In the interest of economy, and wherever possible this structure should be incorporated into the upstream shell, see Figure 4-6. It may be possible to incorporate also the downstream cofferdam but as this is normally much smaller than its upstream counterpart, so cost benefit is correspondingly small.

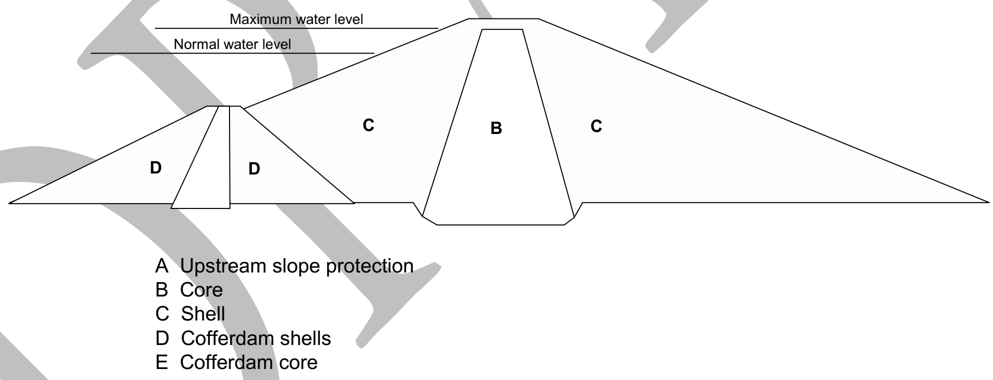


Figure 4-6 Earthfill dam with integrated cofferdam

4.3.3 MAKING A PRACTICAL OUTLINE DESIGN

4.3.3.1 MATERIALS INVESTIGATION: QUANTITIES AND PROPERTIES

Section 2.5 gives guidance on the required site investigations.

There will be uncertainties in the volume estimates of available materials, particularly in early project phases when there are limited investigations. The actual quantities that can be obtained for construction should be taken as one third of the estimated total quantity in the ground for early project phases (e.g. pre-feasibility) and one half for intermediate phases (e.g. feasibility).

Material arising from required excavations (spillway, abutment, dam foundation) should wherever possible be incorporated in the dam body. Some material, e.g. from the abutments, might require double handling which might make re-use uneconomical.

4.3.3.2 MATERIAL ZONES

Completed earthfill dams show a considerable range of material zoning. The complexity arises from heterogeneity of material properties in borrow areas. Knowledge of the quantities and their properties increases from the early design stage until the dam is complete. The zoning of the dam will change in response to the quantities of the various materials, i.e. there will be a trend towards increasing complexity. At the early design stages typical for dam type selection, knowledge of the material availability and properties will be limited and the zoning of the dam relatively simple.

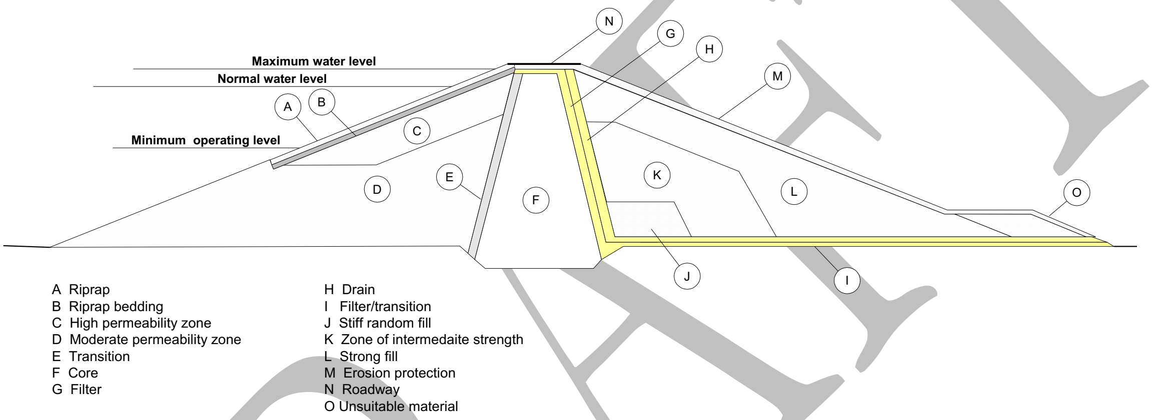


Figure 4-7 A zoned embankment dam, central core

Figure 4-7 shows the principal zones for a central core earthfill dam. The zoning of a rockfill dam is similar to that here described except that the shell fill has a higher permeability and strength than earthfill which leads to steeper embankment slopes.

The zones comprise:

1. Riprap which extends down to the minimum reservoir operating level
13. Riprap bedding
14. A zone of relatively high permeability and high strength which will give stability to the dam during rapid drawdown, normally gravel or gravelly sand, possibly rockfill arising from excavations
15. As C, but may have a lower permeability
16. Transition zone between the core and the potentially coarse upstream fill (zones C and D). If zones C or D is not coarse, e.g. sandy, then zone E may not be required
17. Core of the dam comprising a homogeneous low permeability material which may range from plastic clay to silty sand, possibly containing gravel. Granular core material should be well graded.
18. Filter to prevent piping of core material into the downstream shell. This is commonly extended over the foundation of the downstream shell to prevent pore pressure build-up in the foundation
19. Transition layer and drain between filter and downstream shell

20. A drain to carry seepage water to the downstream toe when the downstream shell is constructed from low permeability material. Alternatively, the zone may function as a transition between the filter and the shell fill when the latter is coarse and free draining.
21. This zone can be used for material that is deemed too weak for inclusion in the outer part of the downstream shell.
22. This zone can be used for material that is deemed too weak for inclusion in the outer part of the downstream shell but is stronger than the zone J fill.
23. The strongest fill available, preferably gravel or gravelly sand.
24. Downstream slope protection
25. Roadway
26. Random fill (waste), not suitable for inclusion in the dam

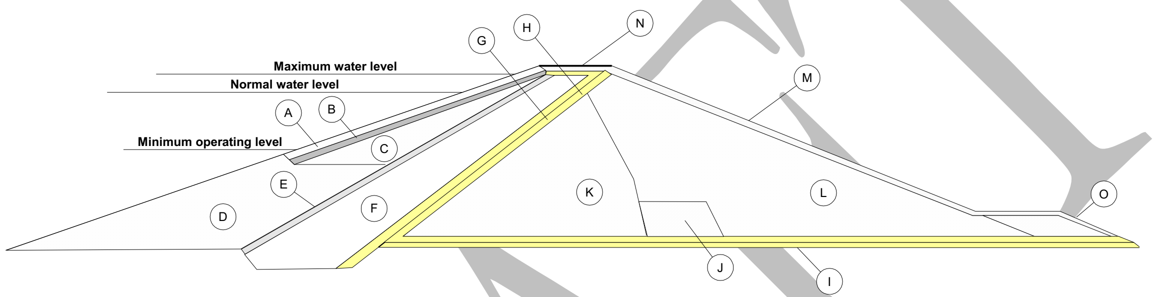


Figure 4-8: Zoned embankment dam with sloping core

Figure 4-8 shows a typical configuration of an earthfill dam with a sloping core. This configuration may be used when there is insufficient core material to make a central core with suitable side slopes, see section 4.3.3.4. As the relatively weak core material now has a greater influence on the stability of the upstream slope, this slope has to be made flatter. Zone K should be relatively stiff to keep deformations of the core as small as possible. The characteristics of the various zones are the same as for a central core dam.

The shell fill may be whatever is available. Successful dams have been constructed using plastic clays though to sand and gravel with the latter having the characteristics of rockfill.

Typical requirements for core fill are:

- A permeability coefficient of less than 1×10^{-7}m/s.
- An organic content (counted as per mass) of less than 2%.
- Little variation in volume in case of immersion or dehydration.

These requirements can be assessed from grading test and index test results.

Whenever certain types of clays are used, including alluvial clay with plasticity more than 20 % and liquid limit more than 40%, expansive soil, hard and dry clay difficult to excavate and compact, frozen soil and dispersive clay, special and particular studies are required to investigate their behaviour and effects on dam performance. High plasticity red clay may be used as the impervious core material but its compressibility should be evaluated specially when used for a high dam. When collapsible loess or loess-like soil is used as impervious core, appropriate water content and compaction criteria should be selected.

In case gravelly soils are used for the core, the content of particles with a grain size larger than 5 mm should not exceed 50%, and the maximum grain size should not be larger than 50 mm in a preliminary design. Often the content of particles with a grain size finer than 0.075mm is required to be more than 15% and the content of particles with a grain size finer than 0.005mm should be 8% or more.

In terms of cost, the shell fill can be assumed to have the same unit rate throughout, subject only to variations in haulage costs. Rockfill and natural gravels will have different unit rates, reflecting the different cost of obtaining these materials. The differences in cost due to different compaction requirements are second order and need not be considered in preliminary designs. The core, filters, drains and riprap with its bedding should be priced separately.

4.3.3.3 CREST WIDTH

The crest width is related to the height of the dam and the space required to accommodate the various fill zones and should be:

- A minimum of 3 m for very small dams only a few metres high
- Typically not less than 5 m to allow access for inspection and maintenance.
- For very large dams the crest with may be 10 m or more.

Where a highway is required on top of the dam, highway design standards may govern the crest width.

4.3.3.4 CORE WIDTH

The minimum top width of the core will be related to the size of equipment used for its construction and thus indirectly the size of the dam. As a general guide, a top width may be set at 4 m for dams that are 15 m high or higher. This may be reduced to 3 m for a 10 m high dam and 2 m for a 5 m high dam.

The core of the dam will typically settle more than the shell fill. In a narrow central core this can cause stress relief on horizontal planes with a risk of hydraulic fracturing of the core material. Cores with sloping sides are therefore preferred. Slopes should preferably be not steeper than 0.2:1 (h:v) with a base width in the range 40 to 50% of the water head. The implied hydraulic gradients are such that good quality filters are required to prevent piping of core material. The core can be made wider than this if there is abundant core material available.

If the core is founded on deep and pervious alluvium, the core width may be substantially increased if a cut-off into the foundation is not provided or such a cut-off is not effective in lengthening the seepage path in the foundation. The slopes of the core could then be 1:1 (45°) but the slopes of the shells would have to be flattened to achieve stability.

A shortage of suitable core material might force the core to be narrower and a sloping core (Figure 4-8) might be preferred as it is not subject to stress relief due to settlement. The hydraulic gradient at the base of the core should be not greater than 1/5.

4.3.3.5 DETERMINING FOUNDATION ELEVATIONS

Loose and compressible superficial material has to be excavated from the entire footprint of the dam. The depth of excavation will depend on the geology and the strength profile with depth. Ideally, the strength of the foundation should be such that it will not affect the stability of the embankment. The deeper the excavation, the more expensive this operation will be with a corresponding increase in the volume of embankment fill. Some measure of optimisation of excavation depth should be made for an outline design. Designing the dam with flatter slopes to compensate for weak foundation remains an option and the only solution where there is no prospect of significant strength increase with depth.

4.3.3.6 FOUNDATION TREATMENT

The core foundation has to be such that water percolating at the core to foundation interface cannot damage the core. There must be no risk of core material migrating into the foundation as this could lead to a piping failure. Dental concrete and mortar treatment followed by slush-grouting the whole core width are used on rock foundations to this end.

The core should wherever possible be founded on a low permeability stratum and will typically be founded deeper than the shells. A grouted or solid wall cut-off will be required under the core with the latter being applicable only to alluvial foundations.

The cost of preparation of core and shell foundation has to be included.

4.3.3.7 DETERMINING THE EXTERIOR SLOPES OF THE EMBANKMENT

The downstream slope of earthfill dams can be set initially at 2:1 (h:v) if it is constructed of gravel and might be 2.5:1 if gravelly sand is used. A weak foundation might require flatter slopes than this. The use of weaker material in the shells will require flatter slopes. There are examples of dams constructed with clay in the shells with slopes of 7:1.

Upstream slopes are commonly in the range 2.5:1 to 3:1 but can be flatter than this if weak and low permeability materials are used. Cedergren (1977) provides a method of determining the pore pressures set up in an embankment subjected to reservoir drawdown and which is applicable to pervious and semi-pervious fills. There will be little or no dissipation of pore pressures during drawdown with low permeability material and in this case the upstream slope of the dam has to be flattened to achieve stability.

Stability analyses should be made to verify and adjust the selected geometry.

4.3.3.8 DIMENSIONING FILTERS AND DRAINS

Filters and drains are commonly manufactured and have a relatively high cost. There is therefore an incentive to keep the width or thickness of these zones as small as possible compatible with methods of construction and assurance that minimum dimensions are maintained. The width is related to the equipment used in construction.

For most dams the equipment (trucks, dozers and roller compactors) will be relatively large and the thickness of vertical or steeply inclined filter and drains should be not less than 3 m.

The thickness of filters and drains on the foundation under the downstream shell should be 0.5 to 2 m depending on foundation roughness and drainage capacity. If nothing else is known, a thickness of one metre may be adopted.

Most filter materials will be processed. There is commonly a need for washing (removal of < 75μ) and correction of grading of natural materials. If a suitable source of natural material is absent, filters may be obtained by crushing and screening naturally occurring coarse alluvium or quarried rock. The unit rate for filters and drains has to be estimated from the anticipated process or suitable precedent.

4.3.3.9 SLOPE PROTECTION

All embankment dams require protection of the exterior slopes to prevent erosion, see Section 4.9.6.

4.3.3.10 EFFECTS OF WEATHER ON CONSTRUCTION PROGRESS

Rainfall has an adverse effect on construction progress. This can affect the core which may soften with access to water. Before core fill can be placed after rain, either the top of the core has to be allowed to dry out (repeated harrowing and air drying) or softened material is removed. Some granular core materials that have been compacted to a high density do not absorb water during rain, necessitating only evaporation of surface water and scarification before filling operations can resume.

Dam fill should be free of frost and core fill should not be placed in freezing conditions.

The design should include estimates of construction periods where the effects of weather have been taken into account.

4.4 EARTH CORE ROCKFILL DAMS, ECRD

4.4.1 INTRODUCTION

The design and construction of an ECRD is similar to an earthfill dam. except that the shell fill is stronger and less susceptible to delays in construction due to rainfall. This section should be read in conjunction with Section 4.3. The embankment volume is smaller as the faces of the dam can be made steeper. In particular, the free-draining rockfill does not give rise to excess pore-pressures in the upstream shell during reservoir drawdown with no attendant requirement to flatten the slope.

The core is designed in the same way and is subject to the same considerations with insignificant differences in the context of a preliminary design.

To get the full benefit of using rockfill, the foundation strength should be such that it does not govern dam slope stability: potential failure surfaces remain within the rockfill.

4.4.2 MATERIAL ZONES

An earth core rockfill dam typically involves an impervious core, filters, drains, transitions, rockfill shells and slope protection. The zoning shown for earthfill dams pertains also to rockfill dams, see Figure 4-7 and Figure 4-8. The rockfill shells are mostly free draining and pore pressures in excess of the external water levels are not considered in the design of the shells. Filters and transitions are required to ensure that fine material cannot migrate into coarser zones. Drains are incorporated where rockfill may have a fines content sufficient to prevent the free drainage of water.

When large differences of compressibility between core and rockfill shoulders are inevitable because of core characteristics, a sloping core is recommended to avoid excessive core hang-up.

The requirements for the core fill is the same as for an earthfill dam, see Section 4.3.3.2.

4.4.3 DETERMINING THE EXTERIOR SLOPES OF THE DAM

The slopes of the embankment are dependent on the shear strength of the shell fill, the core and the foundation. Where the foundation does not affect the stability of the dam, slopes have been constructed in the range 1.3:1 to 2.0:1 (h:v) with slopes in the range 1.5:1 to 1.7:1 being more typical. The upstream slope is usually slightly flatter than the downstream side due to reduced frictional strength caused by a reduction in effective stress.

A stability analysis is required based on assessment of strength parameters for the embankment fill and the foundation, see Section 4.9.8.

4.5 CONCRETE FACED ROCKFILL DAMS, CFRD

4.5.1 INTRODUCTION

A CFRD comprises a rockfill embankment dam with a concrete slab on the upstream face being the watertight element. The concrete slab is tied to the rock foundation with a concrete plinth along the upstream toe of the dam. The rockfill and the dam foundation have to have stiffnesses that limit deformation of the concrete facing to acceptable values.

Details of CFRD design requirements are given ICOLD Bulletin 141 (2010): *Concrete Face Rockfill Dams: Concepts for Design and Construction*.

4.5.2 EMBANKMENT SLOPES

Depending on the rockfill and foundation characteristics, seismicity and height, the upstream slope of a CFRD typically varies between 1.3:1 and 2:1 (H:V) and the downstream slopes 1.2:1 to 1.6:1. For initial dam type selection evaluations, it is recommended to use 1.5:1 for the upstream face and 1.6:1 for the downstream face.

4.5.3 EMBANKMENT MATERIALS AND ZONES

Typical zoning of a CFRD dam is shown in Figure 4-9. Main zones, from upstream to downstream, include a concrete face, a support zone, transition zone, main rockfill zone, downstream rockfill zone and drainage zones. The face of the support zone is commonly formed with concrete kerbs to form a firm base for the concrete face. A special stiff support zone is typically placed beneath the perimeter joint. For high CFRDs, an upstream blanket and a weighted cover zone is typically added on top of the lower part of face slab. Grading requirements for these zones can be found in Bulletin 141.

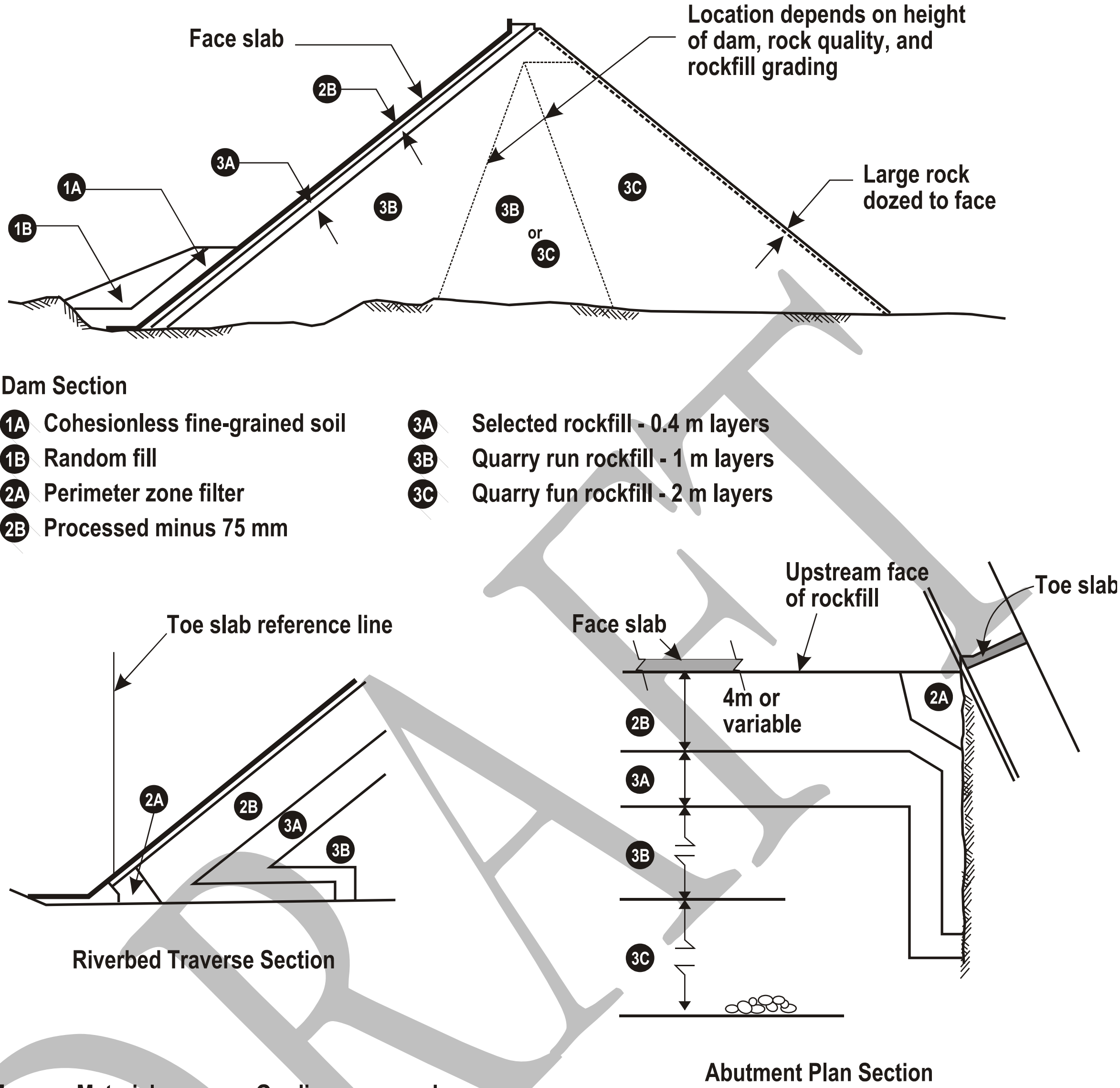
The minimum horizontal width of the support, filter and transition zones is governed by the method of placement and plant size and should not be less than 3 m with 4 m preferable. m is required for the support zone.

To avoid migration of fine material into the coarser rockfill, a transition zone between the support zone and the main rockfill zone is required. For constructability requirements, the horizontal width of the transition zone should not be less than 3 m.

Depending on availability of local materials, the support zone can be natural or manufactured.

When rockfill containing fines is used in the main rockfill zone, a vertical drainage zone in the upstream part of the dam and a horizontal drain at the dam base are required to maintain low pore pressures in the downstream shell. A filter may be provided on the upstream side of the vertical drainage zone when necessary. The crest elevation of vertical draining system should be higher than the normal storage level of the reservoir.

A downstream slope protection or a downstream rockfill zone is also required, see Section 4.9.6.



Zone	Material	Grading	Layer
2A	Filter	Minus 36 mm	0.4m
2B	Crushed rock	Minus 75 mm	0.4 m
3A	Rockfill	Minus 0.4 m	0.4 m
3B	Rockfill	Minus 1.0 m	1.0 m
3C	Rockfill	Minus 2.0 m	2.0 m

Figure 4-9: Zones for CFRD of sound rock fill and sound rock foundation
(ICOLD Bull. 141)

4.5.4 THE PLINTH

ICOLD Bulletin 141 provides extensive guidance on plinth requirements. To pre-dimension the plinth, a width of H/15 can be used, with H being the local hydraulic head. This width is established to control the hydraulic gradient along the foundation. The plinth is typically divided between the external and internal plinths. The external plinth is usually 4 m wide to give a platform for grouting operations. The internal plinth makes up the remaining length required to control the gradient. This allows reducing excavations for the founding the plinth, Figure 4-10.

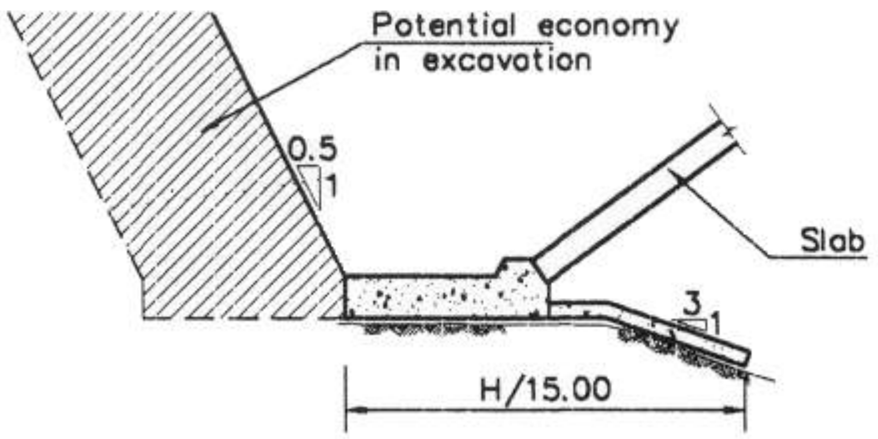


Figure 4-10: Typical Internal Toe Slab Cross-section
(from Marulanda and Pinto, 2000)

4.5.5 CONCRETE FACING

The concrete face slab thickness is commonly defined from the empirically derived equation $0.3 + 0.002 \cdot H$ in meters, where H is the local reservoir head. For smaller dams a constant thickness of 0.3 m is often adopted for ease of construction. A steel reinforcement ratio of 0.3% horizontal and 0.35% vertical is used. Details of the perimeter joint and joints between slabs typically involve several lines of defence, developed through experience and most recently modelling. Bulletin 141 provides a detailed discussion of joint details.

In order to reduce rockfill volume and facilitate compaction of the crest material, a parapet wall of 4 to 5 m height is typically included for this type of dam.

4.5.6 FOUNDATION

The plinth is normally founded on sound rock and a grout curtain is made from it.

Coarse alluvium may be an adequate foundation provide the foundation modulus is high enough. A positive cutoff may then be required to control the very high hydraulic gradients across the plinth.

Foundations with low modulus such as sand will give rise to large settlements when the reservoir load is applied and are not suitable for this dam type.

Bulletin 141 provides details of concrete face rockfill dams made in difficult and exceptional circumstances.

4.6 ASPHALTIC CORE EMBANKMENT DAMS, ACED

4.6.1 INTRODUCTION

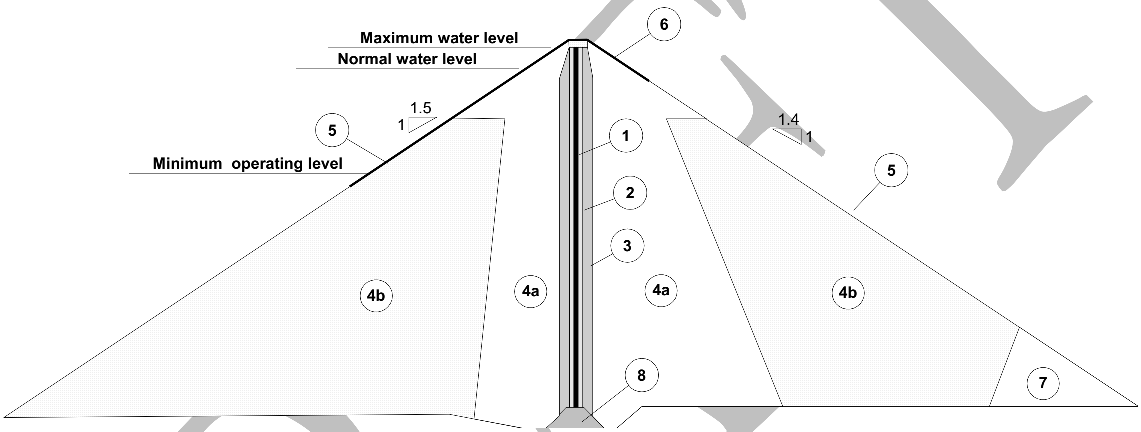
ICOLD Bulletin 179 provides detailed recommendations for the design and construction of ACRDs.

4.6.2 TYPICAL DAM CROSS-SECTION

The watertight element is the asphaltic concrete core. Rockfill shells provide stability. For a well-compacted embankment of good rockfill resting on bedrock, the dam slopes may be set at 1.4:1 (h:v) on the downstream site and 1.5:1 upstream. Slopes as steep as 1.3:1 to 1.4:1 have been used (ICOLD Bull. 179).

Transitions are required between the core and the rockfill. The rockfill adjacent to the core should have a high stiffness to limit settlements next to it. A typical cross section of an asphalt core rockfill dam with the usual zoning is shown Figure 4-11.

The core with filters and transition layer is founded on a concrete plinth which gives a level surface for the start of the core placement and a cap for foundation grouting. Typical dimensions are shown in Figure 4-12.



1. Asphalt Core
2. Transition (0-60 mm)
3. Transition (0-150 mm)

4a. Rockfill (0-500 mm)

4b. Rockfill (0-1000 mm)

5. Slope protection (blocks, minimum 0.03 m³)

6. Crown cap (blocks)

7. Toe drain (blocks, min 0.5 m³)

8. Concrete plinth (sill) for core

Figure 4-11. Typical cross-section of ACRD

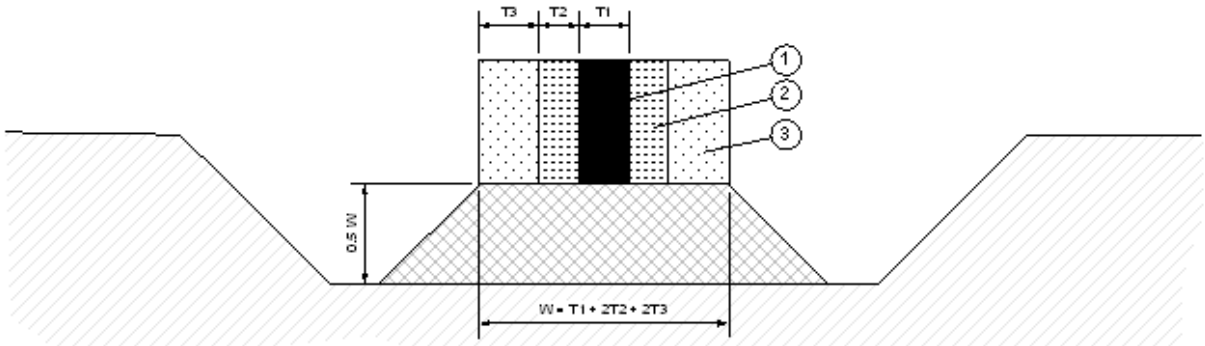


Figure 4-12 Plinth for ACRD

For preliminary designs the core thickness should be at least 0.7% of the head difference between the upstream and downstream sides of the core at that level subject to a minimum of 0.5 m. The connection tote plinth including the abutments should have a thickness of twice the core thickness.

The widths of the transitions, zones 2 and 3 in Figure 4-12 are typically 0.8 m and 1.2 m respectively.

4.6.3 ASPHALTIC CORE PLACEMENT

Placement and compaction of the asphaltic core is not much affected by climatic conditions. Rain rarely causes placement difficulties but extended sub-zero conditions will limit the construction season. A construction schedule should be developed which takes weather into account.

When estimating construction costs, hauling costs of bitumen should be considered.

4.6.4 MATERIALS FOR CORE AND FILTERS

Detail discussion and recommendations in material specifications and requirements for ACRDs should be consulted in ICOLD Bulletin 179 on Asphaltic Concrete Cores for Embankment Dams. Høeg (1993) gives details of asphaltic concrete mix design and properties as well as reference specifications.

Aggregate for the asphaltic concrete has to be obtained either from a quarry or a borrow pit in alluvium within reasonable distance from the dam. The aggregate may be all-crushed or partly crushed if the source is alluvial.

The bitumen content in the asphalt core will normally be between 6.5 to 7.5% by weight. Filler (0 - 0.063 mm) is required which may be obtained from the aggregate plant or other sources such as crushed limestone, Portland cement or fly-ash.

The supporting transition zone material should be produced from crushed rock, well graded, with d_{50} <10 mm and d_{15} <10 mm. The total fines content should not exceed 5%.

4.7 ASPHALT FACED ROCKFILL DAM, AFRD

4.7.1 DAM CROSS-SECTION

An asphalt faced rockfill dam is conceptually similar to a CFRD. The asphalt facing forms a continuous membrane on the upstream face without joints. The facing is joined to the foundation with a plinth in the same way as a CFRD. Asphalt facing is also used for smaller reservoirs where the whole reservoir including its bottom is thus sealed.

The asphalt facing can tolerate substantial deformations without cracking. A gradual deformation of 100 mm per metre can be allowed. This makes this dam type suitable for foundations which would deform to an extent that would distress the concrete slab of a CFRD.

Asphalt linings placed onto a stabilized surface (i.e. cement stabilization or concrete kerbs as used for CFRD) can have an inclination up to 1.25:1 (h:v) but a maximum slope of 1.6:1 is commonly adopted.

The highest constructed asphalt facing rockfill dam to date is around 100 m, which result in a slope length of 200 m, which can be constructed in one stage.

4.7.2 THE PLINTH

The edge of the bituminous lining at a dam face is a plinth or gallery at the dam toe and the lateral parts of the face. Along the crest there may be a concrete structure or the connection to a bituminous crest road. The plinth is also used for connecting the asphalt face with the foundation. Similar criteria as used for CFRD dams can be implemented for dimensioning and founding the plinth. The plinth also serves a grouting platform.

4.7.3 THE BITUMINOUS LINING

Bituminous lining works are normally only undertaken in warm and dry conditions. In order to achieve good quality, the dense layer should be placed when the air temperature is > 5°C. Placing works during light rain is allowable only for binder or drainage layers (Walo, 2014).

The minimum thickness of the binder layer should be 60 mm. Given that the binder layer is the base and foundation for the impermeable layer, the thickness could be increased to 80 – 100 mm in order to compensate a weak foundation.

Normally the dam body is constructed completely before the asphalt works starts. Therefore, only the settlements resulting from impounding and the long-term settlements have to be considered. Due to the bituminous facing showing a flexible and plastic behaviour, normal deformations or movements of the embankment should not influence the quality of the lining itself.

ACRD dams can also incorporate a parapet wall on the crest to reduce fill volumes. Parapet walls up to 5 m are often considered.

A mastic sealing coat is typically applied to the dense bituminous layer as a protective coating or sacrificial layer to protect the dense asphaltic layer against UV-radiation and creates an elastic film at the surface. Mastic consists of bitumen, filler and possibly sand and is spread in liquid form onto the dense layer (Walo, 2014).

Concrete structures like intakes or spillways are part of any impounding system and the bituminous lining will be watertight joint to these structures. Standard design solutions for all different type of joint construction are available.

A typical cross section of an AFRD is shown in

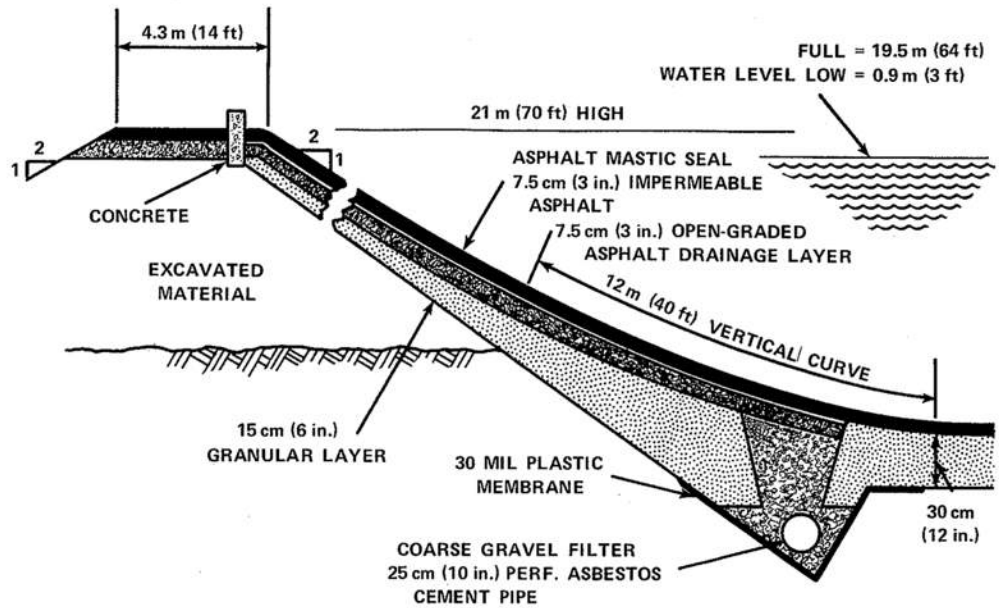


Figure 4-13.

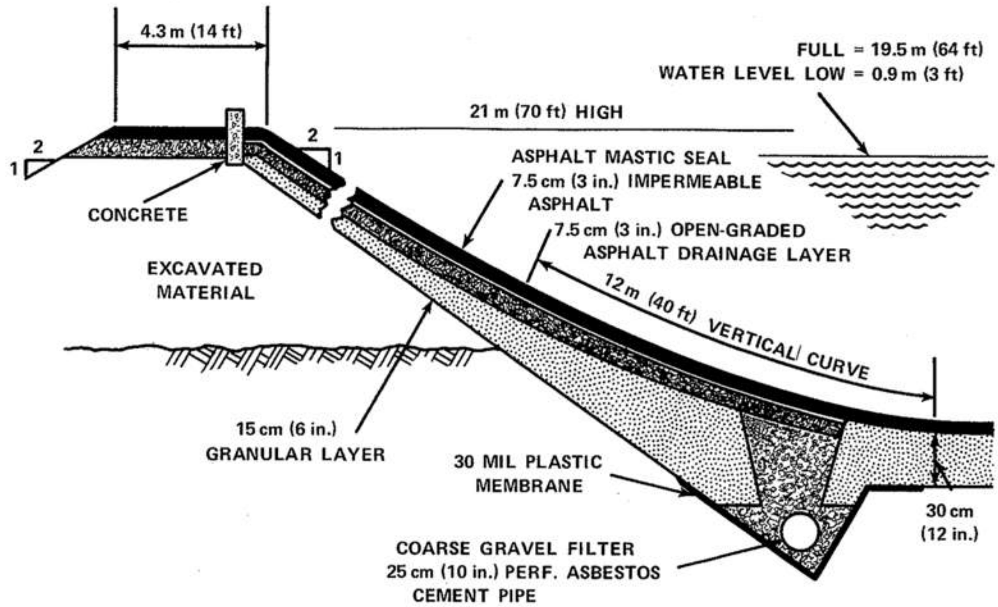


Figure 4-13 Cross-Section of an AFRD
Upper Reservoir, Seneca Power Project, USA. (Asphalt Institute, 1976)

During the dam selection phase, for initial cost estimating purposes, the different layers composing the asphalt membrane, drainage layer and support material can be considered as one layer. Reference unit prices for a "composite layer" can be sourced with specialized contractors in AFRD dams.

The construction of an AFRD is highly specialized and requires experienced contractors. This restricts the local availability of contractors to execute this works in many countries. This aspect should be considered during the dam selection stage.

4.8 GEOMEMBRANE FACE OR CORE DAMS

4.8.1 GENERAL CONSIDERATION

Membranes made from artificial materials can be used as the watertight element of a dam in certain circumstances. A common application is a membrane placed on the upstream face of an embankment dam. Membranes are also use for the repair of leaking embankment and concrete dams. Membranes are not required as an element of new concrete dams.

Membranes may be considered where natural material where either short in availability or missing or of inadequate quality.

There is extensive performance data for dams up to 30 m high. Typically, they are employed where the membrane can be accessed for inspection, repair or replacement by drawing down the reservoir.

An upstream membrane may be exposed or covered by a protective layer of concrete or granular material. There will be a drainage layer below the membrane which may be a discrete layer where the embankment is made from low permeability material (earth) or may be a transition to rockfill where the embankment is constructed from such material.

Exposed membranes have the advantage of being easily inspected and repaired and have a low installation cost. They are, however, susceptible to damage and covered membranes can be used as an alternative. The cover may be concrete or granular material. Covered membranes have a higher installation cost then exposed versions.

The use of such membranes is controversial. In some countries they are not allowed or used only for small dams. Any proposal for use of membranes in dams should conform to national regulations. Membranes should be avoided on dams classified as medium or high hazard where failure of the membrane could lead to significant financial losses or loss of life. Most membranes have been installed on dams that the less than 30 m high.

For all but the smallest dams there should be a second watertight element to retain the water in the event of damage to the membrane. In rockfill dams this will be a low permeability zone close to the membrane following the requirements for CFRD design. In earthfill dams a drainage zone is required to prevent seepage water entering the its downstream shoulder.

A few dams have been constructed with buried membranes located centrally in the dam. There is very little performance data available for this type of construction. Utmost caution should be used when considering such designs.

The effects of normal and rapid drawdown of the reservoir have to be considered in the design of the underlying filter or drain. To avoid disruption of the membrane, there must be no pore pressure head in this zone above the level of the reservoir.

Membrane manufacturers all have their particular methods of installing and anchoring membranes of which some are proprietary and patented. Where a membrane solution is being considered, the advice of a suitable manufacturer and installer should be sought and prices obtained.

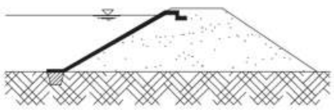
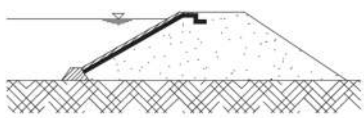
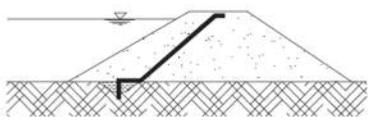
Figure	Total	Type of application Some typical examples
	47*	Upstream exposed geomembrane on fill dam: Arcizans (France), Cracow (Australia), Midtbotnvatn (Norway), Moravka (Czech Republic), Regulating Reservoir (USA), Sa Forada (Italy), Upper Pond Okinawa (Japan), Winscar (UK)
	106	Upstream covered geomembrane on fill dam: Bovilla (Albania), Contrada Sabetta (Italy), Jibiya (Nigeria), Middle Creek (USA), Rouchain (France), Symvoulos (Cyprus), Wenholthausen (Germany)
	20	Central geomembrane on fill dam: Atbashinsk (CSI), Fencheng (China), Hongya (China), Zushou (China), Goose Lake (USA)

Figure 4-14 Typical membrane installations (from Bull. 135)

4.8.2 TYPICAL CONSTRUCTION DETAILS

4.8.2.1 EMBANKMENT SLOPES

As a general guide the upstream face should be no steeper than 1.5:1 (h:v) to avoid complications in membrane installation. The slope adopted will depend on the nature of the embankment material with the following slopes being typical:

Clay	2.5:1 to 3.5:1
Sandy clay and silt	2:1 to 3:1
Sand and gravel	2:1 to 2.5:1
Rockfill	1/5:1 to 2:1

4.8.2.2 THE MEMBRANE SEALING SYSTEMS

Typical composition of membrane systems for an earthfill and rockfill dams are shown in Figure 4-15 and Figure 4-16.

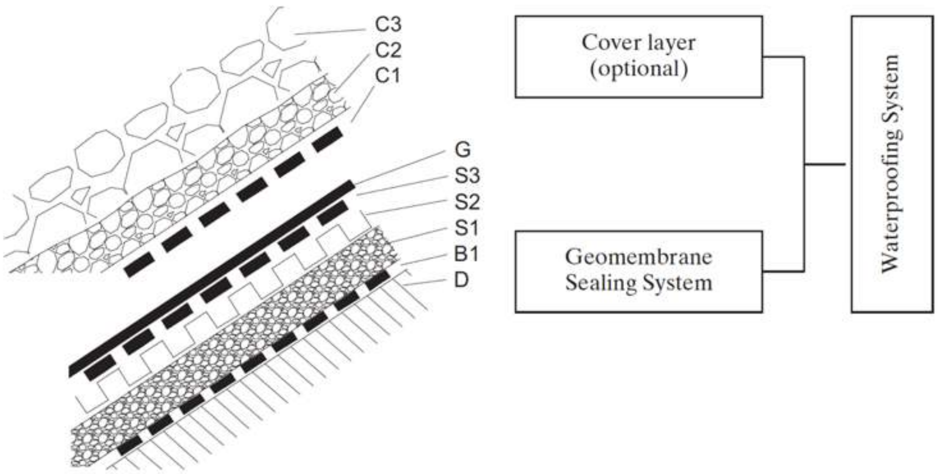
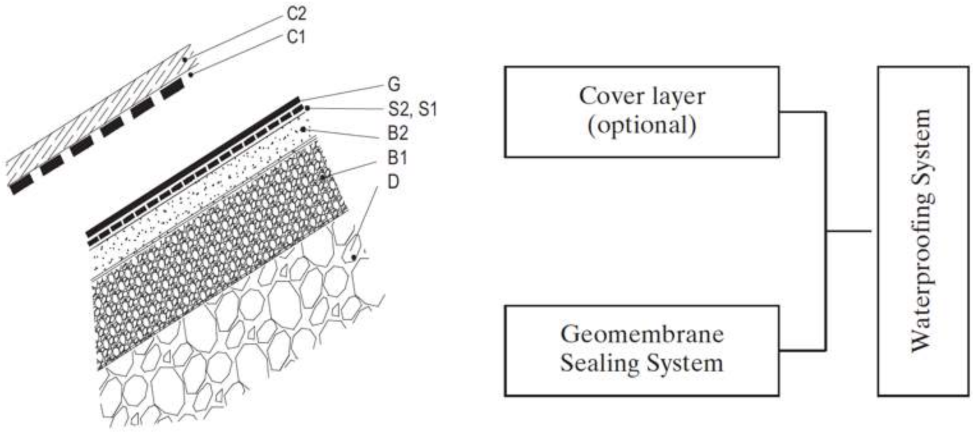


Fig. 48
New earthfill dam
D: dam body

Figure 4-15 Typical composition of membrane system for an earthfill dam
For legend see Figure 4-17 (from ICOLD Bull. 135)



C2: Concrete slabs
C1: Geotextile
G: Geomembrane
S2: Geotextile (optional)
S1: Asphalt concrete (optional)
B1, B2 : Granular transition layers
D : Dam body (Rockfill)

Figure 4-16 Typical composition of membrane system for a rockfill dam

For legend see Figure 4-17 (from ICOLD Bull. 135). Layers C1 and C2 are used only with covered membranes

Number	Function	Material
C3	Mechanical protection	Rip-rap, concrete
C2	Transition	Granular (0/25)
C1	Anti-puncturing	Thick geotextile
G	Watertightness	Geomembrane
S3	Anti-puncturing	Thick geotextile
S2	Drainage	Soil drain or Geospacer
S1	Drainage, support	Granular (0/25 mm)
B1	Filtration	Geotextile

Figure 4-17 Function and type of the various layers

From ICOLD Bull. 135

The membrane has to be protected from puncturing by a rough supporting layer with a geotextile.

For rockfill dams the supporting layer has to have a sufficiently low permeability to reduce seepage flows such that instability of the dam will not occur. With earthfill dams, a drain is required, either close to the membrane or elsewhere in the dam such that a severe leakage through the membrane will not give rise to high pore pressures within the embankment with attendant risk of failure.

4.8.2.3 ANCHORAGE AND PERIPHERAL JOINTS

Membranes require anchorage to prevent damage by wind or waves creasing uplift forces. This may be made with ballast or with linear anchorages running parallel to the slope (the fall line).

Membrane requires seals against a peripheral plinth or concrete cutoff wall. Foundation grouting will typically be made from the plinth. At the crest the membranes are commonly anchored in a trench.

Some non-impounding reservoirs are provided with a membrane covering part of or the entire floor.

4.9 ASPECTS COMMON TO EMBANKMENT DAMS

4.9.1 FREEBOARDS

Sections 2.6.2, 2.6.3 and 2.6.4 set out the principles of freeboard requirements and definitions. For embankment dams the freeboard must be such that the dam will not be overtopped in the event of the maximum design flood. (The only exceptions to this are very small dams with small flood discharges that can flow over the crest without damage, with erosion prevention being a grassed slope.)

The required freeboard will be estimated from the reservoir elevation associated with the maximum design flood with the addition of wave run-up and wind set-up. There may be further potentially governing design cases, being seismic seiches and landslide or rockfall-induced waves. The minimum freeboard above the maximum water level will depend on dam hazard classification and corresponding regulations. A minimum freeboard of 2 m is commonly adopted for large dams. Embankment dams will be subject to consolidation settlements. This settlement has to be estimated and the dam built correspondingly higher to ensure that the freeboard requirements are satisfied in the long term.

4.9.2 EARTHQUAKE SETTLEMENTS

Earthquakes can cause deformations and settlements in dams with a potential loss of freeboard. Potential deformations can be estimated using Sarma (1977, 1979) or Newmark (1965). The deformations depend on the height of the dam, the stiffness of the dam body and its foundation, the shear strength of the embankment materials and any dynamic pore pressures that might develop. For preliminary designs additional freeboards can be set using empirical data provided by Swaisgood, J. R. (2014) where

$$NCS = e^{(5.70\ PGA\ +\ 0.471\ MW\ -7.22)}$$

and is presented here as Figure 4-18 where the definitions of terms can be found.

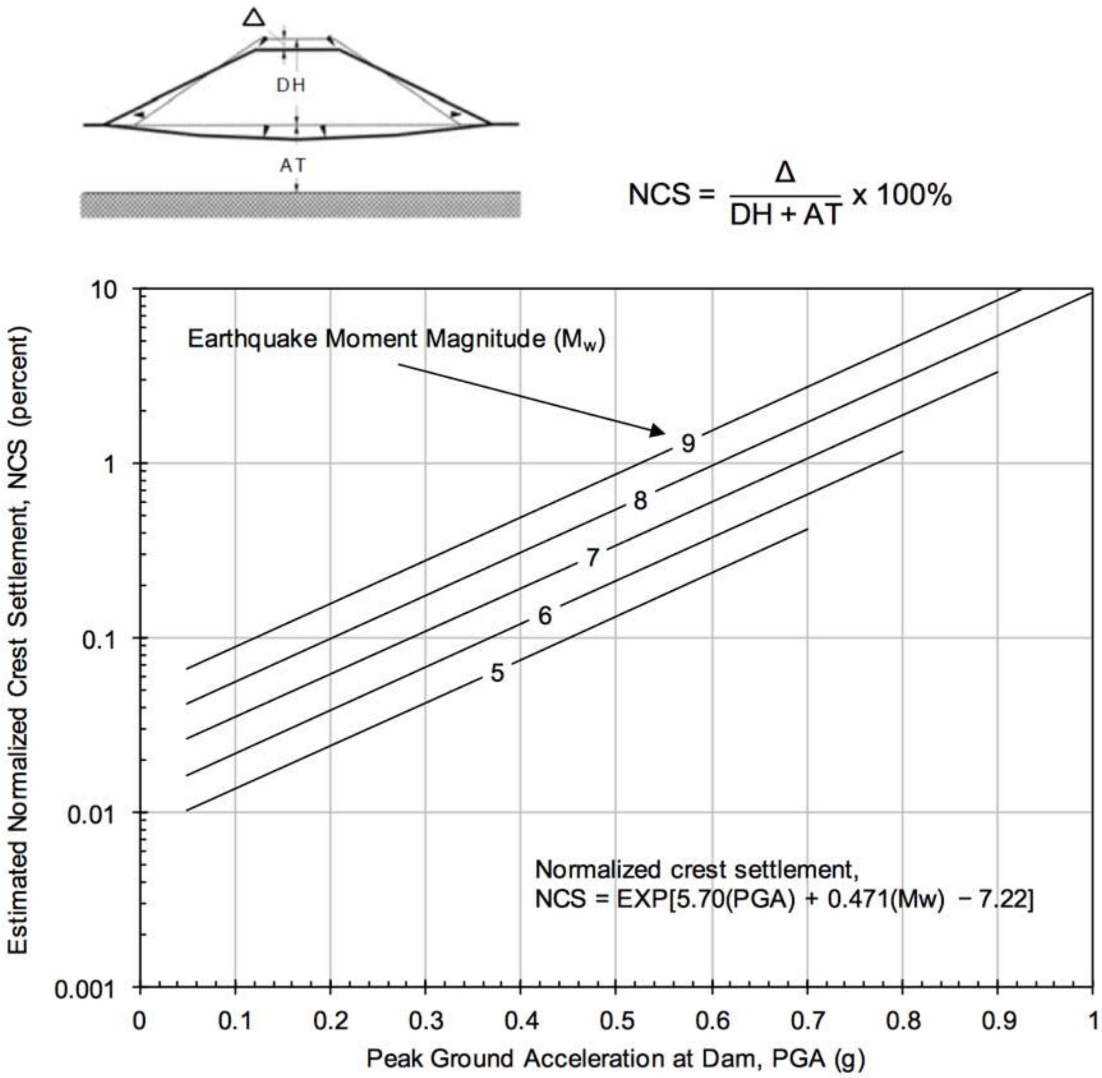


Figure 4-18: Chart for estimating earthquake-induced settlements
(from Swaisgood, J. R., 2014)

The freeboard so calculated is in addition to the allowance for consolidation settlement discussed in Section 4.9.3.

Many high rockfill dams have experienced high seismic loadings with satisfactory performance. In general, rockfill behaves very well under seismic conditions, because the seismic loading generates no pore pressures if the material is free draining. Also, free-draining rockfill is not susceptible to internal erosion or piping if the dam cracks.

Rockfill dams are also a suitable alternative in cases where active faults are detected or presumed at the dam foundation. In particular, central core rockfill dams with generous cores and filters could withstand substantial displacements without failing.

Upstream membranes (CFRD, AFRD) can be disrupted by earthquake motions but such disruption is mostly above the reservoir level at the time of the event. As well-designed dams of these types have permeabilities of fill material that increase towards the downstream, disruption of the membrane will not lead to failure. No particular measures need to be incorporated at the outline design stage.

4.9.3 CAMBER: ESTIMATING SETTLEMENTS

The dam will be subject to consolidation settlement. This has to be compensated for by increasing the crest and core elevation of the dam (camber) by an amount corresponding to this settlement, thus ensuring that the required freeboard is maintained in the long term. However, consolidation settlement of the fill may give rise to a 1% increase in dam volume which is not significant in the context of a preliminary design.

Settlement of foundation strata should be estimated using one-dimensional consolidation theory in the first instance and fill volumes adjusted as may be necessary.

4.9.4 MATERIALS

Much of the embankment can be constructed using material from excavations from the projects works including: spillway, intake works, diversion tunnels and even dam foundation. However, the production of the material from parts of the works may not coincide with the embankment placing requirements and temporary storage areas may be required, and additional costs may need to be considered for re-handling the material.

Ideally, free draining quarried rock should be used for the embankment rockfill to allow for an economic design. However, if additional zones (i.e. drains) are incorporated in the dam, even weathered rock can be used as rockfill. Most igneous and metamorphic rocks yield free draining rockfill. However, some metamorphic rocks (e.g. phylite, gneiss, schist and slate) may break after compaction, yielding a poor draining material, with reduced modulus of elasticity (Fell et al, 2005). Also most sedimentary rocks give the same result.

The rockfill embankment can also be zoned. The best quality rockfill (i.e. highest modulus of elasticity and strength) is typically placed in the upstream shell, especially in CFRDs, to reduce settlement upon impoundment that could affect the concrete face.

A very good source of rockfill is often gravel, exploited from alluvial deposits located in the riverbanks. If hauling distances are short, this is an ideal material for rockfill with high strength and modulus of elasticity (in excess of 300 MPa). This material is often exploited under the water table to wash out fine material. Alluvial deposits are typically the main material source of high quality sand and gravel required for construction of filter.

Rockfill behaviour and properties not only depend on their nature, but also on the placing procedures that govern their density and thus their strength and compressibility.

Specifications for compaction of various zones may be preliminarily selected according to engineering experience. Porosity or relative density, parameters of compaction of embankment materials shall be determined in detail in later design stages.

Gradation of the different rockfill zones is important, but it is typically specified in later design stages and not required for dam selection.

4.9.5 SPILLWAYS

Chapter 5 provides guidance on spillway design and location relative to the dam.

4.9.6 SLOPE PROTECTION

All central core dams (Earthfill, ECRD, ACRD) require protection of the upstream slope to prevent erosion by wave action. Protection is not required below the minimum operating level.

Riprap is the most common protection and the cost estimates may be based on its use. Whereas the final design of the protection will be determined by the fetch, the wind regime, embankment slope and density of the riprap rock, simple assumptions can be used in preliminary designs. Average (D_{50}) riprap sizes vary from 75 kg (0.3 m equi-dimensional stone) to 900 kg (0.7 m equi-dimensional stone). The riprap has to be placed on a bedding layer which is typically a coarse gravel some 0.4 to 0.6 m thick.

A 0.9 m thickness of riprap (measured normal to the slope) can be assumed but this may be reduced for very small dams.

USBR (2014) gives standards and methods for estimating riprap protection where a developed design is required.

4.9.7 EFFECT OF FOUNDATION STRENGTH ON EMBANKMENT SLOPES

With sufficient strength of the foundation, the rockfill properties with the loading conditions will dictate the external slopes of the embankment. Conversely, a weaker foundation will govern the same slopes. Considering the latter case, foundation strengths commonly increase with depth and with greater depth of excavation come steeper embankment slopes. There is thus a depth of excavation which is optimum and which should be estimated.

4.9.8 DETERMINATION OF EMBANKMENT SLOPES

Chapter 2 gives details of required investigations and design data as well as considerations of flood surcharges and freeboards.

The design of a rockfill dam involves addressing foundation conditions and embankment fill requirements:

- Foundation depths and strengths as discussed in Section 2.4. The foundation strength requirements will depend on the dam sub-type and the location within the dam.
- Estimates of foundation strengths (Section 2.4).
- A check that the foundation has adequate stiffness for the proposed sub-type
- Where rockfill properties or foundation strengths affect the slopes of the embankment, carry out stability analyses to determine these slopes
- If only curtain grouting in rock is being considered for any of the dam types under evaluation, then grouting need not be quantified as it will be essentially the same for all dam types. Should one of the dam options involve other types of cut-off, then the extent of grouting and its cost has to be included in the evaluation. Consolidation grouting should be included in the dam type comparison.

For dam type selection designs, the following minimum assessments and design calculations are recommended:

- Assessment of foundation strengths using the Hoek and Brown approach (Hoek & Brown, 1980) for heavily jointed rock, the Barton – Bandis method (Barton & Bandis, 1990) where joint shear strengths will dominate foundation strengths and reasonable estimates of shear strength for soils using empirical correlations.
- Adoption of rockfill strengths from Lepps (ICOLD Bulletin 92) and Marsal (1973). These studies have shown the variation of shear strength of rockfill with normal pressure.
- Estimates of pore pressures in the dam body and foundation
- Definition of important load cases (end of construction, normal operation, flood, earthquake)
- Stability analyses (limit equilibrium analyses) using circular and non-circular two-dimensional slip surfaces. This is normally made with commercial software.
- Adjustment of the slopes of the dam faces until acceptable factors of safety are obtained

4.9.9 FILTERS, DRAINS AND TRANSITIONS

The volumes of filters have to be estimated, but filter design is not required for dam type selection. Filter materials may be assumed to be manufactured with attendant costs. Filters may be manufactured to satisfy any grading criteria.

Filter thickness is typically defined by construction requirements of the placing equipment and not the theoretically small value required to prevent piping for a particular hydraulic gradient (ASCE, 1989). Vertical or steeply inclined filters are commonly 3 to 4 m wide and flat-lying filters may be 0.5 to 2 m thick depending on the size of the dam and the regularity of its foundation. Filters are usually considered downstream of the impervious element of the dam (concrete face, earth core, asphaltic core), but also in sectors of the foundation in contact with the rock shells that are highly weathered (i.e. shear zones, weathered dikes, among others) that could be piped into the rockfill.

Drainage layers are another important component of rockfill dams. The drainage layers are usually placed downstream of filters. A horizontal drainage blanket is commonly placed on the foundation and downstream shell to keep it drained and safely conduct any seepage to the downstream toe (ASCE, 1989). Relief wells are provided for dams with significant under-seepage to reduce uplift pressures, especially in cases with partial cut-offs and deep pervious deposits that are not excavated (ASCE, 1989).

ICOLD Bulletin 95: *Embankment Dams, Granular Filter and Drains* provides a review and recommendations for the design and construction of these dam components.

4.9.10 FOUNDATION REQUIREMENTS AND TREATMENT

Foundation treatment will be required to control seepage pressures and water volumes. With rock foundations this is typically a grout curtain at the location of the watertight element of the dam (under the centre of an earthfill core, along the line of the plinth for concrete or asphalt faced dams). Consolidation grouting will be required for the foundation of an earthfill core. The whole of its footprint is commonly subject to consolidation grouting with the purpose of increasing the foundation stiffness and reduction of permeability, thus as far as possible eliminating seepage paths close to the core. This applies to both alluvial and rock foundations. The depth of grout holes and their spacing will depend on the rock properties and may range in depth from 5 to 10 m with hole spacings of 2 to 7 m for rock ranging from heavily fractured to massive.

Particular note should be taken of soluble rocks and minerals in the dam foundation where in particular karstic voids can cause major problems. Such features are problematic for all dam types and should be discovered at the time of dam site selection. They may affect the technical feasibility for one or all dam types. The treatment of karst may entail cut-off blankets and walls, concrete infills and grouting. The cost of such works, to the extent that they may be different for different dam types, has to be included in the cost estimates for dam type selection.

In seismic areas soil foundations susceptible to liquefaction (i.e. silt or fine sands) must not be present in the foundation. Such soil deposits should be removed but compaction may be possible in some circumstances.

Some rock types such as porous sandstones cannot be easily grouted. Consideration should be given to alternatives such as clay blankets, positive cut-offs or drainage curtains.

When rockfill dams are founded on permeable soil deposits that are susceptible to erosion and seepage, a cut-off wall may be required. The type and depth of the cut-off wall depends on the nature of the material being treated, susceptibility of the foundation to internal erosion, and tolerance for seepage. Low permeability soil blankets may be used on alluvial foundation strata to lengthen seepage paths and thus reduce hydrostatic pressures in the dam foundation and seepage losses. Options that may be applicable are:

- Slurry trenches

- Soil-bentonite walls
- Plastic concrete walls
- Cast-in-place concrete walls
- Sheet piling
- Jet grouting
- Upstream impervious blankets

Selection among these options depends on particular conditions of the project. Grouting works are typically required even for rock foundations. The grout curtain is generally extended up to one third to two thirds of the local hydraulic head (⅓H to ⅔H). Additional consolidation grouting is typically executed around the cut-off.

Selection among these options depends on particular conditions of the project. Grouting works are typically required even for rock foundations. The grout curtain is generally extended up to one third to two thirds of the local hydraulic head (⅓H to ⅔H). Additional consolidation grouting is typically executed around the cut-off.

If the foundation of the dam site features karstic limestone formations, consideration should be given to options that allow for substantial grouting operations outside of the critical path. CFRDs are suitable in these cases, because the concrete plinth can provide a platform for grouting operations and a grouting gallery can be provided for remedial grouting at a later stage. Grouting galleries have been provided in the foundations of large ECRDs.

4.9.11 EFFECTS OF CLIMATE

Frost will preclude placement of fill materials including rockfill where water is used in the compaction process.

Rainfall will hamper the placement of earthfill as its water content has to be controlled for effective compaction. Placements might have to be scheduled to parts of the day or seasons with little or no rainfall. Covers can be used to protect placed material or to allow placement to continue under tenting.

The use of silty sand cores that are compacted to a high density may not be sensitive to rainfall as they do not soften, e.g. San Roque dam in the Philippines.

A common problem in the construction of CFRDs is the erosion of the support zone of the concrete face by heavy rainfall. To avoid this, an extruded concrete "curb" is typically used to protect the support zone.

The installation of an asphalt liner requires certain climatic conditions:

- Drainage or binder layer: Temperature > 0 °C and low humidity
- Dense asphalt layer: Temperature > 5 °C and dry conditions

As soon as the asphalt material has been placed and compacted, the climate will have no influence any more. No special measures are necessary for after-treatment. The lining works can than continue as soon the climate conditions have improved – either within the next day or after the winter season. A protection of the installed asphalt is normally not needed.

4.9.12 RISKS

The particular conditions of a project lead to specific risks. Hence, the standardization of risks should be avoided and a specific risk management plan should be adopted for every project. However, this section identifies certain risks that are generally applicable and should be considered for most projects. Risks associated with the selection of the dam type are highlighted. These risks are identified for each of the project main stages: design: construction, and operation.

4.9.12.1 DESIGN RISKS

As in any other project, a good and rigorous design is crucial for the functionality and durability of the dam. Only experience and sound knowledge of the local conditions can eliminate most risks. Some of the potential risks encountered during the design stage include:

- Inappropriate selection of dam type due to a bias from the designer or owner, generated by a limited experience with certain dam types.
- Inadequate understanding of the geological and geotechnical conditions of the dam site that inhibit the application of proper remedial measures or treatment.
- Lack of understanding of the possible failure modes.
- Deficient multidisciplinary review of the design that leads to contradictions in the design.
- Inaccurate understanding of environmental restrictions imposed to the design.
- Insufficient materials for the rockfill identified.
- Insufficient geotechnical investigations that do not allow a proper characterization of the foundation and fill materials. This could even lead to the wrong selection of dam type.
- Failure to predefine certain warning levels for the instrumentation results.

4.9.12.2 CONSTRUCTION RISKS

Some of the main risks during the construction stage are presented below:

- Overtopping of the cofferdams that leads to destruction of the works and delays.
- Inadequate QA/QC during construction.
- Lack of an independent party performing QA/QC.
- Identification of geotechnical conditions that lead to differing site conditions. This could lead to cost increases and delays.
- Damage of the instrumentation as it is being installed.
- Inappropriate coordination of the construction fronts that generates unavailability of materials for the fill.
- Inadequate compaction of the rockfill, leading to increased settlement.
- Unrealistic original schedule.
- Interference between contractors.
- Grey areas of interface between contractors and suppliers.

4.9.12.3 OPERATION

Some of the main risks during the operational stage are presented below:

- Failure to operate flood control gates
- Failure to monitor the instrumentation on a regular basis.
- Incapacity to react to instrumentation readings that indicate that remedial measures should be taken.
- Seismic event.
- Inadequate maintenance.
- Untrained staff.

4.9.13 RATE OF CONSTRUCTION, SCHEDULE

Section 3.8 gives an outline of the major elements of dam construction that need to be considered in developing a schedule.

Typically, the initial activity on the construction's critical path of an embankment dam is the construction of the diversion. Sometimes foundation treatment and fill placement can begin before the diversion is completed. This usually depends on environmental restrictions and valley shape.

Rainfall can adversely affect the rate of construction of embankment dams. Earthfill cores and shells have to be compacted at particular moisture contents and rainfall can make the fill uncompactable. Following rainfall, the affected fill may have to be removed or allowed to dry naturally or aided by scarification. Some earthfill core materials can be compacted to a very high density and are essentially impervious and require little rectification following rain. Covers can be used to protect vulnerable materials.

The intensity and pattern of rainfall will affect the construction schedule.

Rockfill placements are not normally affected by rain.

Freezing temperatures will mostly preclude placement of embankment fill.

Once fill placement begins, typically the construction schedule is controlled by the main rockfill placement rate. The construction of the impervious element (i.e. concrete face, earth core or asphalt core) can usually proceed in parallel with the main fill placement.

With the development of modern construction equipment, very high placement rates can be reached if the construction is properly planned. Table 4-1 shows average and maximum placement rates for some rockfill dams.

Construction times are largely governed by the resources applied but there are limits set by the size of the dam and the resulting working space. High placement rates required much expensive earthmoving machinery and this cost should be warranted by the benefit of early completion. In many projects the desired time for completion is given by the time water is required for power generation or other use.

Table 4-1 Average and maximum rockfill placement rates for several dams

				Fill placement (m^3/day)	
Country	Project	Height (m)	Volume (Mm^3)	Average	Max
Brazil	Barra Grande	185	11.5	23,300	
Brazil	Campos Novos	210	12.1	26,700	26,700
Brazil	Irape	208	10.3	26,000	30,700
México	Aguamilpa	187	13.0	15,000	
Iceland	Karahnjukar	200	8.5	20,000	
México	El Cajón	188	11.0	14,300	14,300
México	Yesca	220	12.0	12,600	22,200
Colombia	Ituango	225	20.1	12,650	
Colombia	Sogamoso	190	8.2	9,000	29,000
Colombia	Quimbo	151	7.6	12,900	
Philippines	San Roque	204	40		74,000

Table 4.2 shows average and maximum placement rates for several earth embankments.

Table 4-2 Average and maximum placement rates for several earthfill dams

				Fill placement ($1000\ m^3/day$)	
Country	Project	Height (m)	Volume (Mm^3)	Average	Max
Honduras	La Vegona	42	0.6	150	240
Colombia	San Rafael	60	2.8	192	210
Colombia	Cantarrana	36	1.3	91	

4.10 COST-DRIVING ELEMENTS

The main cost driving elements are here defined as those contributing 5% or more to the cost of the dam. The cost of the spillway and diversion is treated separately as the contribution to the total cost can vary considerably. The main cost elements for the embankment are:

- Earthfill and rockfill
- Foundation excavation and surface treatment
- Impervious element
- Grouting or other cut-off (where it is considered in the comparison of dam types)

Usually the main cost-driving item in a rockfill dam is the rockfill itself. That is why special attention is typically given to procuring local materials with short hauling distances. Typically the rockfill dam type that is able to use the most local materials results in the cheapest.

Depending on the foundation characteristics, the weight of the foundation treatment and excavations varies. Typically it represents a medium weight in the budget, but certain foundation conditions (i.e. karst, liquefiable materials, among others) could lead to substantial cost increases and the major driver in the budget.

Another major element in the cost of a rockfill dam is the spillway. Even though, the materials excavated is typically used in the dam, the excavation stabilization and concrete and steel gates (hydro-mechanical elements) tend to have major weight in the budget.

The diversion works are typically at the top of the budget weight as well.

Typically the impermeable element (i.e. concrete face, earth core, asphalt liner, asphalt core) also has significant weight on the cost the project.

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5 SPILLWAYS

5.1 GENERAL/INTRODUCTION

Spillway facilities are designed to convey flood discharges through a dam without endangering the safety of the dam and associated structures. There is a wide variety of spillway types and layouts which need to be compatible with the type of dam and site-specific constraints. All spillways need to operate with high reliability during flood events and control flood discharge without compromising the safety of the dam.

Most embankment dams are vulnerable to sustained overtopping flow as embankment materials are susceptible to erosion. Floods which exceed the spillway capacity can result in dam overtopping and dam failure.

The importance of spillway facilities to a dam is a function of the magnitude of the flood hazard, the frequency which the spillway is operated and the requirements for energy dissipation. In some circumstances the nature of the topography, geology and hydrology at a particular dam site may rule out certain dam types as described in Section 5.8.2.

Typically dam construction requires river diversion works. Control of floods during dam construction can also influence the dam type as outlined in Chapter 6.

Outlet works and their influence on dam type is covered in Section 5.6. Outlet works (e.g. bottom outlets, sluices) are generally not used to convey flood discharges and instead enable draw-off under normal operating conditions or for reservoir drawdown in response to a dam safety emergency.

5.2 ICOLD BULLETINS

ICOLD has produced various bulletins related to the design and operation of spillways. As mentioned, the present bulletin does not intend to supersede these publications, although it is recognised that certain subjects like energy dissipation deserved an ampler treatment. In any case, the aggregated of the following bulletins comprise a significant volume of valuable technical information on the hydraulic design of spillways:

Table 5-1 ICOLD Bulletins dealing with the design of spillways

Nº	Bulletin Title	Year
49a	Operation of Hydraulic Structures of Dams	1986
58	Spillway for Dams	1987
81	Spillways – Shock Waves and Air Entrainment	1992
82	Selection of The Design Flood – Current Methods	1992
108	Cost of Flood Controls in Dams	1997
125	Dams and Floods – Guidelines and Case Histories	2003
130	Risk Assessment in Dam Safety Management	2005
142	Safe Passage of Extreme Floods	2012
156	Integrated Flood Risk Management	2014

5.3 ECONOMICS, RISK AND SAFETY ISSUES IN SPILLWAY DESIGN

Spillways are the main safety-assurance feature of any type of dam. For this reason, direct cost reduction attempts in these structures will commonly mean increases in the risk of lowering the safety of the overall dam project. However, in the formulation of the overall dam project layout the analysis of different types of spillway alternatives, for the same safety criteria, may have a significant impact on the overall cost of the project. In addition, different spillway configuration, such as a gated structure versus combination of gated and ungated spillways, for example, may lead as well to important cost savings.

5.4 SELECTING SPILLWAY FLOOD CAPACITY

Section 2.7 gives requirements for determining the design and check floods which are the basis for selecting flood discharge capacity.

Using the design flood, the procedure for selecting the spillway design capacity has the following broad steps:

- Make a first estimate of required overflow length for a free overflow control structure
- Route the design flood through the reservoir
- Check that the unit discharge is acceptable
- Check that the width of the spillway can be physically accommodated and is adequate for the provided freeboard

As outlined in the next paragraphs, the spillway and outlets may consist of a range of facilities, sometimes operating at different water levels.

5.5 SPILLWAY CLASSIFICATION

ICOLD Bulletin 58 classifies spillway facilities as either (i) surface spillways draw off surface flow with only a slight rise in water level, or (ii) submerged spillways set well below full supply level. Surface spillways can be further subdivided into gated and uncontrolled spillway types. Submerged or orifice spillways are sometimes further classified as mid-level and bottom outlets.

The USBR (2014) uses the functional classification of spillways as outlined in the following, depending on the frequency of use;

(a) Service Spillway

A service spillway provides continuous or frequent regulated (controlled) or unregulated (uncontrolled) releases from a reservoir without significant damage to the dam, dike, or appurtenant structures due to releases up to and including the maximum design discharge.

(b) Auxiliary Spillway

The discharge facilities may comprise a single spillway designed for the full range of flood discharges. In other cases it may be optimal to provide a service spillway for frequent floods and an auxiliary spillway which comes into operation for extreme floods. The latter will normally have a free overflow made of concrete and a discharge channel which may be lined or unlined depending on the flood return period used for its design and potential damage upon discharge. The auxiliary spillway may contain a fuse-plug. During operation there could be some degree of structural damage or erosion to the auxiliary spillway due to releases up to and including the maximum design discharge.

(c) Emergency Spillways

An emergency spillway is designed to provide additional protection against overtopping of a dam and/or dike and is intended for use under extreme conditions such as miss-operation or malfunction of the service spillway or other emergency conditions or during very large, remote floods (such as the PMF). As with auxiliary spillways, some degree of structural damage and/or erosion would be expected due to releases up to and including the maximum design discharge.

5.6 SPILLWAY COMPONENTS AND TYPES

5.6.1 COMPONENTS

Selection of an appropriate spillway type and layout depends on site specific requirements of the dam site and can influence the selection of the overall dam type as outlined Section 5.8.3.

Spillways can be composed of a variety of structures, but all generally have the following four components: an inlet or approach channel, a control or regulation structure, a conveyance structure, and an outlet (including energy dissipaters) as shown in Figure 5-1.

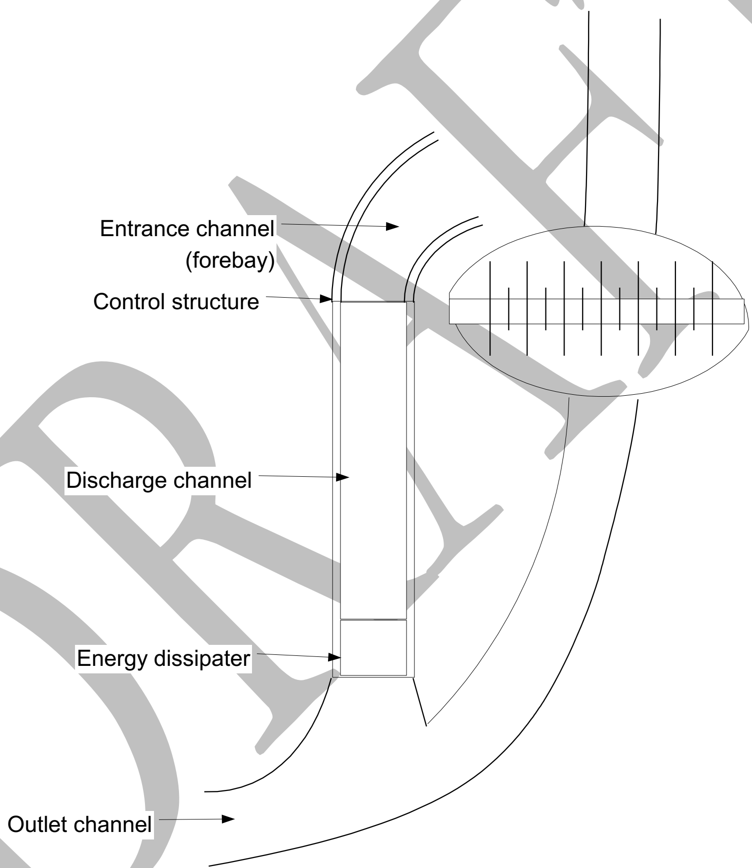

Figure 5-1 Spillway components
(after Shand 2009)

5.6.2 SPILLWAY COMPONENT TYPES

Figure 5-2 summarises common spillway components and most spillways will consist of a combination of these components. However, the selection of the appropriate spillway components requires a site-specific assessment and needs to take into consideration the dam type and other factors outlined in Section 5.8.3.

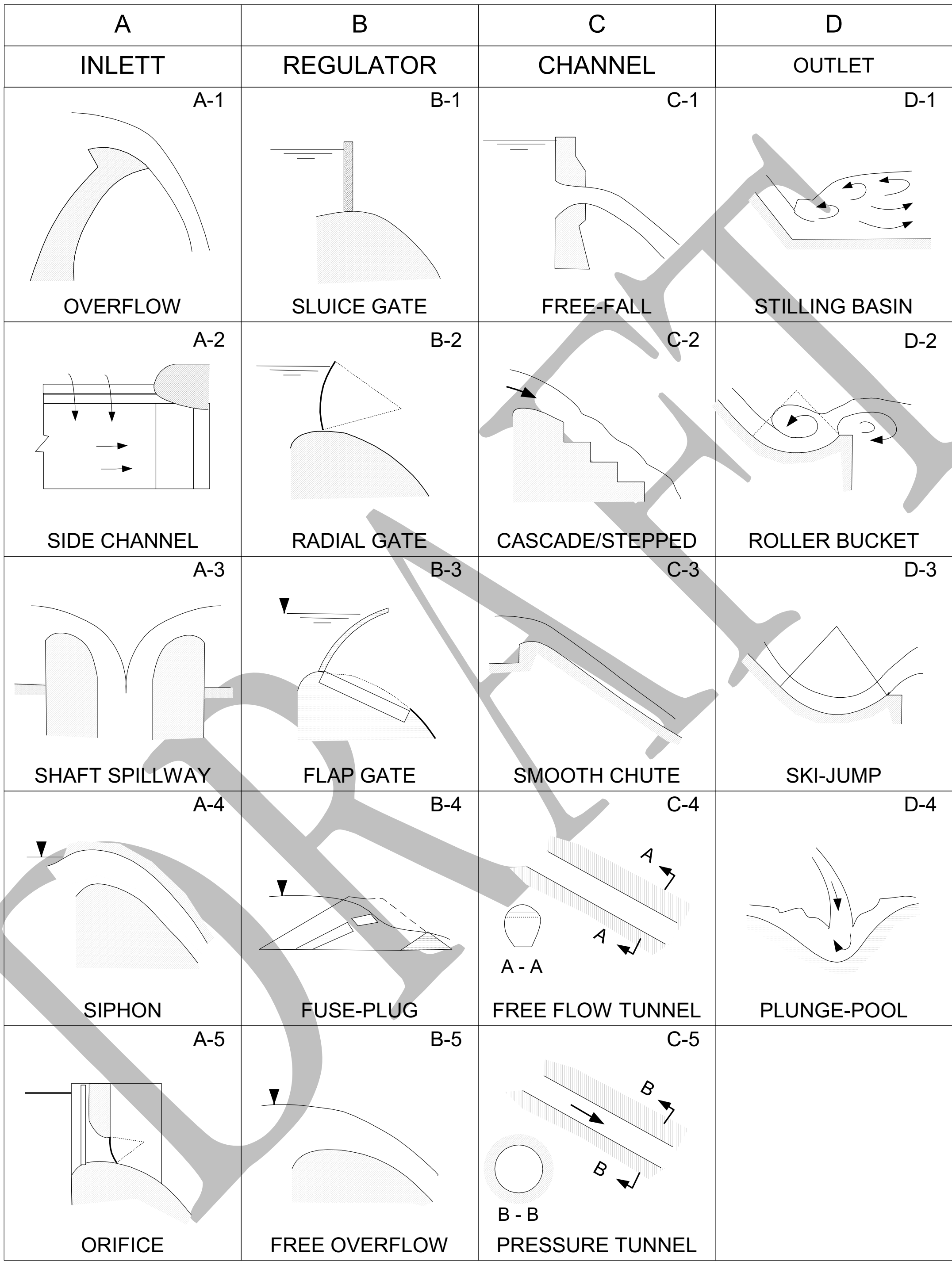

Figure 5-2 Common spillway components
(after Vischer, 1998)

5.6.3 DESIGN CONSIDERATIONS

In the more usual arrangements of spillways the flow in each of these regions would be as follows:

- In the approach channel, the flow velocity is kept subcritical and as low as possible to minimise hydraulic losses in the approach to the crest structure
- At the crest structure the flow accelerates through critical depth, regulating the flow whether gated or ungated
- The channel conveys the flow back to the river, the flow in the channel downstream of the crest will generally be supercritical and be designed to ensure that the flow over the crest structure is not drowned out by the backwater from the flow in the channel or chute
- The outlet structure provides for the safe return of the water back into the stream bed, transitioning the flow from supercritical flow in the channel to the appropriate conditions in the stream downstream and will generally be designed to minimise erosion where the flow enters the stream.

While the forgoing discussion implies that control should be at the crest, there are forms of tunnel spillways where the control for low depths of overtopping is retained at the crest, but as the flow increases, the crest can drown out as control transfers to the tunnel section of the spillway.

One advantage of the use of surface spillways is that the flow increases in proportion to the 1.5 power of the head over the crest, whereas for tunnel and orifice spillways the flow is less sensitive to changes in head over the inlet. Where such spillways are used, consideration should be given to the use of secondary, auxiliary or fuse-plug spillways.

5.6.4 STEPPED SPILLWAY CHUTES

A further spillway type has a stepped chute, commonly with an uncontrolled overflow, and a small stilling basin. These may be stepped with nappe flow where the water impinges on the step treads, or designed for skimming flow where the water flows down the slope as a coherent stream with energy being lost in vortices that form between the step noses. Thus, energy is dissipated on the chute and a small stilling basin dissipates the residual energy. Skimming flow stepped chutes are most commonly employed. Frizell (2006) and Frizell & Frizell (2015) provide guidance on stepped spillway design.

For preliminary designs the following may be considered:

- Unit discharges of up to 25 $m^3/s/m$ may be considered for regular floods while extreme flood discharges might be up to 40 or 60 $m^3/s/m$
- With flow velocities exceeding 14 m/s, air entrainment should be provided.
- Step height is not critical but should not be smaller than one-third of the critical depth. With RCC construction, the step height will be the same as the RCC lift thickness or a multiple thereof.
- The edge of the steps should coincide with the ogee profile and not protrude beyond it. The first steps may have to be 0.15 m high and increase in increments until the uniform slope of the spillway chute is reached.
- The spillway steps may be formed in GE-RCC or CVC without reinforcement.
- There are a range of formulae for estimating when skimming flow will occur and these should be used for analysis, however as a general guide, skimming flow will occur when the critical depth over the spillway exceeds 80% of the step height.
- The onset of uniform flow, the point at which the energy of the flow becomes constant, can be estimated from $H/d_c = 15$ to 20 where H is the vertical distance below reservoir and tailwater level and d_c is the critical flow depth.
- The residual energy is dissipated in a stilling basin. Table 5-2 gives an indication of the proportion of the total energy dissipated on the chute.
- Chutes that are shorter than $H/d_c = 15$ will not have developed uniform flow and the above indication of residual energy will give an underestimate. A more detailed study will be required to estimate the residual energy and the required stilling basin size.

For stepped spillways the equivalent clear water depth and equivalent terminal velocity should be determined at the chute toe upon the base of the residual energy (Boes and Hager, 2003b).

Table 5-2 Energy dissipated for a range of step heights, unit discharges and heads
(from ICOLD Bull. 172)

Gravity dams

	Step height, m	0.9			1.2			1.5		
	Unit flow, $m^3/s/m$	5	10	20	5	10	20	5	10	20
Head, m	30	68%	48%	32%	68%	49%	32%	69%	49%	33%
	50	78%	67%	48%	78%	68%	49%	79%	69%	50%
	100	87%	81%	72%	88%	81%	72%	88%	82%	72%
	150	91%	86%	79%	92%	87%	79%	92%	87%	80%

Embankment dams Step height = 0.3 m

	Embankment slope	3H/1V			2H/1V			3H/1V	
	Unit flow, $m^3/s/m$	1	3	5	1	3	5	0.2	0.3
Head, m	15	91%	81%	65%	86%	74%	55%	90%	89%
	30	95%	90%	86%	93%	85%	79%	91%	91%
	50	97%	94%	81%	96%	90%	86%	92%	92%
	Method	Boes & Hager (2003)			Boes & Hager (2003)			André (2004)	

For small to moderate unit discharges, stepped spillways offer considerable economy in construction costs as the high-quality concrete of a smooth chute is absent and the stilling basin is small. Ski-jumps with plunge-pools can also be used for energy dissipation of these strongly aerated follows.

5.7 ENERGY DISSIPATION

Appropriate energy dissipation is critical for all spillways and must bring the flow into the normal regime of the downstream watercourse without endangering the safety of the dam. A wide range of energy dissipation structures have been developed to suit different conditions (Figure 5-2). The selection and design of an appropriate energy dissipation structure requires consideration of many factors including hydraulic considerations, topography, geology, dam type, frequency of usage and layout of other associated structures.

- Stilling basins are the most common type of energy dissipater for spillways and outlets but are not recommended for dams with large spillway jet velocities because of concerns with cavitation damage, large spray production, unstable flow and tailwater wave generation (Hager et al., 2004).

- Roller buckets require substantially higher tailwater levels than conventional stilling basins and in the case of gated spillways require symmetrical gate operation. Energy dissipation is provided primarily by two rollers: a counter-clockwise roller near the water surface and above the roller bucket and a second roller on the river channel bottom downstream of the bucket. The bed of the river channel downstream of roller bucket needs to either be composed of rock resistant to erosion or provided with heavy protection against erosion.
- Flip bucket, ski-jumps and free-falling jets force the water jet some distance downstream of the dam so that scour caused by the jet does not endanger the dam. Flip bucket or ski jump energy dissipation are often used at high overflow dams where spray from the jet can be tolerated and erosion of the plunging jet can be controlled. Flip buckets and ski jumps are typically combined with plunge pools which dissipate hydraulic energy from the plunging jet and controls the progression of scour that could undermine the dam, its abutments or associated structures. Experience in a number of dams internationally has shown the need for plunge pool protection systems to prevent scour from spillway jets impinging onto natural rock. The deep scour pool downstream of Kariba Dam has been well documented in the literature.
- Particular precautions have to be taken for arch dams because deep scour can affect the stability of the dam foundations. Dams may be required downstream of the main dam to create a plunge pool (tailwater dams).

5.8 FACTORS INFLUENCING SPILLWAY SELECTION

5.8.1 BACKGROUND

ICOLD Bulletins 142, 125 and 58 list the following factors that should be considered in the spillway selection process:

a) Flood hydrology (reservoir outflow and surcharge to accommodate the design flood)
b) Dam type
c) Topography and geology
d) Frequency and duration of operation
e) Water control management objectives
f) Reliability of operation
g) Dam safety
h) Seismicity
i) Presence of debris or ice
j) Operation and maintenance requirements

Items a, b and c are the more important factors in the context of this bulletin.

The cited bulletins provide further discussion on the factors listed above.

5.8.2 SPILLWAYS FOR EMBANKMENT DAMS

Embankment dam materials (earthfill and rockfill) are susceptible to erosion due to overtopping and require spillway structures outside the dam body (Figure 5-3) or might discharge into an adjacent valley.

The spillway is most commonly constructed on one of the abutments of the dam. The definition of the most suitable valley side depends on the location of other structures (i.e. diversion tunnels, intake and bottom discharge), the geology and topography. It is also dependant on the height of the excavated cut face above the spillway and its stability and cost. If the dam is located on a river meander, the spillway is often located on the margin coinciding with the direction of the river downstream to facilitate discharging water back to the natural river.

Spillway chutes or side channels adjacent to the body of an embankment have to have high sidewalls to ensure that overtopping cannot occur even under extreme flow conditions, noting that bulking due to aeration will increase the flow depth. They may also involve high cuts into the abutment which will likely require considerable rock/soil stabilisation and drainage works. The discharge control structure may be a free overflow or gated. Tunnel spillways and culverts through or under the body of an embankment dam are strongly discouraged.

The treatment of the chute can have a significant bearing on the overall cost of a spillway. Approach channels are characterised by low velocity and the cost will be dominated by the cost of excavation. The crest structure will normally involve significant concrete works and possibly gated structures.

The chute that conveys the floodwater back to the river may be lined are may be unlined if hard rock that is not susceptible to erosion is present. If an unlined chute is being considered, the erodibility of the rock should be assessed (Annandale 2006). A lined chute should be founded on rock and inferior material is commonly replaced with concrete, typically as RCC on larger projects. Stepped chutes are commonly employed as described in 5.6.4

If it is not feasible to place the spillway outside the body of an embankment dam the spillway can be constructed in a concrete gravity structure incorporated in the earth fill or rockfill embankment, forming a composite dam structure. If the concrete gravity structure foundation is alluvial material and not rock, this solution will limit the height of dam that can be built with this arrangement.

Low embankments can be protected to a support overflow for short periods and small velocities by grass covers, geotextiles, reinforced rockfill, stones or concrete blocks and RCC or soil cement (ICOLD Bulletin 142, CIRIA 116). However, the depths of safe overtopping are quite limited and it is more usual to design the embankment and spillway crest with some nominated freeboard above the maximum flood level, that is without deliberately allowing flow over the crest of the embankment.

Figure 5-3: Embankment dam with spillway in adjacent valley

The construction of a spillway over an embankment dam, either an earthfill or a rockfill dam has been done in a number of cases but requires careful design and detailing. It is best avoided when considering preliminary alternatives and layouts. Settlement of the fill can cause cracking of the spillway structure and subsequent leakage can lead to a failure of the embankment. Poor detailing around the crest and training wall has been responsible for a number of piping failures in spillways abutting embankment dams.

Material originating from the excavations for the spillway is often the main rockfill quarry for the dam. This reduces hauling distances. In order to minimize doubling handling of excavated materials to place them on the dam fill, the construction schedule of the spillway excavations and the dam placing need to be coordinated. In cases where the top level of the excavation is constituted by fine quaternary soil deposits, these materials are often used for the downstream shell of the dam.

When excavations for the spillway are too large or show potential stability problems, tunnel spillways are often considered. The discharge capacity from bottom discharges is often used to complement the overall capacity of the system. Fuse plugs are sometimes used as well to complement spilling capacity for long return period events.

Given that rockfill dams are not resistant to overtopping, large flow spillways over the dam are usually not considered for high dams. However, for smaller structures, lined sections over the dam body and gabions have been considered. Special precautions should be taken in case these alternatives are considered.

A guide to the preliminary choice of dissipater is given by Mason (1982)

5.8.3 SPILLWAYS FOR CONCRETE DAMS

All or part of a spillway can be built into a concrete dam provided sufficient energy dissipation is provided. A typical arrangement comprises a control structure (gated or uncontrolled), a chute that conveys the water down the face of the dam with either ski-jump followed by a plunge-pool or a stilling basin.

If overtopping is permitted the dam abutments may need protection against scour or overtopping can be restricted to the river section by walls.

Spillways types often incorporated in arch dams are free flow orifices or an uncontrolled crest overflow with appropriate downstream energy dissipation.

Concrete dam types which can withstand overtopping are more suitable where upstream hazards exist such as landslide induced reservoir seiches, landslide dam failure floods or glacier lake outburst floods (GLOF).

5.8.4 GATED OR UNGATED SPILLWAYS

Ungated spillways are generally preferable due to their operational reliability and are considered the safest type of spillway (ICOLD Bulletins 142, 125, 82). Such spillways are less likely to block with debris and are generally more reliable because they do not depend on mechanical equipment or operational rules that can malfunction or be misused (ICOLD Bulletin 142). Ungated spillways will involve a higher dam height than gated spillways for a given full storage level and given maximum inflow rate. Labyrinth and piano key weirs can be used to increase discharge capacity for a given spillway channel width. However, the additional flood storage available when using an ungated spillway will attenuate the flood peak and can actually reduce the combined costs of the dam and the spillway. Optimisation of the spillway and dam body configuration is an important process and can result in significant savings to project costs.

Gated spillways can provide operational advantages in terms of elevated reservoir levels but have many factors that can reduce their reliability during flood conditions as outlined in ICOLD Bulletin 142, 125 and 82. The most important factor to be considered is risk of failure to open gates due to poor maintenance, operator errors, power failure and no operational standby power source or other causes. Specific project considerations dictate whether selection of a gated spillway is appropriate, and may include:

- seismicity of project site;
- manning or lack of it at the dam site;
- difficult to access dam site for emergency operation;
- possibility of spillway blockage from floating debris;
- insufficient capacity for gate maintenance, control and back-up systems;
- potential for gate operation to adversely impact on downstream population, property and infrastructure; and

- operation in very cold climate.
- The many factors that can reduce their reliability during flood conditions as outlined in ICOLD Bulletin 142, 125 and 82.

Ungated fixed crest spillways begin to operate once the reservoir rises above the fixed crest, as the water level rises the inflow flood is attenuated through the flood storage so that the outflow is always less than the inflow. Owners cannot be judged to have increased the flood hazard downstream. The opposite is true of gated or regulated structure and there have been a number of cases of gate operation causing damaging floods.

Gated spillways can cause a dam to fail due to a failure of the gates to operate or to operate as designed. The converse is also true, if the gates operate too early or open due to a malfunction in the control systems, this can create a flood wave downstream when one would not be expected. Rubber dams likewise have experienced bursting and uncontrolled reservoir release and should be used with caution.

The advantage of gated spillways is that they can be operated with less flood surcharge, resulting in lower dam heights. However, when comparable risk control measures, (such as back-up power and control systems, secondary and auxiliary spillways, additional flood surcharge and fuse plug spillways etc), are incorporated into the design, the project cost of a gated spillway project is not necessarily more economical than a fixed crest design.

5.8.5 SPILLWAY CAPACITY OPTIMISATION

In some cases, a single spillway is designed for the full range of flood discharges required for the dam. In other situations there may be a service spillway for frequent floods and an auxiliary spillway which comes into operation for extreme floods. For cost reasons, auxiliary spillways are often unlined rock channels where the risk of channel erosion and repair works is accepted because it is a rare event that will cause a damaging level of discharge. It is important in unlined spillways to have an erosion resistant crest so that the spillway crest is not eroded down or undercut, increasing the flow rate and volume beyond the design intent.

5.9 COST CONSIDERATIONS

Spillways often represent a significant proportion of the project cost and optimisation of the design is an important step in the process of producing an efficient and effective design. The process for optimisation involves:

- Selecting a trial spillway arrangement and width for the required storage level
- Routing the inflow design flood through the reservoir
- Using the flood surcharge together with an appropriate freeboard allowance to set the dam height
- Assess the cost of the dam, spillway and outlet works
- Repeat the analysis as appropriate to select the most cost-effective arrangement

The principal items of cost in a chute spillway for example are the cost of the excavation and the concrete works for the crest and spillway lining. However, these costs can sometimes be mitigated where the rock from a spillway excavation can be used as the fill zone for the embankment.

The cost estimates need to be developed to the stage where reasonable costs can be estimated for each of the major components of the dam, the spillway including all the appropriate feature and structures, the dam body, the outlet works and access arrangements. The estimates should be repeated for each combination of spillway width and embankment height so that the optimum arrangement of the works can be identified.

Figure 5-4 Harding Dam; unlined spillway was used as a source of rockfill

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6 DIVERSIONS

6.1 INTRODUCTION

Almost all types of dam construction at river sites require the foundation preparation and initial material placement to be performed in the dry. Even if "wet" embankment placement is elected (i.e. rock fill placement into water), the river flow at the location of rock fill placement must be curtailed.

Thus, diversion arrangements are integral to dam construction and are usually undertaken in stages:

- Preparation of the diversion channel or conduit (tunnel)
- Closure of the river – or the initial redirection of the flow into the diversion conduit or channel
- Handling of the flow in the diversion arrangements during floods during construction.

There are a multitude of factors contributing to the selection of the type of diversion arrangements and associated design criteria including (but not limited to):

- Topography
- Type of Dam
- Flow magnitudes and seasonality (variability)
- Schedule demands and partial project operation
- Necessity for maintaining river traffic
- Environmental factors
- Risk tolerance
- Debris and ice passage
- Material availability
- The extent to which elements of the diversion arrangements will be incorporated into the final project.

A comprehensive discussion on river control during dam construction is provided in ICOLD Bulletin 48a. This chapter is included to highlight the extent to which diversion considerations must be taken into account in the choice of dam type for a particular site.

6.2 ROLE OF RIVER DIVERSION IN SELECTION OF DAM TYPE

The diversion arrangements are an integral part of the dam construction and thus the cost of, and risks associated with construction. While associated facilities such as a powerhouse, spillway and low-level outlet can often be ignored in a cost comparison of dam solutions if they are remote from the dam location, the diversion arrangements are almost always in close proximity to the dam and should therefore always be considered in the cost comparison between different types of dam.

It is assumed in this discussion that the design of the diversion and its cost will factor in to the type of dam, but in some cases the diversion may completely govern. For example, an earth dam is not feasible if the diversion works cannot be configured to prevent the partly completed dam from being overtopped during construction. It is also possible that the overall schedule demands a particular type of diversion that suits both the dam construction and an associated powerhouse construction.

Additionally, the risks associated with each type of diversion have a significant role in the cost of each type of dam. The ideal location for a particular type of dam within a stretch of river can be dependent on the type of diversion, which translates into a potential for comparison of dam configurations at slightly different dam locations for different types of dam being considered.

6.3 TYPES OF DIVERSION

6.3.1 SINGLE STAGE DIVERSION

The single stage diversion is usually used in narrow valleys, and is normally conducted in the following sequence:

1. build a partial cofferdam, allowing construction of diversion tunnels, culverts, channels and control works
27. build diversion tunnels, culverts, channels and control works
28. divert flow through these passages using a closure structure
29. build full-size cofferdam on or behind the closure cofferdam
30. build permanent works
31. close diversion passages, modify for permanent use if integrated into the final ancillary works water control scheme
32. impoundment begins upon final river closure.

In conceptualizing the diversion works, attention must be given to the establishment of the closure cofferdam at a time of reduced flow. Particular design features adopted for the single stage diversion will be interrelated and progressed along with the type of dam (as discussed in 5.0 below) and the tolerance of the cofferdam and permanent works to overtopping.

6.3.2 MULTI STAGE DIVERSION

The multi stage diversion is appropriate for wide valleys and wide rivers where tunnel diversions would not form part of the completed facilities. In the multi stage approach, the sequence is normally:

1. build a cofferdam - which often extends into the river channel – to provide a dewatered area for construction of the permanent outlet works (or temporary sluices) and part of the main dam. In medium width valleys, or where topographical conditions dictate, this may involve excavating the opposite bank of the river to provide adequate hydraulic capacity during initial construction activities;
33. the outlet works, any partial concrete dam sections, and temporary or permanent sluices, are built in the dry behind the cofferdams;
34. the cofferdams are removed to divert the river through the outlet works, and temporary or permanent sluices, although sometimes, in order to facilitate the diversion without damage to structures the dewatered area must be enlarged slightly by extending the cofferdams further into the flow during a dry season before removal of the first cofferdams;
35. the second stage cofferdam is built in the river to fully divert flow;
36. the remainder of the project structures are built in the dewatered area behind the second stage cofferdam;
37. final closure is achieved by closing the temporary and permanent sluices.

Similar to the single stage diversion, particular design features adopted for the multi stage diversion will be guided by the type of dam and the tolerance of the cofferdam and permanent works to overtopping. In addition, the selection of fill materials for the parts of the cofferdam subject to relatively high flows in the river are critical, particularly during the second stage diversion of the river. The use of protective sheet piling and possible use of Tetrapods and their like may be appropriate, as well as model studies to verify the diversion process.

6.3.3 CONSTRUCTION ABOVE RIVER LEVEL

The third method of diversion, essentially without riverine interference, relies on the building of temporary or permanent sluices and any power plant etc. on the river banks away from the main river channel without any diversion. Thus, the method is only possible at wide river valleys, where cofferdams need only to be introduced for the construction of the second stage of the works.

6.4 APPLICABILITY TO DAM TYPE

6.4.1 FACTORS TO BE CONSIDERED

All three types of diversion can be used in conjunction with all types of dam. However key considerations in adopting them relate to:

- the return period of the chosen diversion flood
- the capacity for the dam to survive overtopping during construction
- the provision of adequate connection between the first stage permanent works and the second stage
- the closure arrangements

The use of cellular cofferdams may allow for multistage diversion arrangements to be more predictable and less risky at sites with special considerations such as lack of space or high velocities that could erode a rockfill cofferdam, although cellular cofferdams will imply additional costs.

6.4.2 EMBANMENT DAMS

All three types of diversion may be used with embankment dams. However, overtopping during construction is not viable and thus the diversion arrangements must be adequately sized to significantly lower the risk of overtopping. Return periods for diversion floods of the order of 50 years are commonly adopted. Generally larger diversion tunnels or culverts with high cofferdams will be necessary.

6.4.3 CONCRETE DAMS

All three types of diversion may be used to construct a concrete dam. Concrete dams can be overtopped which allows adoption of floods with short return periods. The return period adopted will depend on the expected flood durations and peak flows which allow estimates of disruption of the works to be made. Return periods for diversion floods of between one year (accepting annual overtopping) and ten years have been used. The lateral extent of overtopping can be restricted to a central section by keeping the elevation of the dam higher on the abutments. Diversion conduits can be constructed on the dam foundation or through the dam body.

6.4.4 INTEGRATED COFFERDAMS

Cofferdam volumes and costs can be considerable and they should therefore be integrated into the main dam construction as far as technically possible. The diversion will typically comprise a small cofferdam with diversion conduits sufficient to take a low-flow season flood. The main dam will then be constructed in this first period to an elevation where it will not be overtopped by the design diversion flood. This part of the dam forms the main cofferdam. Such cofferdams are most easily achieved where there is a watertight element at the upstream edge of the dam. This applies to concrete, asphalt and membrane faced dams and all concrete dams. Integrated cofferdams for rockfill dams with central cores typically contain a separate watertight element such as an upstream facing, earth core or concrete or sheet pile wall.

6.5 INCLUSION IN ANALYSIS OF TYPE OF DAM

Essentially the cost and risk of diversion arrangements must be included in the cost comparison of the various dam types, using the following outline methodology:

- Select possible dam types, based on site geology; foundation characterization; material availability; and other factors;
- Select possible dam axis for each type of dam under consideration;
- Assess construction schedule for each scheme, and vulnerability to flooding;
- Assess least cost diversion scheme for each dam configuration;
- Perform first optimization of cofferdam height, and tunnel or diversion channel dimensions;
- Estimate cost of diversion for each dam type, usually excluding the cost of any facilities that form part of the permanent works;
- Carry diversion costs into overall comparative cost estimate for each dam configuration.

7 EVALUATION OF TECHNICALLY FEASIBLE DAM TYPES

7.1 INTRODUCTION

The starting point for the evaluation will be dams that have been found to be technically feasible using the information given in Chapters 2 to 7. A bill of quantities (BQ) with unit rates has to be developed for each. The costs thus obtained are central to making the selection of dam type. Other elements not reflected in these costs and important in making the final choice might be environmental and socio-economic factors.

External interests and regulatory bodies may affect the evaluation process or there may be no such involvement. At its simplest, the process will involve a small team that has to produce a convincing report without external involvement. Whichever the case, the core of the work remains the same and this process is described in Sections 7.2 to 7.6 below.

The budgets for dam type selection studies are typically small and simplified methods of estimating comparable costs may have to be adopted. Such methods must reduce the probability of the results being misleading, see Section7.7.2. The dimensioning and preliminary designs should be detailed enough to capture and reflect the main differentiating factors and cost determinants.

Selection of dam type is not always a single exercise. A choice may be made at an early project phase such as a pre-feasibility study when the information available is small. The choice should be re-visited at later project stages as more information, particularly on ground conditions, becomes available. This should certainly be done at the feasibility stage and the selected dam type may well change. In some cases, competing dam types may have costs and other selection criteria that are so close that there is no clear choice. It may then be prudent to either pursue two dam types to full tender design (a costly and therefore rare option) or to allow the tenderers to propose an alternative dam type. This should include a full design specification for the alternative.

Dam type selection is commonly made using a deterministic approach where the cost estimates of the alternative dams are compared with socio-economic and environmental factors brought in to the decision-making process. A probabilistic approach may be adopted as an alternative or subsidiary method. Both approaches are treated in the text below. Whenever possible factors affecting a dam alternative should be translated into monetary values to promote an objective quantitative decision-making approach.

7.2 QUANTITIES

The main quantities for each dam type will be estimated from the designs produced with the guidance provided in Chapters 3 to 7. In the first instance the major and cost-driving elements will be estimated. These are given for each of the dam types treated in Chapter 3 and are typically:

- Diversion
- Excavation volumes and treatment
- Dam body volumes, if necessary broken down into different zones, where such zones are expected to have different unit costs.
- Areas of dam faces
- Spillways
- Outlets
- Construction schedule

The depth of excavation will commonly not be known with any precision. If the assumed depth of excavation affects the dam types differently, then quantities pertaining to alternative depths may have to be estimated. One approach is to make a best estimate of the depth of excavation and make the dam designs accordingly. Different dam types may have different strength and stiffness requirements for the foundation and may therefore have different estimated foundation depths. Given the importance of foundation depth when comparing dam alternatives, emphasis should be made to investigate this factor in early decision-making stages. Even with restricted time and budget, seismic refraction surveys could provide a valuable input to reduce the excavation uncertainty in a time and cost-effective way.

7.3 ALLOWANCES FOR CONSTRUCTION ITEMS NOT EXPLICITLY QUANTIFIED

These allowances are typically given as percentages of the total cost of the work and are based on precedent. Chapter 3 gives guidance on these numbers.

7.4 UNIT RATES

7.4.1 DATABASES

Unit rates are commonly obtained from databases held by individual consultants or owners. These are mostly derived from a miscellany of sources where tendered rates often predominate. There may be considerable spreads in the tendered rates that reflect strategic pricing or the particular circumstances of a tenderer such as access to existing equipment, creative sourcing of materials or labour or extension of the construction period to allow the use of smaller equipment. Other factors, such as macroeconomic conditions and contractual risk allocation may affect reference unit prices.

As dam type selection may be made years before the work is put out to tender, it will normally not be possible for the evaluation team to take such factors into account. However, the database should allow estimates of expected average, minimum and maximum rates for use in the evaluation process.

The items in the database may be of different dates. Thus, to account for inflation, cost escalation needs to be considered to bring the rates to a common reference date.

Cost databases of this kind have no universal validity. Costs will vary from country to country or region to region and will be affected by the relative remoteness of a site, i.e. access and transport costs and the proximity of material and energy sources and labour costs.

7.4.2 CONTRACTOR TYPE ESTIMATES

Such estimates are based on the cost of materials, labour and equipment. The quantities required of each to produce a unit of completed work are estimated. The materials costs per unit mass or volume, labour cost per hour for different grades of labour and the costs of owning and operating machinery are required to complete the unit cost estimate. The cost of overheads and profit has to be added.

The success of this method is dependent on knowledge of the various unit rates. In some countries these may be well known and reasonably constant, but in others there may be considerable variation. Without detailed knowledge of the contracting environment in a particular country, the results of such an analysis may be misleading. If such estimates are to be used, the end result should be checked against typical tendered rates for similar work.

When preparing this type of estimates, it should be noted that contractor bid prices are not only determined by direct costs. Their unit prices are also influenced by market factors such as: availability and price volatility of labour, equipment and materials, expected number of bidders, current backlog of contractors, expectations of new work in the future, a contractor's appraisal of the competitiveness of other bidder's, reputation of the client, and the contractor's risks assessment.

Developing a complete list of unit rates by such methods is time-consuming and may be beyond the scope of a typical dam type selection study. It may be helpful to employ a experienced costing analyst who has put together bids for contractors on the types of dams under consideration

7.5 ITEMS NOT INCLUDED IN THE ESTIMATES

In some cases, there will be work items common to all dam types under consideration and where the quantities are substantially invariant. As an example, this may apply to grouting and spillways where alternative embankment dam types are under consideration. If the extent of such work is assessed as being about the same, the work involved in making an estimated of the quantities may not be warranted.

7.6 CONTINGENCIES

Contingencies in tender documents relate mainly to uncertainties in ground conditions and quantities thus affected. In dam type selection, these uncertainties should be expressed as alternative estimates of quantities and not percentage additions to a base cost.

7.7 COST ASSOCIATED WITH CONSTRUCTION SCHEDULE

Construction schedules have to be drawn up for each dam type and compared to the schedule for other major components of the project. The data used to develop the schedules is often a combination of precedence (the time it has taken to do similar work) estimates of plant and labour productivity and how much of each can be usefully employed at any one time. The flood hydrology and river diversion plans can have a marked effect. Correct sequencing of the works is vital as it is important to allow for delays due to climatic effects and floods. There are always uncertainties in the estimated construction times due to changed ground conditions, unusual and unpredictable floods, rainfall or durations of frost, breakdowns in vital plant and external factors such as failure of timely supplies. Some dam types may be more sensitive to such factors than others.

Examination of the overall project schedule may show that costs are associated with early or later completion relative to these other elements. An example from hydropower projects is that early impoundment and generation normally has a large cost benefit through early income. If the dam is completed after the power component, there will be a loss of income which should be included in the cost comparison. Likewise, if the dam is completed and impounded well ahead of the power component, the cost of the investment without return from power sales, will also affect the overall project cost

Whenever possible, the difference in the total schedule duration for each dam type should be translated into a monetary value. This could be achieved for example estimating the loss of benefits (delayed income) in net present value and estimates of interest on capital during construction.

7.8 ACCURACY OF ESTIMATED COSTS

Comparable costs of alternatives are obtained when the unit rates are consistent. Absolute accuracy is less important than this internal consistency.

7.9 METHODS OF COST COMPARISON

7.9.1 DETERMINISTIC METHODS

The most common method of resolving the choice of dam type is a comparison of costs. Environmental and socio-economic factors should be part of this evaluation to the extent that there may be differences between the options under consideration. Thus, the choice is based on best estimates of rates and quantities.

Where the cost differences are large, there will be little doubt about the result.

Sensitivity analyses should be made by varying critical rates and quantities to check the robustness of the choice. Alternatively, probabilistic methods can be used as set out below.

7.9.2 PROBABILISTIC METHODS

The cost of a dam may be estimated from quantities and unit rates which are not known precisely. A plausible minimum and maximum rates or quantities may be set and an informed guess at the most probable value can be made. The probability distribution should properly characterize the expected distribution of the data. Often, due to data limitations and is ease of use, a triangular distribution of values is commonly adopted. This has a continuous probability distribution with:

lower limit = a

upper limit = b

mode = c = best estimate or guess

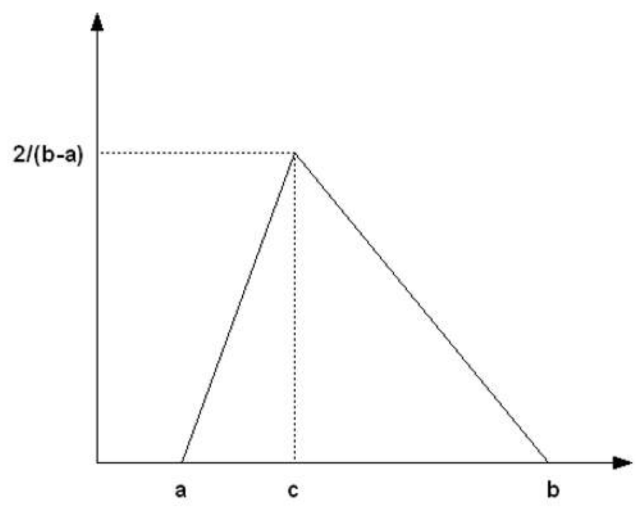

Figure 7-1 Probability density function for triangular distribution

These values can be tabulated for each rate or quantity under consideration. Each quantity can be randomised within the triangular distribution using the following equations:

$$X = a + [U\,(b-a)\,(c-a)]^{1/2} \qquad 0 < U < F(c)$$

$$X = b - [(1-U)\,(b-a)\,(b-c)]^{1/2} \qquad F(c) \leq U < 1$$

where U = random number in the range 0 to 1

$F(c) = (c - a)/(b - a)$

X = current value

The objective of the exercise is to determine the proportion of times one dam costs less than the other, when using the randomised rates and quantities. The minimum, median and maximum cost for each for the dam type are also desired results. This can conveniently be performed with a Monte Carlo simulation. In this procedure a large number of estimates are made with randomised variables (unit rates, quantities) using the equations above. The method is illustrated in the example that follows.

Table 7-1 shows a simplified bill of quantities with unit rates for two sample dams. The unit rates have been given three values: probable minimum, best estimate and probable maximum corresponding to the factor a, c and b respectively in the triangular distribution (Figure 7-1) with the quantities treated in the same way except as follows:

- In line 8 and 9, cement and pozzolan for the RCC dam, the content per cubic metre has been randomised (the mix proportions will typically not be known at the time) and the randomised unit rates are multiplied with the randomised RCC volume (column 12).
- The percentage allowance for items not quantified has been randomised and the rate then applied to the sum of the cost estimates for the items in the lines above.
- The cost of the spillway has been presented as a lump sum.
- The benefit from early completion has been given as a fixed sum per month. Values (months) for early completion (- 4) and late completion (+ 4) are given.

Table 7-1 shows one instance of a calculation in a Monte Carlo simulation where the total number of instances was 5000.

Table 7-1 Bill of quantities for Monte Carlo simulations
One random instance of the simulation

1			Unit rates						Quantities						
2	Basic BQ	Column no.	1	2	3	4	5	6	7	8	9	10	11	12	13
3	Factor in trianglular distribution		a	c	b	U	F(c)	Xr	a	c	b	U	F(c)	Xq	
4–5	RCC	Units	Probable Min	Best estimate	Probable Max	rand		Unit rate	Probable Min	Best estimate	Probable Max	rand		Quantity	SUM
6	Excavation	m^3	5	15	25	0.740	0.500	17.78	90,000	105,000	130,000	0.205	0.375	101,092	1,797,806
7	RCC	m^3	32	38	45	0.939	0.462	42.64	700,000	710,000	780,000	0.746	0.125	742,263	31,649,291
8	Cement	kg/m^3	85	103	115	0.067	0.583	90.94	70	80	90	0.284	0.500	57,554	5,233,693
9	Pozzolan	kg/m^3	65	75	95	0.925	0.333	88.27	80	100	120	0.373	0.500	97	8,586
10	Facing	m^2	25	30	35	0.501	0.500	30.01	25,000	30,000	35,000	0.537	0.500	30,190	905,876
11	Foundation treatment	m^2	8	10	15	0.245	0.286	9.85	12,000	13,500	15,000	0.018	0.500	12,281	121,013
12	Items not quantified		7.00%	8.00%	9.00%	0.012	0.500	0.072						39,716,265	2,841,007
13	Spillway	lump sum	4,000,000	5,000,000	7,000,000	0.664	0.333	5,580,831	1	1	1	0.969	1.000	1	5,580,831
14	Benefit from early completio	Currency units/mo	-300,000	-300,000	-300,000	0.621	0.500	-300,000	-4	0	4	0.407	0.500	-0.4	117,992
15	Duration of construction, mo	Month				0.187			21	21	21			21	
16	Cost of money	%/year	6.00%	7.00%	8.00%	0.707	0.500	0.07							2,124,157
															50,380,252
	ECRD														
17	Excavation	m^3	11	15	25	0.571	0.286	17.25	200,000	210,000	250,000	0.983	0.200	244,190	4,211,965
18	Rockfill	m^3	10	12	17	0.419	0.286	12.49						2,319,809	28,975,819
19	Core fill	m^3	12	18	25	0.636	0.423	19.05						347,971	6,627,679
20	Filters	m^3	20	22	30	0.054	0.200	21.04						69,594	1,464,372
21	Foundation treatment														
22	core	m^2	8	10	15	0.628	0.286	11.39	12,000	13,500	15,000	0.512	0.500	13,519	154,023
23	shells	m^2	3	4	6	0.856	0.333	5.07	39,000	41,000	43,000	0.484	0.500	40,967	207,669
24	Spillway	lump sum	6,500,000	8,000,000	11,000,000	0.823	0.333	9,452,351	1	1	1	0.583	1.000	1	9,452,351
25	Items not quantified		4.00%	5.50%	7.00%	0.067	0.500	0.05				0.130			2,324,352
26	Benefit from early completio	Currency units/month	-300,000	-300,000	-300,000	0.904	0.500	-300,000	-2	-6	-10	0.587	0.500	-13.6	4,090,544
27	Duration of construction, mo	Month				0.389			27	27	27			27	
28	Cost of money	%/year	6.00%	7.00%	8.00%	0.744	0.500	0.07							2,171,291
29															59,680,064

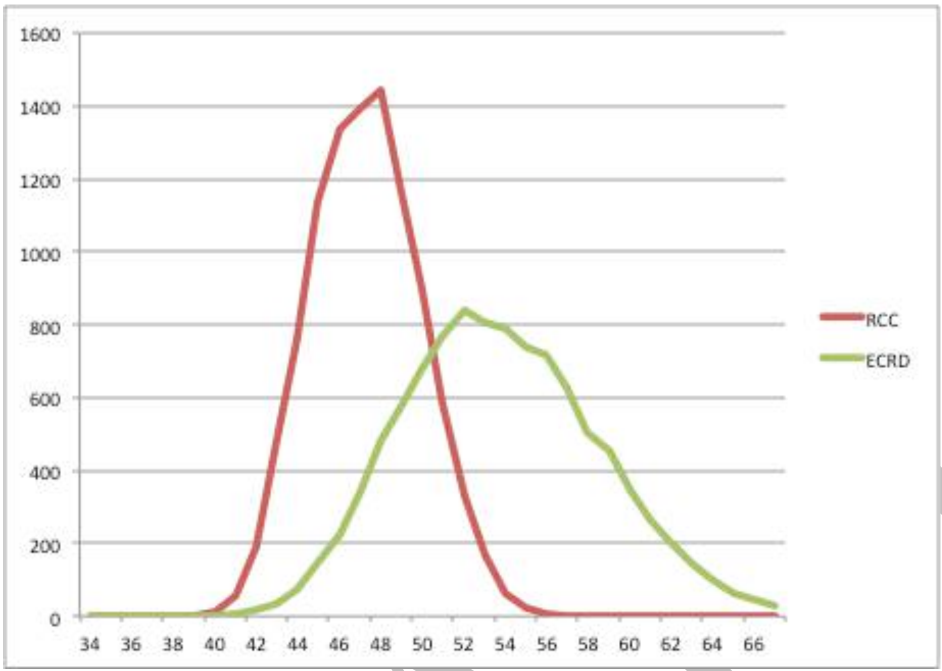

Figure 7-2 Histogram of values from Monte Carlo simulation

- The duration of construction has not been randomised but is varied according to the randomised quantity of early (or late) completion.
- The cost of money (interest on investment prior to completion) is estimated from a randomised annual interest rate and the duration of construction.

For the ECRD the following has been applied:

8 The dam fill volumes have been given a dependence (based on calculations) on the randomised excavation volume and only the rates have been randomised.

9 The other rates have been treated as for the RCC option.

In this example the Monte Carlo simulation has been run 5000 times. From the data generated the histograms for each dam have been obtained, Figure 7-2. The following can be obtained from a simple count of the results:

	RCC	ECRD	RCC/ECRD
Median	46,730,000	53,104,000	91.4%
RCC < ECRD	89.0%		

In this example the difference in median cost is less than 10% but the RCC has a lower cost than the ECRD in 88.7% of cases. This gives confidence that an RCC dam is the lowest cost option.

Monetised environmental and socio-economic factors can be included in the simulations to the extent that such data is obtainable and that there also are significant differences between dam types under consideration.

The method of randomising rates and quantities or alternative methods of setting them has to be devised for each dam type selection study. The above procedure is provided only as an illustration of the approach.

9.1 MULTI-CRITERIA EVALUATION

As mentioned before, prevalence should be given to compare feasible dam types for a particular site translating the various differentiating factors into monetary values to facilitate an objective evaluation. However, sometimes important decisive factors for a dam selection cannot be properly translated into monetary quantitative values. This is the case for example of environmental considerations, local experience, availability of certain technologies and precedent.

In light of this, it is convenient to complement the qualitative cost and benefit comparison of the possible dam types with a multi-criteria evaluation. The different relevant factors can be evaluated using a qualitative scale (often from 1 to 10) and weighted depending on the importance given to each factor by the evaluator. This method is sensitive to the subjective criteria and beliefs of the evaluator. It is especially valuable when the estimated budget for alternative dam types are very similar. However, when the cost or schedule estimates spread between two alternatives is considerable, the evaluator should beware reaching contrary conclusions by implementing this complementary method.