CRITICAL STUDY OF DESIGN OF CORE FOR EMBANKMENT DAMS WITH SPECIAL REFERENCE TO TEHRI DAM

A DISSERTATION

submitted in partial fulfillment of the
requirements for the award of the degree
of
MASTER OF TECHNOLOGY
in
WATER RESOURCES DEVELOPMENT

By
SUBERDI. Z.

WATER RESOURCES DEVELOPMENT TRAINING CENTRE
INDIAN INSTITUTE OF TECHNOLOGY, ROORKEE
ROORKEE - 247 667 (INDIA)

JANUARY, 2002

CANDIDATE'S DECLARATION

I hereby certify that the work which is being presented in the dissertation entitled **"CRITICAL STUDY OF DESIGN OF CORE FOR EMBANKMENT DAMS WITH SPECIAL REFERENCE TO TEHRI DAM"** in partial fulfillment of the requirement for the award of the ***Degree of Master of Technology*** in Water Resources Development (Civil) submitted in the Department of Water Resources Development Training Centre of Indian Institute of Technology is an authentic record of my own work carried out during a period from July 2001 to January 2002 under the supervision of Prof.Dr. B.N. ASTHANA, Emiritus Fellow of Water Resources Development Training Centre and Prof.Dr. RAM PAL SINGH, Professor, Water Resources Development Training Centre of ***Indian Institute of Technology, Roorkee, India.***

The matter embodied in this dissertation has not been submitted by me for the award of any other degree.

Dated : January 24, 2002

SUBERDI. Z.

This is to certify that the above statement made by the candidate is correct to the best of our knowledge.

RP Singh

(Prof. Dr. RAM PAL SINGH)
Professor,
Water Resources Development
Training Centre (WRDTC),
Indian Institute of Technology, Roorkee,
Roorkee, INDIA.

(Prof. Dr. B.N. ASTHANA)
Emiritus Fellow,
Water Resources Development
Training Centre (WRDTC),
Indian Institute of Technology, Roorkee,
Roorkee, INDIA.

ACKNOWLEDGEMENT

It is my privilege and pleasure to express my profound sense of deep respect, sincere gratitude and indebtedness to my guide of dissertation Prof.Dr. B.N. Asthana, Emiritus Fellow, Water Resources Development Training Centre; Prof.Dr. Ram Pal Singh, Professor, Water Resources Development Training Centre; and to my M.E. faculty adviser Prof.Dr.Nayan Sharma, Professor, Water Resources Development Training Centre Indian Institute of Technology, Roorkee, for their continued inspiration, valuable guidance, advice, cogent discussion and encouragement throughout this dissertation work. Also, I express my gratitude, with high regards, to Prof.Dr.Bharat Singh, Professor Emiritus, Water Resources Development Training Centre Indian Institute of Technology, Roorkee, India, for his advice, discussion and inspiration for this dissertation.

I am very much grateful to Professor Devadutta Das, the Director and all faculty members and staffs of Water Resources Development Training Centre for their inspiration, kind cooperation and for the facilities provided for this work.

I wish also to express my wholehearted gratitude to the Ministry of Settlement and Regional Infrastructure (formerly Ministry of Public Work), Ministry of Office of State Enterprises Development Republic Indonesia, and also my company, PT Nindya Karya (Persero), the state owned company where I have been working for more than fourteen years that gave me an opportunity to study in Master of Technology Degree course at Water Resources Development Training Centre, Indian Institute of Technology, Roorkee, India.

Special sincere thanks to Mr. B.G. Agarwal (Deputy General Manager Contract), Sandeep Singhal, M.E (Deputy Manager Design), P.S. Rawat, M.Tech. (Deputy Manager Design) , and Dr. Ajay K. Singh (Deputy Manager Design) of Tehri Hydro Development Corp.Ltd., for valuable help and advice; and also to the officers of Irrigation Department Uttar Pradesh for their help to support this work.

Finally, special sincere thanks to my Parents, my wife Dra. Siti Saniah, my two sons M.Ardhy Rizki Ananda (9 yrs.old) & A.Fitrah Riyadhi (5 yrs.old),and to my other family members in Indonesia for their tolerance, encouragement and infatigable prayers in supporting throughout my study in this Water Resources Development Training Center of Indian Institute of Technology, Roorkee, India.

Special thanks are also to my previous government University : Polytechnic University of Indonesia, Jakarta, and University of Diponegoro, Semarang, Indonesia, for their valuable training in the basics of Civil Engineering.

The last but the least thank to all friends in Water Resources Development Training Center of Indian Institute of Technology, Roorkee, India, for their suggestions and helps during this work.

(S U B E R D I. Z.)

SYNOPSIS

Core is a key structure of a high zoned embankment dam. It controls seepage through dam. Its thickness, material properties, slopes, position (geometry) contact & interface condition with the foundation, abutments, and shell zones or transition / filter layers, ~~and~~ are the important parameters of its design.

In this dissertation, an attempt has been made on critical study of design of core for high zoned embankment dams with special reference to Tehri Dam.

A study of literature has been carried out from ICOLD, ASCE Journal of SMEFD / GED, Canadian Geotechnique, Water Power & Dam Construction, etc. Critical factors affecting design of core for embankment dams have been discussed in Chapter 2 and in Chapter – 3 a review of design of core of 25 high embankment dams of the world (130 m to 335 m high), which have different core shape or geometry or position (vertical, moderately inclined, and inclined type), material characteristics (with or without blended), thickness, foundation condition has been given. The performance of core in some cases has also been given. All the information has been summarized in a table, and the inferences have been made.

The design data of core of Tehri Dam, the highest zoned embankment dam under construction in India have also been collected from Tehri Hydro Development Corporation (THDC) office in Rishikesh and critically examined. The study revealed conformity of its design with the inferences drawn from the review of core design of high dams.

This study has shown that the design of core is site specific and depends on quantity and quality of the locally available material. However, in high dams in Seismic Zones it shall be checked for stresses and deformations by FEM analysis.

CONTENTS

	Page
CANDIDATE'S DECLARATION	i
ACKNOWLEDGEMENT	ii
SYNOPSIS	iii
CONTENTS	iv
LIST OF TABLES	vii
LIST OF FIGURES	viii
Chapter 1: Introduction	1
1.1. General	1
1.2. Objective and Scope of Study	3
1.3. Methodology of Study	3
1.4. Organisation of Report	3
Chapter 2: Critical Factors Affecting Design of Core for Embankment Dams	5
2.1. General	5
2.2. Core-Slope	6
2.3. Core Thickness	7
2.3.1. Factors Governing The Core Thickness	8
2.3.2. Usually Adopted Core Thickness	9
2.4. Core Material	10
2.4.1. Size of Material	12
2.4.2. Permeability, Erosion Resistance, Compacted Density, Flexibility, Compressibility, Moisture Content, and Plasticity	13

2.4.3. Control of Water/ Moisture Content, Dry Density, and Compactive Effort 19
2.4.4. Shear Strength 22
2.4.5. Pore Water Pressure 22
2.4.6 Cracking, Leakage and Hydraulic Fracturing Problems 24
2.4.7. Stress-Strain Relationship Model with FEM and Related to Earthquake Consideration. 27
2.5. Core Position (Geometry) 28
2.6. Core Contact Treatment Design 33
2.7. Filters / Transition Design 35
2.7.1. Design Criteria 35
2.7.2. Filter Thickness and Layers 37
2.8. Summary 38

Chapter 3: Review of Literature of Cores of High Embankment Dams 40
3.1. General 40
3.2. Central Core Embankment Dams with & without Blended Core Material 41
3.3. Moderately Inclined Core Embankment Dams 85
3.4. Inclined Core Embankment Dams 111
3.5. Study and The Inferences 115

Chapter 4: Design of Core for Tehri Dam 121
4.1. General 121
4.2. General Description of Tehri Dam Project 121
4.3. Design of Core 127
4.3.1. Core Slope 127
4.3.2. Core Thickness 127
4.3.3. Core with Blended Material 127
4.3.4. Core Position (Geometry) & Seismic Consideration 130

4.3.5. Core Contact Treatment 134
4.3.6. Filter / Transition Design 135
4.4. Summary of Study 136
Chapter 5: Conclusions and Suggestions for Further Study 138
5.1. Conclusions 138
5.1.1. General 138
5.1.2. For Tehri Dam 139
5.2. Suggestions for Further Study 139
REFERENCES 145

LIST OF TABLES

Tittle	Description	Page
Table 2.1.	Hydraulic Gradients in the Cores	9
Table 2.2.	Some Important Details of the Core of some important Embankment Dams	11
Table 2.3.	Average Properties of Soils (After U.S.B.R.)	12
Table 2.4.	Suitability of Soil for Construction of Dams	12
Table 2.5.	Maximum size of particles in the core of some Dams	13
Table 2.6.	Classification of core materials on the basis of resistance to concentrated leak (after Sherard, 1967, and IS 8826-1978).	14
Table 2.7.	Impervious Core and Properties of Core Material	16
Table 2.8.	Estimate of the relationship between the coefficient of permeability of relatively thick impervious cores and dissipation of pore pressures in the central part of the core.	23
Table 2.9.	U.S.B.R. Filter Design Criteria	36
Table 3.1	General engineering properties of the core material, together with other zones, Trinity Dam.	61
Table 3.2.	Properties of core material & other materials of Mo r n o s Dam.	84
Table 3.3.	Main parameters of the conveyor lines for delivering of Core Materials, Nurek Dam.	87
Table 3.4.	Shear test properties of core & other zones material of Oroville Dam, USA.	101
Table 3.5.	Typical "As placed" Densities and Moisture Contents of the W.A.C. Bennett Dam, Canada.	103
Table 3.6	List of High Embankment Dams with its Core Properties & its Important Characteristics.	118
Table 4.1.	Salient Features of Tehri Hydro Power Complex, India.	124
Table 4.2.	List of Alternatives for Optimization of dam section, and core geometry of Tehri Dam.	132
Table 5.1.	Vertical Core Embankment Dams (13 nos).	140
Table 5.2.	Moderately Inclined Core Embankment Dams (6 nos).	142
Table 5.3.	Inclined Core Embankment Dams (6 nos).	144

LIST OF FIGURES

Tittle	Description	Page
Fig. 2.1.	Cross Section with Vertical Core	6
Fig. 2.2.	Cross Section with Inclined Core	7
Fig. 2.3.	Core Plasticity (After *Rajcevic*)	18
Fig. 2.4.	Possible Core Distortions	18
Fig. 2.5.	Zones in which Different Core Materials are Distributed.	19
Fig. 2.6.	Variation of OMC with Compactive Effort	20
Fig. 2.7.	Moderately sloping core, Oroville Dam, California (USA).	30
Fig. 2.8.	Sloping core, Miboro dam (Japan)	31
Fig. 2.9.	Central Core, Nurek Dam, Russia.	31
Fig. 2.10.	Moderately Inclined Core, Rogun Dam (Russia), Typical Cross Section	32
Fig. 2.11.	Section of a near Vertical Core of Mica Dam, Canada	32
Fig. 3.1.	Types of Embankment Dams – Face Slopes indicative only (Slope protection and transition zones not shown).	40
Fig. 3.2.	Typical Sections for different foundation condition	41
Fig. 3.3.	Cross Section of the Nurek Dam, Russia	42
Fig. 3.4.	Longitudinal section along the abutment grouting galleries of Nurek Dam.	43
Fig. 3.5.	Grading Curves for the Core Materials of Nurek Dam	44
Fig. 3.6.	Result of Experimental Centrifuge Investigations of the Core, Nurek Dam	46
Fig. 3.7.	Maximum Section of Chicoasen Dam, Mexico	47
Fig. 3.8.	Geological profile along the dam axis of Chicoasen Dam.	47
Fig. 3.9.	Index Properties of the Clayey Material of the Core of Chicoasen Dam.	48
Fig. 3.10.	Gradation of the filters, transition zones and compacted rockfill materials (Field laboratory Data) of Chicoasen Dam.	49
Fig. 3.11.	Section along Dam axis showing treatment for abrupt change in the abutment slope, Chicoasen Dam	51
Fig. 3.12.	Location of Consolidation and Curtain Grouting below The Impervious Core of the Chicoasen Dam	51
Fig. 3.13.	Plastified Zone at the end of Construction, Chicoasen Dam	53
Fig. 3.14.	Crack crossing the crown and the downstream berm (elev.370), Chicoasen Dam	54
Fig. 3.15.	Maximum cross section of Darmouth Dam, Australia.	55
Fig. 3.16.	Average Gradation & its envelopes for Materials in dam zones, Darmouth Dam	56

Fig. 3.17.	Embankment construction : Properties of Core Material of Darmouth Dam	57
Fig. 3.18.	Pore pressures and earth presures in the core; End of construction. of Darmouth Dam.	58
Fig. 3.19.	The maximum cross section of Trinity Dam, USA.	59
Fig. 3.20.	Profile on axis of the dam, Trinity Dam, USA.	59
Fig. 3.21.	Palo Quemado Dam, Cross Section, South Africa.	62
Fig. 3.22.	Typical Cross Section of Swift Dam, USA.	63
Fig. 3.23.	The profile along the dam axis of Swift Dam.	64
Fig. 3.24.	Typical Gradation Curves of Embankment Materials (including Core Material) of Swift Dam.	65
Fig. 3.25.	Cross section of Geoscheneralp Dam, Switzerland.	67
Fig. 3.26.	Grain size distribution curves for Core material, Geoscheneralp Dam	68
Fig. 3.27.	Typical dam cross section and Grain Gradation curves of Gepatsch Rockfill Dam	70
Fig. 3.28.	Maximum cross section of Infiernillo dam, Mexico.	73
Fig. 3.29.	Cross Section of Foundation condition of Infiernillo Dam Site.	73
Fig. 3.30.	Index Properties and other Materials placed in the dam, Infiernillo Dam	74
Fig. 3.31.	Maximum Cross Section of Netzahualcoyotl Dam, Mexico.	76
Fig. 3.32.	Plasticity chart and grain size distribution curve of Netzahualcoyotl Dam.	77
Fig. 3.33.	The maximum section of the Beas Dam, India.	79
Fig. 3.34.	Grading Curves for the core material of Beas Dam, India.	80
Fig. 3.35.	Maximum Section through the Ambuklao Dam, Philippines.	81
Fig. 3.36.	Cross section of Ambuklao Dam sites, Philippines.	81
Fig. 3.37.	The grain size curve of core material and other zones of Ambuklao Dam.	82
Fig. 3.38.	The typical cross section of the M o r n o s Dam, Greece.	83
Fig. 3.39.	Design of Rogun Dam, Maximum Section, Tadjikstan	85
Fig. 3.40.	Maximum Cross Section of Boruca Dam, Costa Rica.	88
Fig. 3.41.	Section of the Guavio Dam, Colombia.	90
Fig. 3.42.	Foundation / Geology condition of the dam site, Guavio Dam.	91
Fig. 3.43.	Gradation curves of the core material and other zones of Guavio Dam.	92
Fig. 3.44.	Cross section of Mica Dam, Canada.	93
Fig. 3.45.	The cross section along axis of Mica Dam, showing geology looking downstream.	94
Fig. 3.46.	Core Widening at abutments, Mica Dam.	95
Fig. 3.47.	Gradation of core material and other zones of Mica Dam.	95

Fig. 3.48.	Chivor Dam – Plan and Sections, Colombia	96
Fig. 3.49.	Chivor (Esmeralda) Dam – Core and Filter Material Characteristics.	97
Fig. 3.50.	The maximum section of Oroville Dam, California, USA	98
Fig. 3.51.	Gradation Curve of Core and other Zones Material of Oroville Dam, USA.	100
Fig. 3.52.	Dam section of the W.A.C. Bennett Dam, Canada.	102
Fig. 3.53.	The typical core-contact surface treatment details of W.A.C. Bennett Dam Canada.	103
Fig. 3.54.	Gradation Cuves of Core Material & other zones, of the W.A.C. Bennett Dam, Canada.	104
Fig. 3.55.	The typical Cross Section of the Cougar Dam, USA.	105
Fig. 3.56.	The maximum section of Round Butte Dam, USA.	107
Fig. 3.57.	The section of Pueblo Viejo Dam, Guatemala.	109
Fig. 3.58.	The typical cross section of Tarbela Dam, Pakistan.	111
Fig. 3.59.	Profile across the river valley at the dam axis, Tarbela Dam.	112
Fig. 3.60.	Typical section of Miboro Dam, Japan.	113
Fig. 3.61.	Gradation curves of the core material of Miboro Dam.	114
Fig. 4.1.	Location Map of Tehri Complex, India	122
Fig. 4.2.	Typical Cross Section of Tehri Dam, India.	126
Fig. 4.3.	Gradation Curves for Blended Core Material of Tehri Dam, India.	129
Fig. 4.4.	Alternatives for Optimization of Dam Section, and Core Geometry of Tehri Dam.	134
Fig. 4.5.	Gradation Curves for Filter Material of Tehri Dam	136

Chapter 1
INTRODUCTION

1.1. General

The design of core in high zoned embankment dams has been rationalized in recent years due to the advances in soil mechanics, laboratory techniques, theory of seepages, computers, finite element analysis, and the experiences from some important high embankment dams such as the 335 m high Rogun and the 300 m high Nurek Dam in USSR, the 263 m high Chicoasen and 148 m high Infiernillo Dam in Mexico , the 244 m high Mica Dam in Canada, the 235 m high Oroville Dam in USA, 130 m high Miboro Dam in Japan, etc.

Embankment dams provide distinct advantage over concrete alternatives at most of the sites. Only geologically sound dam sites are suitable for concrete dams. In this respect embankment dams have much more flexible requirements with respect to foundations, as well as construction materials. They spread their load on a much wider base and can withstand greater deformation, both absolute and differential in comparison to concrete dams [109 and 121].

The reasons for preference of zoned embankment dams, especially for high dams, are therefore its suitability to almost all kinds of foundation, are its flexibility and better resistance against earthquake and above all its lower construction cost, which is mainly due to the use of materials available at or near the dam site and the revolutionary developments in heavy earth moving equipment.

In zoned embankment dam, core provides impervious element and outer shells of pervious material provide stability. The upstream pervious zone affords stability against rapid draw down and downstream pervious zone acts in steady state condition. The objectives of embankment zoning are: (a) to maximize the use of materials available from project excavation and to minimize the need for distant borrowing, (b) to develop the least complex embankment cross-section for economy of construction taking into consideration logical and cost effective sequence of placement, (c) to provide an embankment zoning arrangement that properly defends against excessive seepage pressures, erosion and piping, both internally and at its contact with foundation, abutment or adjacent structures. In rockfill dams, generally, the upstream rockfill has better shear strength than the filter zones, and the filter zones better than the impervious core, so the weakest shear plane is the impervious core [109 and 121].

The *design of core* for a zoned embankment dam, related to availability of materials, would involve: (a). selection of suitable material, (b).determining the core thickness , and (c).determining the core position within the dam section. All these factors / aspects are interlinked and have to be decided in relation to each other. When deciding on the soil to be used it should be borne in mind that the material most suitable for making impervious elements are clayey soils with natural moisture content equal to or exceeding that at plastic limit but lesser than that at liquid limit [15 and 121].

Cracking of the impervious core is one of the most troublesome problems that the engineer faces in designing an embankment dam. In high dams, deformation of the fill induces a tension zone located in the upper part of the structure, close to abutments. Also due to differential settlement between the core and the pervious zones, horizontal cracks may develop. The cracks are transverse cracks, horizontal cracks, longitudinal cracks and shrinkage cracks. If the cracks develop, it can be catastrophic when leakage starts through the cracks and brings about erosion of soil. Therefore measure must be taken into account in core design for *cracking, leakage, and erosion.* Also simultaneous study of the interaction of the three properties is needed. With such study, it is possible to *select a core material that has least damage potential* [121, 123a, 132 and 147a].

In the past many engineers dealing with design of dams considered that clay was the best material for the impervious core because of its very low permeability. In view of current experiences the trend is against highly plastic clays. From the consideration of erosion resistance of clay under concentrated leaks, the current trend is not to rely on clay. All clays will erode under severe condition and it is not considered adequate to rely on the relatively feeble erosion resistance of a clay soil to provide defense against piping due to concentrated leak. The principal function of the core is to provide impervious barrier. Prevention of piping is the function of filter. In the recent past, a good core material has been made by blending together two materials with radically different properties. For example, at some dams cohesionless fine silts were mixed with sandy gravel to obtain a core material with the imperviousness of silt and the high shear strength and low compressibility of the sandy gravel. In some dams, designers have used even silty materials for core by blending it with gravels (examples Tarbela, Chicoasen, Tehri). This blending has been done on almost all major dams. The current trend is to use the least costly of the available impervious soils for impervious core [54 and 82].

1.2. Objective and Scope of Study

The objectives of this study are: *(1)* to identify critical factors affecting the design of core, *(2)* to study core properties and other important characteristics of actual design of core of some high embankment dams of the world, *(3)* to identify important inferences regarding *(1)* and *(2)*, and to compare important characteristics of design of core with what has been adopted in Tehri dam, and *(4)* to identify scope for further study.

1.3. Methodology of Study

This study is based on the review of literature from ICOLD, ASCE Journal of SMEFD / GED Geotechnique, Canadian Geotechnique, Water Power & Dam Construction, etc., for 25 high embankment dams of the world and design data from Tehri Hydro Development Corporation (THDC) Project. From these data, study and analysis have been done and inferences have also been made for getting some critical factors for design of core for embankment dams, in general, as well as for Tehri Dam and suggesting scope for further study.

1.4. Organisation of Report

The study is presented in this report in five chapters. The contents of the chapter are briefly as below.

The *first* chapter gives an introduction of the problem, the objective, scope and methodology of the study.

The *second* chapter gives a study of critical factors affecting design of core for embankment dams which includes core slope; its thickness, material, position / geometry; core contact treatment design and transition / filter design.

The *Third* chapter gives review of literature of cores from 25 high embankment dams (130 m to 335 m high), which have different core shape (geometry or position), its material characteristics (with or without blending), its thickness, foundation condition, and performance of some of them.

The *fourth* chapter gives design of core for Tehri Dam.

The *fifth* chapter gives conclusions (in general, and for Tehri Dam) and the suggestions for further study.

Chapter 2

CRITICAL FACTORS AFFECTING DESIGN OF CORE FOR EMBANKMENT DAMS

2.1. General

The core may be defined as membrane built within a zoned embankment dam to form an impermeable barrier. The balance of the dam section is provided to ensure stability. The core may be of natural soil (clay or silt) or materials such as cement, asphaltic concrete, or of metal, plastic, rubber, etc.

The earliest form of core was built by driving two parallel lines of sheet piles, 3 or 4 m apart, and filing the space between them with puddle clay (often consolidated by the feet of sheep driven backwards and forwards across the dam). The sheet piles would be supported with random filing. The correct functioning of such a dam depended mainly upon the seal between the clay and foundation. The invention of the sheepfoot roller around 1904 simplified the compaction of the core material, the width of the core was increased and the piles abandoned [132].

As observed by the General Paper Committee of USCOLD (1964), a significant change has occurred in the location of the earth cores within the zoned embankment section. The usual choice in the past was either a vertical core or a sloping core, with former in the central portion of the dam and the latter laid on slope of the sides of 1.35H:1V. Recently, however, in many dams core has been located in an intermediate position, such that they can be called "moderately sloping". The term "moderately sloping" can be applied to a core position when the downstream face of the core is at about 0.5H:1V slope (Cooke,1964). A *"moderately sloping core"* is often adopted after considering: (1) the effect on the ease of construction, (2) the effect on stability and total volume, (3) the site topography, and (4) the possible development of tension cracks near the abutments [150].

Singh, et al [121] have explained about core position. The central core type would normally have a symmetrical core. When the downstream face of the core is inclined upstream at about 0.5H:1V, the core can be defined as moderately sloping.

Sharma [109] have also explained that the term 'moderately sloping core' can be applied when the slope of its upstream face lies between about 0.5H:1V and 1H:1V.

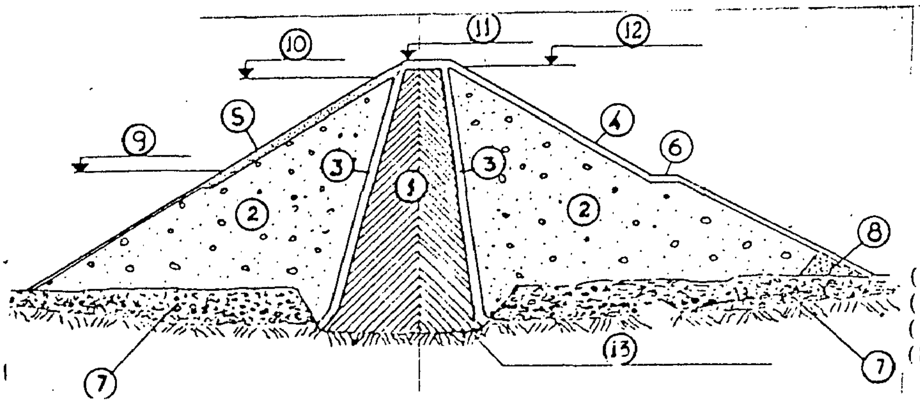
The thickness of the core will depend primarily on the material available, i.e. if good clay is available at low cost one would tend to be liberal with the size of core. On the other hand depending on local constraints thin cores are provided. The core in Furnas Dam, Brazil, is only one third of the height. In case much longer haulage of suitable core material is involved, thin core is preferred.

The principal factors considered in determining core dimension in a zoned embankment are: availability of core material; the relative economics of earthfill and rockfill; the plasticity of the available core material and its effect on the risk of core cracking; the extent and rate of reservoir draw-down; and the nature of the foundation rock under the core [132].

The design parameters for a core are: (a) selection of suitable material, (b) determination of core thickness and (c) determination of core position (geometry of core) within the dam section. All these are interlinked and have to be decided in relation to each other [121]. These parameters are discussed in detail in the following paragraph.

2.2. Core Slope

The side slopes of core shown in (Fig.2.1) are said to be positive slopes towards both sides. The core shown in (Fig.2.2) is tilted upstream at the lower part. In this case the downstream slope is negative. The downstream negative slope usually is not to exceed 1H:1V.



(1) Core.
(2) Shell.
(3) Filter.
(4) Downstream shell protection.
(5) Riprap.
(6) Berm.
(7) Natural alluvium of the river.
(8) Toe filter of the dam.
(9) Minimum operating water level.
(10) Maximum water level.
(11) Crown of the dam.
(12) Crown of the core.
(13) Cleaning up of rock surface. If possible, no blasting.

Fig.2.1. Cross Section with Vertical Core

The slope can be either positive and /or negative, as the project may require, and it is more convenient to have a vertical component of one material acting on the material of the subsequent zone. It is suggested that a minimum slope of 0.1H:1V to meet this condition in soils and 0.05H:1V in contact with concrete structures is adequate [139].

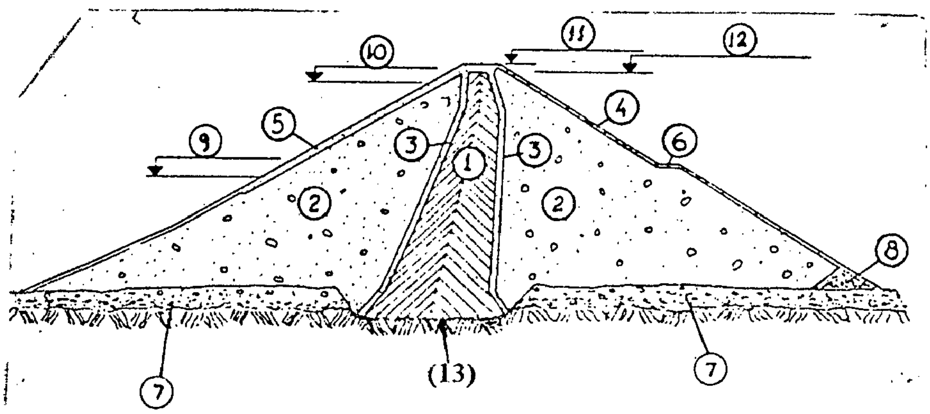


Fig. 2.2. Cross Section with Inclined Core

2.3. Core Thickness

The earth core is in direct contact with the foundation at the bottom and at the abutment. The thickness of the core is determined by three factors: the first is to minimize leakage to an acceptable amount (very small), the second is to permit placing with mechanical equipment, and the third is to prevent rupture (cracking) due to possible differential settlement [42].

In Indian Standard [89], I.S. 8826 –1978, about Guidelines for Design of Large Earth and Rockfill Dams, para 4.2.3. deals about core thickness. It has been mentioned that the practical considerations which shall govern the thickness of the core are:

a) Availability of suitable impervious material;

b) Resistance to piping;

c) Permissible seepage through the dam; and

d) Availability of other materials for casing, filter, etc.

The core material has normally lower shear strength than the rest of the embankment. Further, being impervious it retains higher pore pressures which further reduce its stability. It has to be compacted in thinner layers with fairly accurate moisture control and requires adequate number of roller passes [121].

From the standpoint of stability, the thinner the core the better it is, but the higher is the demand for suitable properties of the core fill and adjoining dam fill. A thin core also requires very strict construction control [31]. However, from standpoint of crack formations and earthquake consideration the thicker the core the better it is [89]. Also, the use of larger thickness would only be justified where a large quantity of impervious material may be available or where pervious material may not be available within economic distance [121]. The compromise between the two is the solution for optimal core thickness. The minimum top width of the core should not less than 3.0 m.

2.3.1. Factors Governing The Core Thickness

Generally, the thickness of the core is governed by the following factors:

1. Permissible Seepage Losses

Seepage losses depend on the permeability of the material. The permissible seepage losses vary depending on the functions of reservoir.

2. Type of Material Available

If the available materials have a high erosion resistance as well as good flexibility, smaller thickness can be used than otherwise.

3. Design of Transition or Filter Zones

For a given material type, the core thickness can be kept less if the filter or transition zone fully meets the filter specifications and is of adequate thickness.

4. Permissible Crack

The thinner the core, the greater the is likelihood of a through crack forming in it. In a thicker core it is more likely that crack would penetrate only part of the thickness.

5. Seismic Zone Condition

A larger thickness has to be used in seismic areas where there is greater likelihood of cracking.

6. Hydraulic Gradient

The hydraulic gradient relative to the core is the ratio of maximum head of water that is retained in the reservoir to the thickness of the core. The maximum hydraulic gradients H/B (maximum head/core thickness) permitted in some of the high embankments are given in Tabel 2.1 [121 and 132].

Table 2.1. Hydraulic Gradients in the Cores

Dam	*Type of Core*	*Country*	*Height (m)*	*Approximate Hydraulic Gradient (H/B)*
Aswan	Vertical	Egypt	110	2
Blowering	Vertical	Australia	112	1
Blue Mesa	Vertical	U. S. A.	92	1
Chivor	Inclined	Colombia	238	3
Copeton	Vertical	Australia	109	1.2
Darmouth	Vertical	Australia	183	1.4
Holjes	Inclined	Norway	81	2
Jindabyne	Inclined	Australia	72	3
Kajakai	Vertical	Afghanistan	100	2
Kennei	Inclined	U. S. A	100	3
Llyn Brianne	Vertical	U. K.	90	3
Mica	Inclined	Canada	235	3
Mont Cenis	Inclined	France	81	2
Nantahala	Inclined	U. S. A	80	9
New Don Pedro	Vertical	U. S. A	180	2
Ord River	Vertical	Australia	98	2.5
Sayansk	Vertical	CIS	225	5
Talbingo	Slightly Inclined	Australia	162	1.3

2.3.2. Usually Adopted Core Thickness

Data in Table 2.1 reveals that the core thickness usually adopted varies between 0.5 and 0.33H. Such cores have proved satisfactory. Cores with base thickness of 0.15 to 0.20H are usually considered thin and are rarely used. Other cores with base thickness less than half the dam height are also considered thin.

Cores with base thickness less than the dam height and more than half the height are considered moderately thick. When the cores base thickness is more than the dam height, the core is considered thick [121, page 148 and 441].

A guide to core thickness [121] may be obtained as :

$$B/H = (\gamma_{\omega} \times t) / (2\,\tau_{l})$$

Where : B = Core Thickness (metre)
H = Head of water (metre)
γ_{ω} = The unit weight of water (g/cm^3)
t = Thickness of crack (measured upper range 2 to 2.5 cm).
τ_{l} = The limiting shear stress (may taken as 3 to 5 times the critical value at which erosion begins), in kg/cm^2.

2.4. Core Material

The materials that comprise the cores of dams are widely diversified, from silts and clays to fine granular soils with some silt, to coarse-grained soils, such as glacial till or moraines, which contain an appreciable percentage of gravel and cobble-size material. The particulars of some of the cores actually provided in important embankment dams are tabulated in Table 2.2.

The embankment dams for economy have to be designed to utilize materials available within a relatively short distance of the dam site. These are to be investigated for available quantity and relevant soil properties such as its grain size distribution, Atterberg limit, etc.

Average properties of soils according to the Unified Soil Classification have been tabulated by U.S.B.R (1968, 1982) on the basis of a large number of tests carried out in their laboratories. These are given in Table 2.3 (Engineering for Embankment Dams by Bharat Singh and R.S. Varshney [121] - Table 4.1).

Table 2.2. Some Important Details of the Core of some important Embankment Dams.

No.	*Dam*	*Country*	*Height (m)*	*Type of Core Material*	*Incli-nation*	*U/S Slope (H:V)*	*D/S Slope (H:V)*
1.	Nurek	Russia	300	Natural Mixed of Rocky Clay	V	0.25:1	0.25:1
2.	Chicoasen	Mexico	261	Clay-Gravel Mixtures	V	0.35:1	0.2:1 (vertical in nearly 60m lower part)
3.	Tehri	India	261	Blended Material (Clay with Pebbles)	Vi	0.5, 0.4 & 0.3:1*	-0.1, 0.05 & 0.1:1*
4.	Mica	Canada	244	Glacial Till	Vi	0.4:1	-0.1:1
5.	Oroville	U.S.A	235	Cobble, Gravel + Clayey Sand	Vi	0.9:1	-0.5:1
6.	Keban	Turkey	207	Clay	V	0.16:1	0.16:1
7.	Gepatch	Austria	153	Nature Moraine Clay (+ 1% Bentonite in u/s portion)	V	0.25:1	0.25:1
8.	Infernillo	Mexico	148	Sandy Clayey Silt Medium Plasticity	V	0.0891:1	0.0891:1
9.	Tarbela	Pakistan	145	Gravel, Sand Silt mix	I	1.8:1	-1.22:1
10.	Ambuklao	Phili-pine	131	Quarried Loam	V	0.25:1	0.25:1
11.	Miboro	Japan	130	Disintegrated Granite + Clayey Soil	I	1.5:1	-0.85:1
12.	Ramganga	India	126	Crushed Clay shale	V	0.25:1	0.25:1

Notes for Table 2.2:

* : From upper part to lower part respectively.

V = Vertical (Central)

Vi = Moderately Inclined

I = Inclined

(-) = Sloping upstream or negative downstream.

Table 2.3. Average Properties of Soils (After U.S.B.R)

Soil classification group	Proctor compaction: Maximum dry density in t/m³	Optimum water content, %	Void ratio, e_0	Permeability, k 10^{-6} cm/s	Compressibility, %: at 1.4 kg/cm²	at 3.5 kg/cm²	Shearing strength: C_0, kg/cm²	C_{sat}, kg/cm²	tan ϕ
GW	>1.90	<13.3	(.)	(.026 ± .013)	<1.4	(.)	(.)	(.)	>0.79
GP	>1.76	<12.4	(.)	(0.064 ± 0.033)	<0.8	(.)	(.)	(.)	>0.74
GM	>1.82	<14.5	(.)	>0.3	<1.2	<3.0	(.)	(.)	>0.67
GC	>1.84	<14.7	(.)	>0.3	<1.2	<2.4	(.)	(.)	>0.60
SW	1.90 ± 0.08	13.3 ± 2.5	0.37 ± (.)	(.)	1.4 ± (.)	(.)	0.4 ± 0.04	(.)	0.79 ± 0.02
SP	1.76 ± 0.03	12.4 ± 1.0	0.50 ± 0.03	>15	0.8 ± 0.3	(.)	0.23 ± 0.06	(.)	0.74 ± 0.02
SM	1.82 ± 0.016	14.5 ± 0.4	0.48 ± 0.02	7.3 ± 4.7	1.2 ± 0.1	3.0 ± 0.4	0.52 ± 0.06	0.20 ± 0.07	0.67 ± 0.02
SM-SC	1.90 ± 0.016	12.8 ± 0.5	0.41 ± 0.02	0.8 ± 0.6	1.4 ± 0.3	2.9 ± 1.0	0.51 ± 0.22	0.15 ± 0.06	0.66 ± 0.07
SC	1.84 ± 0.016	14.7 ± 0.4	0.48 ± 0.01	0.3 ± 0.2	1.2 ± 0.2	2.4 ± 0.5	0.76 ± 0.15	0.11 ± 0.06	0.60 ± 0.07
ML	1.65 ± 0.016	19.2 ± 0.7	0.63 ± 0.02	0.57 ± 0.22	1.5 ± 0.2	2.6 ± 0.3	0.68 ± 0.10	0.09 ± (.)	0.62 ± 0.04
ML-CL	1.74 ± 0.03	16.8 ± 0.7	0.54 ± 0.03	0.125 ± 0.07	1.0 ± 0.2	2.2 ± 0.0	0.64 ± 0.17	0.22 ± (.)	0.62 ± 0.06
CL	1.73 ± 0.016	17.3 ± 0.3	0.56 ± 0.01	0.08 ± 0.03	1.4 ± 0.2	2.6 ± 0.4	0.88 ± 0.10	0.13 ± 0.02	0.54 ± 0.04
OL	(.)	(.)	(.)	(.)	(.)	(.)	(.)	(.)	(.)
MH	1.31 ± 0.06	36.3 ± 3.2	1.15 ± 0.12	0.16 ± 0.10	2.0 ± 1.2	3.8 ± 0.8	0.74 ± 0.30	0.20 ± 0.09	0.47 ± 0.05
CH	1.50 ± 0.03	25.5 ± 1.2	0.80 ± 0.04	0.05 ± 0.05	2.6 ± 1.3	3.9 ± 1.5	1.04 ± 0.34	0.11 ± 0.06	0.35 ± 0.09
OH	(.)	(.)	(.)	(.)	(.)	(.)	(.)	(.)	(.)

The ± entry indicates 90 per cent confidence limits of the average value.
(.) denotes insufficient data, > is greater than, < is less than.
C_0—effective stress value for samples compacted at Proctor's dry density.
C_{sat}—effective stress value for samples compacted at Proctor's dry density and saturated before testing.

As per I.S code 12169-1987, the suitability of soil for construction of cores and shell of a dams is given in Table 2.4.

Table 2.4. Suitability of Soil for Construction of Dams

No.	*Relative Suitability*	*Homogeneous Dam*	*Impervious Core*	*Pervious Shell*	*Impervious Blanket*
1.	Very Suitable	GC	GC	SW, GW	GC
2.	Suitable	CL – CI	CL – CI	GM	CL , CI
3.	Fairly Suitable	SP, SM CH	GM , GC SM, SC, CH	SP , GP -	CH, SM SC, GC
4.	Poor	-	MI, MI, MH	-	-
5.	Not Suitable	-	OL, OI, OH	-	-

2.4.1. Size of Material

The details of Table 2.2 show that in major dams the core material used is clay blended usually with sand and gravel. The thickness of the lifts for the core material in some major dams has varied from 6 in.(15 cm) to 20 in.(50 cm). The maximum allowable particle size for the core is usually specified to range ½ to 2/3 the maximum lift thickness. The maximum size of particles used in the core of some dams is given in Table 2.5 [121 and 150].

Table 2.5. : Maximum size of particles in the core of some dams.

No.	Dam	Height (m)	Country	Core Material Description	Maximum Size (mm)
1.	Lower Cabin Creek	30	U.S.A.	20% Finer than 0.074 mm	50
2.	Mattmark	120	Switzerland	40% Finer than 0.1mm	120
3.	Talbingo	160	Australia	The tests were showed that core clay without these larger stones (up to 115 mm) would have been quite unrealistic [132, page 404].	115
4.	Darmouth	183	Australia	50% Finer than 4.75mm, and of it not less than 15% shall pass 0.074mm.	120

2.4.2. *Permeability, Erosion Resistance, Compacted Density, Flexibility, Compressibility, Moisture Content and Plasticity*

The permeability of the compacted core material should not exceed 1 x 10^{-5} cm/sec. Various types of materials with a permeability of 1 x 10^{-5} cm/sec or lower have been used in earth-dam cores. Clays containing organic material and peat soils shall be avoided [121 and 132]. In order to obtain required permeability, the core material should be well compacted, and should have high density with natural moisture content. Clays with plasticity index (P.I.) ranging between 15 and should be used. But plastic clays (P.I. > 50) should be avoided because these may crack under high pressure.

The core particles should withstand the erosive action of water passing through the core. Erosion resistance is mainly derived from two sources-cohesion of the fines and the resistive action of coarse particles to the flowing water and their tendency to wedge up in the leakage channel. This effect is best obtained in a well-graded sand-gravel mixture with enough finer particles to provide imperviousness. In high plastic clays, on the other hand resistance to erosion is provided by strong inter particle adhesion.

Sherard has graded materials according to their erosion resistance as given in Table 2.6.

Table 2.6. Classification of core materials on the basis of resistance to concentrated leak (after Sherard, 1967, and IS 8826-1978)

No.	Description
1.	Very Good Materials Very well-graded coarse mixture of sand, gravel, and fines. D_{85} coarser than 50 mm, D50 coarser than 6 mm, if fines are cohesionless, not more than 20% finer than the 75μ size.
2.	Good Materials. a. Well-graded mixture of sand, gravel, and clayey fines. D_{85} coarser than 25 mm. Fines consisting of inorganic clay (CL) with plasticity index greater than 12. b. Highly plastic tough clay (CH) with plasticity index more than 20.
3.	Fair Materials. a. Fairly well-graded, gravelly, medium to coarse sand with cohesionless fines. D_{85} coarser than 19 mm. D_{50} between 0.5 mm and 3.0 mm not more than 25% finer than the 75μ size. b. Clay medium plasticity (CL) with plasticity index greater 12.
4.	Poor Material. a. Clay of low plasticity (CL and CL-ML) with little coarse fraction. Plasticity index between 5 and 8. Liquid limit greater than 25. b. Silts of medium to high plasticity (ML or MH) with little coarse fraction. Plasticity index greater than 10. c. Medium sand with cohesionless fines.
5.	Very Poor Materials. a. Fine, Uniform, cohesionless silty sand. D85 finer than 0.3mm. b. Silt from medium plasticity to cohesionless (ML). Plasticity index less than 10.

Experimental investigation carried out at the WRDTC, Roorkee (B.Venkatesh, 1973, Bony 1974) for fine grained soils indicated that erosion resistance normally increases with plasticity index except that for highly plastic, expansive, organic clays in which it might be lower than for well-graded soils of lower plasticity [121].

Blending with bentonite was found to improve erosion resistance in all cases but markedly in the case of well-graded soils. Blending or mixing of ingredients in dam improves properties of materials, but it increases construction cost. For very high dams, the possibility of improving erosion resistance of the core materials by blending a small percentage of bentonite and / or by addition of coarse graded fractions should be considered where satisfactory core materials is not available.

In case of erosion resistance Bharat Singh [121] has made the following recommendations:

(i). The rate of erosions decreases with increasing plasticity index (P.I) up to value of 15, after which its influence is small. Higher compacted density reduces the rate of erosion.

(ii). The addition of bentonite significantly improves erosion resistance, particularly in well graded soils. The addition or inclusion of stone chips to the extent of 10 to 20% also improves erosion resistance.

(iii). An attempt should be made to select soil with a P.I between 15 and 20%. Stones up to 10 to 20% should be included if found in deposit, the maximum size being limited by compaction thickness. The soil should be compacted to high density.

The requirements of erosion resistance and flexibility are conflicting, thus the best category of material for erosion resistance (Sherard criteria Table 2.6) may not be equally good so far as flexibility is concerned. Similarly well compacted high density fill is good for erosion resistance but not flexibility.

The core material, its properties along with other characteristics for a number of high embankment dams are tabulated in Table 2.7. [121].

Non-cohesive granular material can not retain open cracks but such material usually are too pervious to be used in the core. According to a study made by *Sherard at al*, inorganic clays of low to medium plasticity, P.I < 15, with grain size variation from 0.002 mm to about 1.20 mm or so were comparatively more susceptible to cracking [121].

Flexibility does increase with an increase in P.I., however , very high values may again be associated with high compressibility. Placement moisture content also plays an important role. *Rajcevic* [132] has suggested that at least three values of moisture content for plasticity might be considered for placement as shown in Figure 2.3.

Fig.2.7. Impervious Core and Properties of Core Material

Sl. No.	Dam	Height of dam, m	Country	Impervious material	Core: Incli-nation (a)	Core: Slopes u.s.	Core: Slopes d.s.	Maximum hydraulic gradient	Perme-ability, cm/s	ϕ, degree	Cohesion c, kg/cm²	Compacted dry unit weight, t/m³	Moisture content, %	Compaction: Layer thick-ness, cm	Compaction: Method (b)
1.	Rogun	335	Russia	Processed natural loam and pebbles	I	0.9 : 1	0.4 : 1							30	50t - R.T.R.
2.	Boruca	302	Costa Rica	Alluvial deposits of silt + sand + gravel	I	0.65 : 1	0.25 : 1	2 : 1							
3.	Nurek	300	Russia	Loess, Loamy soil	V	0.25 : 1	0.25 : 1	1.9 : 1	2×10^{-8}			1.750	13-15	20	50t - R.T.R.
4.	Chicoasen	261	Mexico	Homogenization of available clay shale	Vi	0.35 : 1	Vertical			24–31	0	1.900			
5.	Tehri	261	India	Blended material	Vi	0.5 : 1	0.2 : 1								
6.	Mica	242	Canada	Glacial till	Vi	0.4 : 1	0.1 : 1	3 : 1	1×10^{-7}	34	0	1.380		25	R.T.R.
7.	Oroville	235	USA	Cobble, gravel + clayey sand	Vi	0.9 : 1	0.5 : 1	2 : 1		14	0.3	2.280		25	4 Cov, 100t - R.T.R.
8.	Keban	207	Turkey	Clay	V	0.16 : 1	0.16 : 1								
9.	Portage Mountain	183	Canada	Silty sand	Vi	1 : 1	0.1 : 1	1.1 : 1	1×10^{-7}		0–3.5				
10.	Talbingo	167	Australia	Landslide material of basaltic origin	I	1 : 1	0.2 : 1	1.3 : 1							
11.	Grand Maison	160	France	Crystalline gneiss + schist	V	0.16 : 1	0.16 : 1								
12.	Goscheneralp	155	Switzerland	Alluvium + clay	V	0.16 : 1	0.16 : 1	3.1 : 1	5×10^{-7}	30–34		2.150	6	30	50t - RTR
13.	Gepatsch	155	Austria	Residual schist, gneiss (1% bentonite)	V	0.25 : 1	0.25 : 1	3.7 : 1	1×10^{-9}	29	0	2.100	8	30	3 Cov; 40t - RTR
14.	Charvak	154	Russia	Loam	V	0.25 : 1	0.25 : 1	1.8 : 1						33	5 Cov, heavy RTR
15.	Infiernillo	148	Mexico	Clay	V	0.089 : 1	0.089 : 1	5.5 : 1	5×10^{-8}	12-14	0.35–0.5	1.586	23	15	15t - SFR
16.	Tarbela	147	Pakistan	Gravel-sand-silt	I	1.8 : 1	1.22 : 1	4 : 1	10^{-6}						
17.	Derbendi Khan	137	Iraq	Loamy sand	V	0.3 : 1	0.3 : 1	1.3 : 1		13	1.1	1.870	14.5	30–15	2 Cov, RTR or SFR
18.	Cougar	136	USA	Basalt talus + sandy silt	Vi	0.45 : 1	0.25 : 1	3.7 : 1	2×10^{-7}			1.900	15.3	30	2 Cov, 50t - RTR
19.	Round Butte	134	USA	Silty sand	Vi	0.38 : 1	0.087 : 1	2.9 : 1						30	50t - RTR
20.	Ambuklao	131	Philippines	Clay	V	0.25 : 1	0.25 : 1	1.9 : 1		15	2.5			45	2 Cov; 50t - RTR
21.	Miboro	130	Japan	Disintegrated granite + clayey soil	I	0.7 : 1	0.85 : 1	1.2 : 1	1×10^{-5}	35–40	2.8	2.050	7–4	20	6 Cov; 20t - SFR

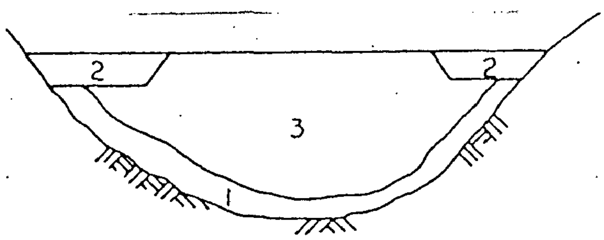
Contd.

Fig.2.7. Impervious Core and Properties of Core Material *(Continued)*

Sl. No.	Dam	Height of dam, m	Country	Impervious material	Core: Inclination (a)	Core: Slopes u.s.	Core: Slopes d.s.	Maximum hydraulic gradient	Permeability, cm/s	ϕ, degree	Cohesion c, kg/cm²	Compacted dry unit weight, t/m³	Moisture content, %	Compaction: Layer thickness, cm	Compaction: Method (b)
22.	Netzahualcoyotl	128	Mexico	Weathered conglomerate	V	0.2 : 1	0.2 : 1	2 : 1		22	0–2	1.750	16–28	20	3 Cov; 4t - VR
23.	Furnas	127	Brazil	Sandy and silty clay	I	0.7 : 1	0.3 : 1	3 : 1		15	1.0	1.677	19.2	30	50t - RTR
24.	Ramganga	126	India	Crushed clay shale	V	0.24 : 1	0.24 : 1	2 : 1	8×10^{-7}	22	0.98	1.760		15	
25.	Mangla	125	Pakistan	Clay	Vi	0.7 : 1	0.2 : 1	1.7 : 1							
26.	Angat	125	Philippines	Clay	I	1.5 : 1	1 : 1	7 : 1							
27.	Serre Ponçon	125	France	Silty clay	V	0.2 : 1	0.2 : 1		1×10^{-7}						
28.	Brownlee	122	U.S.A.	Clay	I	1.5 : 1	1.37 : 1	7 : 1	9.7×10^{-6}	18	5.0	1.760	20–25	15	6–8 Cov SFR
29.	Trangslet	122	Sweden	Moraine	I	2 : 1	1.3 : 1	2.7 : 1	1×10^{-7}	45	0	2.050	8	30	3 Cov; 40t - RTR
30.	Mud Mountain	122	USA	Sand, gravel + silty till	V	0.15 : 1	0.15 : 1	2.6 : 1						15	
31.	Mont Cenis	120	France	Natural earth	I	1.3 : 1	0.3 : 1					2.200	7.9	40	6 Cov; 8t - VR
32.	Akosombo	113	Ghana	Non-plastic red silty clay	V	0.15 : 1	0.15 : 1			16		1.955	13		6 Cov; 50t - RTR

Note: (a) V = vertical core; I = inclined core; Vi = slightly inclined core which may be considered as vertical core.

(b) Cov = coverage (number of roller passes); RTR = rubber typed roller; SFR = sheep foot roller; VR = vibratory roller.



1 Moisture content 10 percent above plasticity limit
2 Moisture content 3-6 percent above plasticity limit
3 Earth material on plasticity limit

Fig. 2.3 : Core Plasticity (After *Rajcevic*)

He has recommended high plasticity in foundation cover, a little above the plastic limit towards the crest level near each abutment where cracking frequently occurs, and soil at the plastic limit for the balance of core. However *Rajcevic's* recommendations have to be considered vis-a-vis the requirements of compaction and development of construction pore pressures [132].

Vallespir [139], has suggested that the core materials shall be compacted with slight variations in their placement conditions due to the possible distortions which could be suffered by the core at its crest as shown in Fig.2.4. This may bring about cracking as shown.

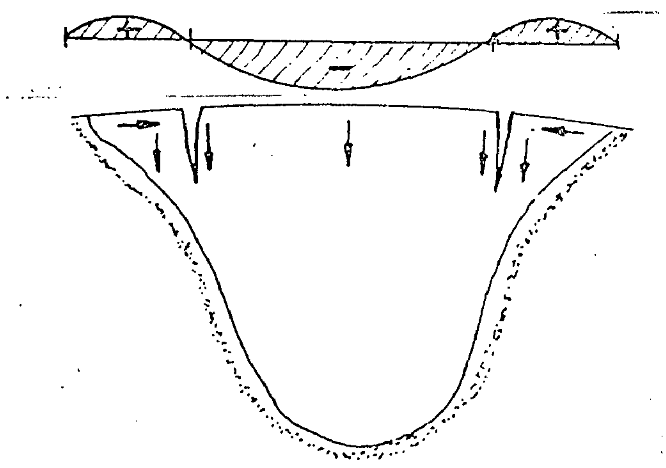


Fig.2.4. Possible Core Distortions

He has pointed out 6 zones to be compacted using slight variations as shown in the Fig.2.5.

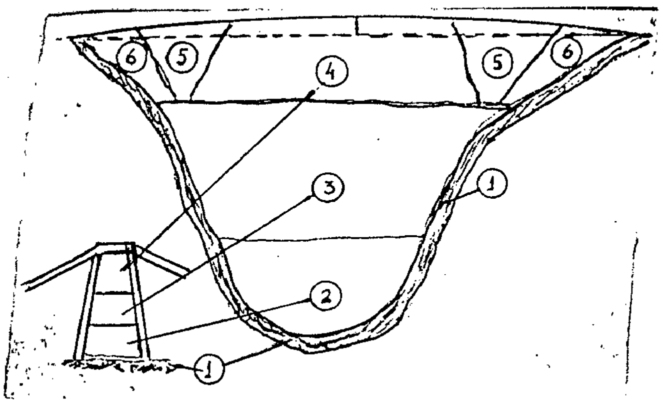


Fig.2.5. Zones in which different Core Materials are distributed

In zone 1 the material must have higher water content than the rest of the core, and it should be compacted with manual equipment in thin layers of about 7 cm.

In zone 2, 3 and 4 plasticity must be varied, but only slightly, from a lower degree in zone 2 to a higher one in zone 4. The material's water content will generally be less than optimum in zone 2, the same in zone 3 and a little higher in zone 4. Densities will be the opposite, higher in zone 2, average in zone 3, and slightly less than optimum in zone 4.

In zones 5, material will have slightly higher plasticity. In zone 6, material ranging between zones 4 and 5 shall be placed.

In all zones, compaction of the core must be carried out so that the lamination phenomenon does not take place, and a good adherence should be obtained between compacted layers.

2.4.3. Control of Water / Moisture Content, Dry Density and Compactive Effort.

Generally, two limiting moisture contents exist for any particular core material; the lower limit is that which will not result in additional settlement of the core upon saturation; the upper limit is that which makes the material difficult to work with during placement and compaction. Between these limiting values, a moisture content can be selected that takes into account both design requirements and economic considerations [150].

Generally a dry density, obtained in the Standard Proctor test, is specified for an embankment (see Fig.2.6), and the same can be obtained at a higher or lower moisture content, though with a higher compacting effort. If an embankment is placed at less than optimum, the fill is rigid and stiff and is liable to cracking but construction pore pressure are low. On the other hand, placement of fill at higher than optimum makes it more plastic than placed on moisture content less than optimum. Thus it is a more desirable core material since it is less susceptible to cracking, and has lesser compressibility on saturation than for that which is compacted at less than optimum to the same dry density. Compaction at a moisture content higher than optimum may require flattening of dam slopes for stability or slowing down the rate of construction, especially in dams with thick cores. Flexibility, however, is a definite advantage as possible leakage through cracks is a major concern for the designer, particularly for high dams in narrow valleys with steep abutment slopes [121 and 109].

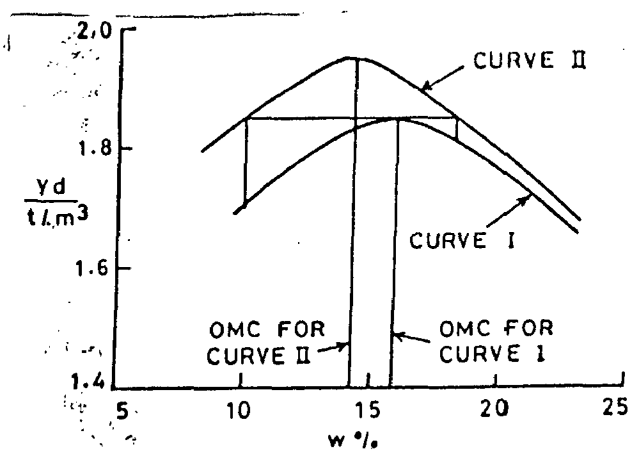


Fig.2.6. Variation of OMC with Compactive Effort.

The experience of the USBR indicates that by placing impervious materials at an average water content between 1 and 3% below Standard Proctor Optimum, the pore pressures can be kept to reasonable values. These values are usually limited to a maximum of about 30% of the weight of the overlying embankment.

Their experience over the last 15 years has demonstrated that, at least in the semiarid geographical areas where most of their dams are built, it is practicable to hold construction pore pressures to tolerable maximum values. In the last 10 years many other major dams in addition to those of the USBR have been built with low water contents to reduce pore pressures [113 and 121].

The Army Corps of Engineers, are so concerned about this danger of cracking that they prefer to compact the core at or above optimum water content. [113].

The current trend is that the cores of major zoned-earth and rockfill dams are placed and compacted at or more than the optimum moisture contents. For example, at Netzahualcoyotl Dam in Mexico, the core was placed at an average moisture content well above optimum, that is about 7 percent. The core material was silt (ML to MH), derived from the weathering of a weakly cemented conglomerate (Gamboa and Benassini,1967). The cores of Trangslet (Persson, 1964), Summersville (Barnes,1964), Peruca (Nonveiller, 1964), Mont-Cenis (Marchand, et al, 1967) and Cougar (Basgen, 1964) dams were compacted at slightly wet of optimum; at Mangla (Binnie, et al, 1967), Gepatsch (Lauffer and Schober, 1964), Holjes (Reinus, 1964) and Akosombo (Ware and Hooper, 1964), the cores were compacted at moisture content 2 percent above the optimum. At Messaure Dam the wet fill methods was used and the core placed and compacted at about 4 percent above the optimum moisture content (Bernell,1964). This fill method, originally devised in Sweden, allows for the placement and compaction of sandy or silty moraines in a short and wet construction season (Nilson and Lofquist,1955). Exceptions to this range are the earth dams of the Bureau of Reclamation that are constructed mainly throughout the Western United States. The range of compaction moisture contents in these dams usually varies from 0.7 percent wet to 2.5 percent dry of optimum (Esmiol, 1953) and (Collins and Davis, 1958), such as the earth zone of Blue Mesa Dam, a 300 ft high zoned earth dam constructed (1966) in Colorado by the Bureau, was placed and compacted with an average moisture content 1.2 percent dry of optimum (Collins,1964) [150].

Placing core material at moisture content more or less than optimum has an effect on compactive effort requirement. But an increase in compactive effort is more effective in increasing the density of the soil when its water content is on the dry side of the optimum than when on the wet side [113].

2.4.4. *Shear Strength*

Shear strength of a soil resists shearing stress which developes in the core at a particular plane or surface.

Since different conditions of consolidation, drainage, and loading exist in an embankment dam, different types of test procedures are required to simulate these conditions in the laboratory. The main types of shear test used in the stability analysis of embankment dams are: unconsolidated undrained or quick (UU or Q) test, consolidated undrained or rapid (CU or R) test, and consolidated drained or slow (CD or S) test.

Shear strengths obtained from UU or Q, and CU or R tests may be expressed in terms of total stress σ acting on the failure plane, or the effective stress σ' = $(\sigma - u)$, where u is the pore pressure at the time of failure. These two methods are respectively known as the total stress and the effective stress methods, and the shear strength parameters obtained in the two cases are express as c, Φ, *and* c', Φ'. The effective stress tests are carried out with pore pressure measurement and the stress strain curve is plotted in tems of σ_1'/σ_3' versus strain [109].

2.4.5. *Pore Water Pressure*

The development of high pore pressures between different zones of an embankment dam must be considered in the analysis and design. Excessive dam slope deformations or even stability failures during construction may be caused due to high pore pressures [18].

The magnitude of these construction pore pressures depends on the placement water content, compressibility and permeability of fill material, overburden pressure, rate of construction and gaps in construction and the conditions of drainage. For given conditions of loading and compressibility, the closer the compacted soil is to saturation, the higher will be the pore pressures[109].

Some important observations taken from Sherard [113] about pore water pressure are as below:

a) A high dam constructed of almost any type of soil with an average water content near Standard Proctor Optimum water content will develop construction pore pressures.

b) Embankments of well-graded, clayey sands and sand-gravel-clay mixtures have the highest construction pore pressures, and those of uniform silts and fine, silty sands are the least susceptible to the development of pore pressure.

c) Pore pressures which have developed in embankment with the highest contents of clayey fines, however, appeared to be lower than predicted values.

d) For thick core of very impervious material, the pore pressures at the center usually are not reduced appreciably by drainage during the time of typical construction periods, but are always reduced at the outside edges.

e) The relationship between the coefficient of permeability of the impervious core and development and dissipation of construction pore pressures is not well known. But rough estimate has been made as shown in Table 2.8.

Table. 2.8 Estimate of the relationship between the coefficient of permeability of relatively thick impervious cores and dissipation of pore pressures in the central part of the core.

No.	Coefficient of Permeability	Dissipation of Pore Pressures During Constructions
1.	Less than 0.5 ft./yr. (0.5 x 10^{-6} cm/sec.)	No dissipation of Pore Pressure
2.	0.5 to 5 ft./yr. (0.5 to 5.0 x 10^{-6}cm/sec.)	Some dissipation
3.	5 to 50 ft./yr. (5 to 50 x 10^{-6} cm/sec.)	Appreciable dissipation
4.	More than 50 ft./yr. (50 x 10^{-6} cm/sec.)	Complete dissipation

f) For dams with thin central cores (width at any level equal to 40 percents or less of the water height), appreciable dissipation of construction pore pressure occurs in the central part of the core.

At the end of construction or during construction condition, pore pressures may be critical for either the upstream or downstream slopes.

After reservoir has been filled for sufficient time and a steady seepage condition has developed, it is critical for downstream slope.

After steady seepage has been established, and the reservoir level is lowered suddenly or so rapidly that the drainage of the upstream shell of the dam cannot keep pace, the condition of draw-down results. This condition is critical for the upstream slope. It is usual to check the safety of the upstream slope under total draw-down which may occur under some emergency situation [129].

2.4.6. Cracking ,Leakage and Hydraulic Fracturing Problems

The cracks have occurred in many high fill dams, even when these have been well designed and constructed. [149].

There have been a number of well-studied cases in which dams have failed or been damaged by concentrated leaks for no apparent cause. In some of these experiences investigators concluded that differential settlement cracks were the probable cause, even though no cracks were seen on the surface. In these examples it was not determined whether the crack had opened before the reservoir was filled or afterward [114].

Cracks occur in regions of tensile strain resulting from the deformations created by the dead load of the structure, the filling of reservoir, and seismic action. The types most commonly recorded have been near-vertical transverse or longitudinal cracks in the crest, although there is some reported evidence of the occurrence of horizontal cracking through approximately centrally cores placed in several dams. Any transverse cracks extending through the core could lead to progressive erosion and failure. Longitudinal cracks, while are seldom in themselves a cause for major concern, could intersect transverse cracks and thus increase the possibility of hazard from the latter. Horizontal cracks may present the greatest problem in that erosion may progress significantly through them before they are detected [149].

Usually, transverse cracks occur without any vertical displacement between opposite walls of cracks. Also transverse cracks generally have occurred perpendicular to axis of the dam. One of defensive measures against transverse cracking is to construct the section of the upper portion of the dam, susceptible to cracks, after the rest of the dam has been completed and the settlement under most of the weight of the dam has been accomplished [67].

Horizontal cracks through the core may occur if the deformation modulus of the core is much less than of the adjacent shells so that the core tends to be supported by the shells with consequent reduction in the vertical stresses in the core. Adjacent to abutment discontinuities in particular, the vertical stresses in the core may even be locally reduced to the extent that tension is induced. Several reports point out the danger of horizontal cracks developing in the central core type of embankment dams due to hang up of the upper part of the core on the shells [67].

Longitudinal cracks have commonly appeared in the last stages of reservoir filling due to the differences in settlement of the flooded upstream shell, the core, and the downstream shell, especially in dams where the shells were more compressible than the cores. Such cracks may also accompany the slumping which tends to occur at the crest under earthquake action. Longitudinal cracks usually occur at upstream face of the impervious core.

According to Sherard [112], leaks can develop through differential settlements cracks caused by embankment slumping under the shaking action of earthquake, landslides on the abutments, movement of faults, foundation shifting due to shear strain and cracks in the outlet conduits.

Shrinkage cracks develop upon drying out of embankment material having high plasticity. To avoid this type of cracking problem either the high plasticity impervious material should be overlain with a sufficient thickness of non-shrinking material to keep the highly plastic material from drying out, or, non-shrinkage type impervious material should be used [67 and 149].

If the soil is fined-grained, then the leakage resistance is wholly a result of the cohesion of the material. For example, tough clay is highly plastic which can withstand high velocities of flow on its surface without appreciable erosion.

For cores consisting of a mixture of coarse-grained soils and fines, the resistance to leakage may be derived from both the cohesion of the fines and the resisting action of the coarse particles to the flowing water.

Probably the best core material is well-graded sand-gravel mixture with a maximum size of 10 to 15 cm. For a crack 10 cm wide or less, it seems apparent that there would not be much danger of progressive erosion with such material. The larger particles would not move out of place, and the finer particles would move slightly under the action of the leak but would soon become lodged between the large gravels, forming a natural filter which would seal a leak. There are no quantitative guides for evaluating the merits for this purpose of different mixtures of gravel, sand, and fines. As stated above, probably the size of coarsest fraction is very important. Other things being equal [112, p.390], the inherent leakage resistance of a gravel-sand-fine mixture will increase with the square of the D_{85} size or even at a faster rate, especially for the potentially large cracks at the top of the dam.

There is no quantitative way to compare the relative merits of a clay of high plasticity with a well-graded, coarse sand-gravel mixture. Both are good materials, but their ability to resist concentrated leaks derives from completely different action. Selection of core materials on the basis of resistance to concentrated leaks may be guided by Table 2.6. [112].

Hydraulic Fracturing has been defined [109] as the condition leading to creation and propagation of thin physical separation in soil or rock mass due to high fluid pressure. The hydraulic fracturing will develop at the point of least resistance. The hydraulic fracturing has been a major cause of cracking, and sometimes leads to development of concentrated leaks, both in homogeneous and zoned embankments. The hydraulic fracturing is caused by :

a. Creation of zones of low stress due to differential settlements at abrupt changes in foundation profile.

b. Low values of total stress at the some depth to arching in narrow trenches, local depression in the profile and other such reason.
c. The hydraulic separation can occur between soil and adjacent dissimilar materials as concrete or rock (conduit or abutment contact) as soon as the water pressure reaches the same magnitude as the normal stress across the interface..
d. Stress transfer from the core to the shells due to interfacial movements.
e. Injection of fluid through a drill hole at pressure in excess of total stress at a particular depth through pre-existing cracks.

Sherard has presented observations supporting the conclusion that small concentrated leaks by hydraulic fracturing commonly develop in well-designed and constructed dams without being recognized, even in dams not subjected to large differential settlements. Often these occurrences are associated with first filling of the reservoir. In dams where hydraulic fracturing is believed to have been a cause of such leakage, complete failure has been averted only in those dams that had filters capable of preventing erosion of the core material [109 and 114].

Another likely zone for hydraulic fracturing is core-abutment contact, especially when the abutment slopes are steep. It is further aggravated by abrupt change in abutment slope or by a step in the abutment. In general, soil deformation from the abutments towards the center of the valley occurs, which lowers the contact pressure, especially in the upper portion of the dam. Cut-off trenches with steep slopes also result in arching, creating areas of low compressive stress susceptible to hydraulic fracturing [121].

2.4.7. Stress-Strain Relationship Model with FEM Related to Earthquake Consideration.

The behavior of the type of soil chosen for core material and the stress and deformation of the dam and its foundation can be made and predicted by FEM (Finite Element Method) analysis. The behavior under seismic forces can also be realistically predicted.

The predicted behavior is often dependent on the choices of stress-strain relationships. It can be linear, multi linear, hyperbolic, elastoplastic, and elastoviscoplastic. The elatoplastic and elastoviscoplastic stress-strain relationships have the advantage that they model more realistically the behavior of soils close to failure, at failure, and after failure. They have the limitation that they are more complex. Finite Element Method (FEM) offers a solution to the problem. One of the major advantages of the FEM is that the loadings and soil properties can be simulated in the analysis [28 and 141].

For embankment dams located in seismically active areas, it is essential that these are designed with anticipated earthquake forces. The static as well as dynamic response of the embankment dams due to earthquake influenced by the components of the dams and topography of the site can be predicted by FEM analysis [44].

The most frequent type of damage produced in dams by earthquake motions are longitudinal cracks, which develop parallel to the crest, and transverse cracks, which run transverse to the dam.

Longitudinal cracks develop near the crest, ordinarily on the upstream face. The upstream face is most prone to this type of damage due to the submersed upstream shell.

These damages may be predicted and measures may be evolved with the help of FEM analysis. All major dams have been analyzed before construction and these have behaved satisfactorily during severe earthquakes [97].

2.5. Core Position (Geometry)

The core can be located in one of the following three positions defined in para 2.1 :

1) Central (vertical) Core
2) Moderately Inclined (Slanting or Sloping) Core
3) Inclined (Slanting or Sloping) Core

The relatively merits and demerits of vertical and sloping cores are discussed as below.

a. *Inclined (Slanting) Core.*

1) Advantages

i. Downstream rockfill can be placed in advance and laying of filter, core and upstream zone can be taken up later.

ii. Foundation grouting of the core can be carried out while the downstream shell is being placed.

iii. Since a very small part of the slip surface intersects the slanting core, this results in a steeper slope of the downstream shell in steady state condition. However, a larger part of the slip surface for the upstream slope passes through the core material in sudden draw down condition.

iv. In the case of cracking of the core, the inclined core will have a large mass of stable rockfill on the downstream side to support.

2) Disadvantages

i. It may need a flatter slope for upstream face.

ii. For same quantity of material, a core thinner than vertical core will be possible.

iii. Raising of dam will be difficult.

b. *Central (Vertical) Core.*

1) Advantages

i. Provides higher pressure on the contact surface between the core and the foundation.

ii. For given quantity of soils, the central core provides slightly greater thickness than slanting (inclined) core.

iii. Provides better facility for grouting of foundation or contact zone or any cracks in the core if required afterwards, as this can be done through vertical rather than inclined holes.

2) Disadvantages

i. Thick central core with pervious shells will result in a slightly flatter downstream slope of the dam.

ii. The problem of differential settlement between the core and the shell zone may result in cracking parallel to the dam axis.

Other important factors determining core location (geometry) are the contour configuration and geology of the site. It is desirable to locate the top axis of the dam such that it is wedged against converging contours so that any movement of the dam would cause longitudinal compression of the core. Also, the base of the core should be located so as to obtain a watertight contact against as sound and impervious a material as possible.

In the 235 m high Oroville dam (USA) whereas a moderately sloping core was adopted (Fig.2.7), a core block structure was provided to fill the steep-walled river channel beneath the impervious core to avoid an otherwise abrupt change in height of the core as it crossed the channel.

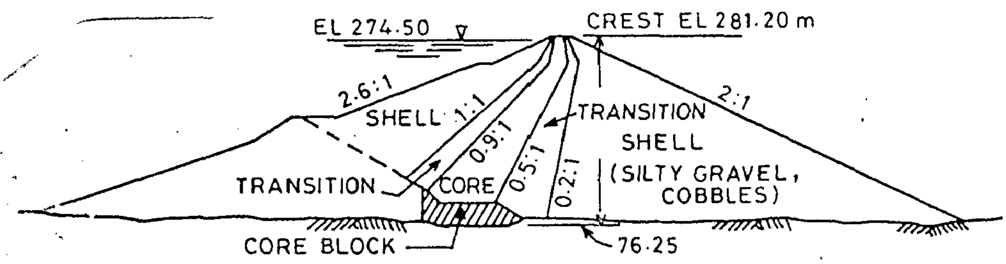


Fig.2.7. Moderately sloping core, Oroville Dam, California (USA)

At Miboro dam in Japan, a sloping core (Fig.2.8) was adopted, as there was a larger body of stable rockfill downstream of the core. The inclination of core was dictated by the requirement of the upstream and downstream slopes of the downstream shell and the consideration that the core can be placed independently.

The 300 m high Nurek dam of Russia (Fig.2.9) is a classical example of dam constructed with central core.

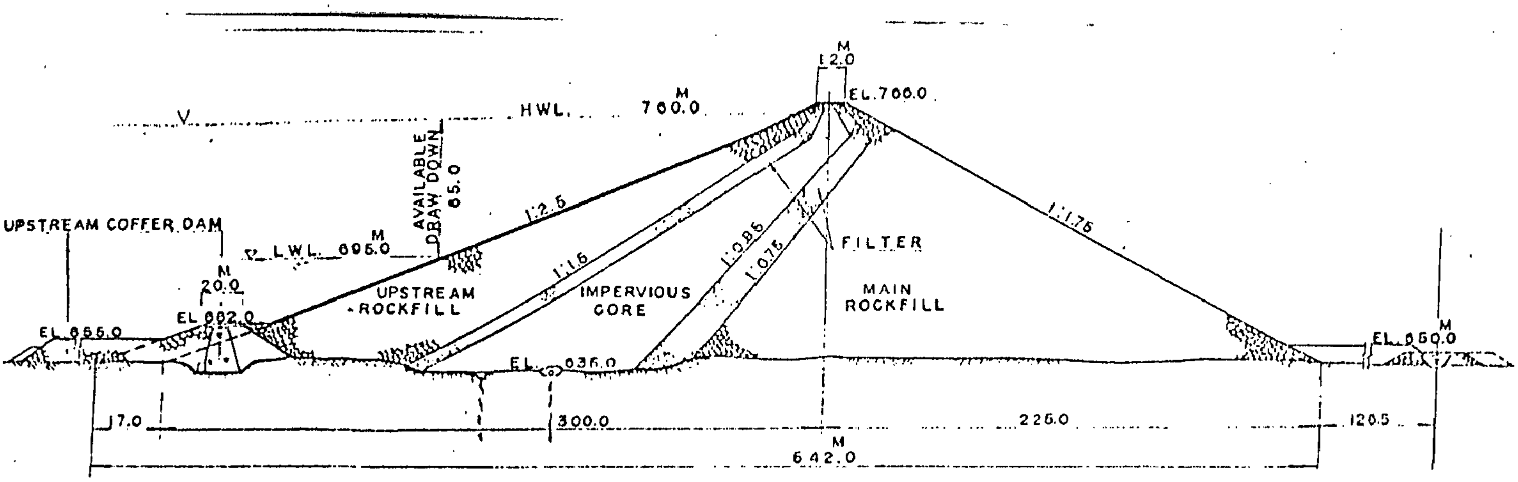


Fig.2.8. Sloping core Miboro Dam, Japan.

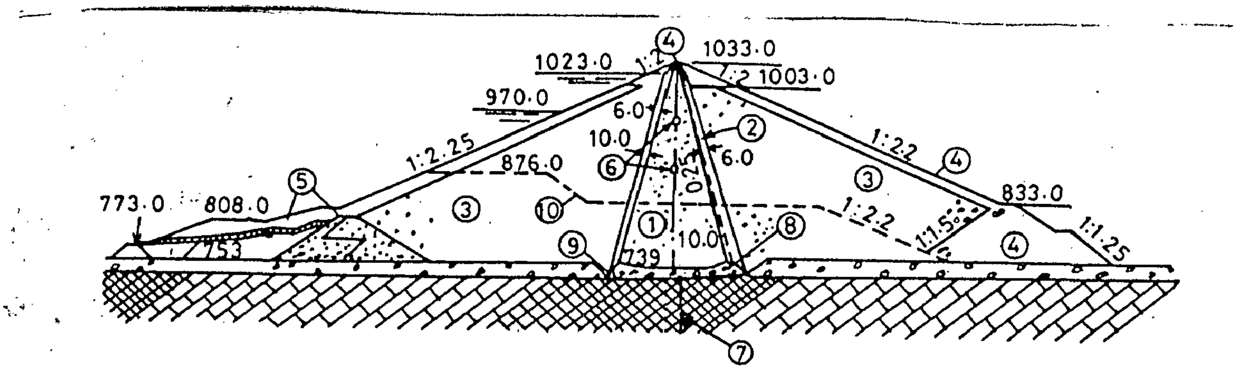


Fig.2.9. Central Core (1) Nurek Dam, Russia

About pressure distribution on the base, Reinus has concluded that [121, page 151], an inclined core results in more advantageous stress and deformation conditions of the fill downstream of the core than a vertical core. However, for a core still flatter than 0.5H:1V, no advantage is gained in respect of stress conditions in the downstream part of the dam.

On the contrary, disadvantages may arise because of differences in the effective vertical foundation pressure between the upstream and downstream side of a thin core resulting in differences in settlements if the foundation is deformable.

Opinions differ regarding suitability of three types of core under seismic loading [121]. Nandakumar and S.S. Saini, et al [102b], has indicated that a dam with a sloping core is less prone to failure due to earthquake and hence is superior to a dam with a central core. In this study it is shown that the dam with a central core would fail along a curved rupture line. Conclusion by Mundal [80] about sloping core type referred to Model tests carried out by Laboratory of University of California indicate that the sloping core is somewhat more earthquake resistant than central core. The highest dam in the world (Rogun Dam in Russia) has a moderately inclined core (Fig.2.10).

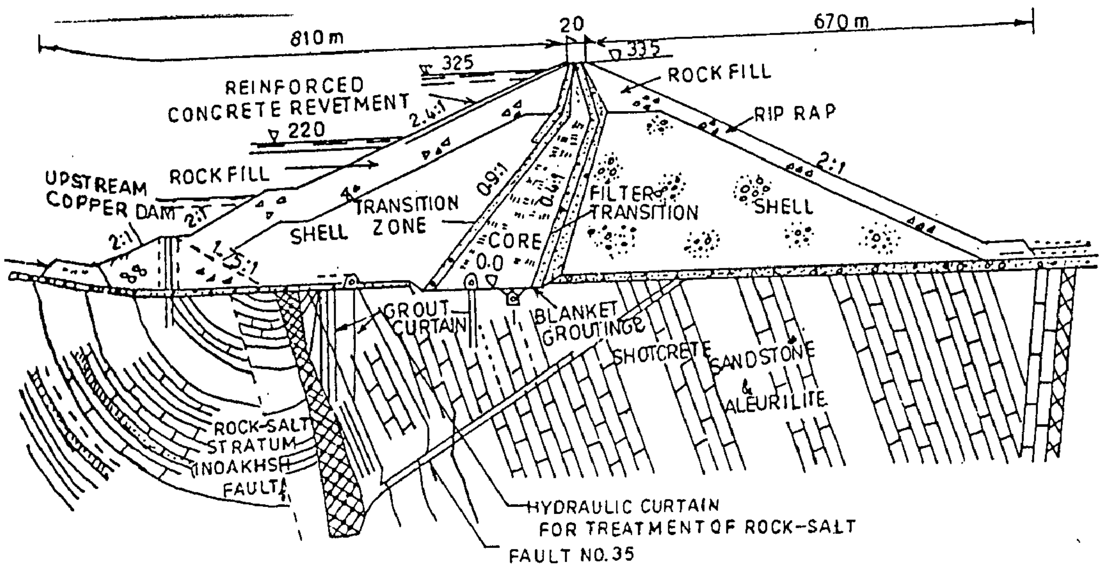


Fig.2.10. Moderately Inclined Core, Rogun Dam (Russia), Typical Cross Section.

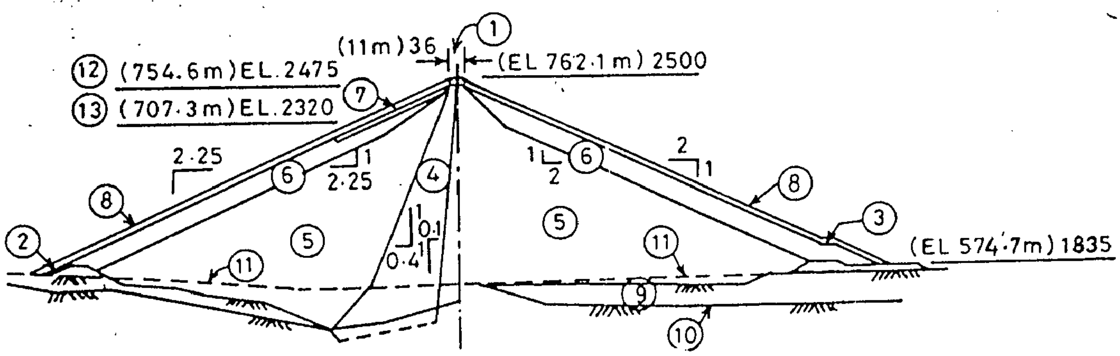


Fig.2.11. Section of a near vertical core (4) of Mica Dam , Canada.

On the other hand, Gupta [44] in his results of static and dynamic analysis has indicated that the central core has a distinct advantage over the inclined cores in seismic areas. Determination of the optimum position (geometry) of the core has been the subject of many studies, including for the core in seismic zone [69, 83 and 94].

At the 244 m high Mica dam in Canada, Webster [149], a near vertical core has been provided to ensure stability and reduction in deformations under an earthquake loading condition (Fig.2.11).

The term 'moderately or gently sloping core' (Sharma [109], page 68-69), is explained as core located at an intermediate position. Among various points in favour of a moderately sloping core, the two main points are: *(a)* in moderately sloping core FEM analysis has shown that it has distinct advantages with respect to arching and hydraulic fracturing over a vertical symmetrical core. The *'optimum position of the core* in this respect, as defined by the inclination of the upstream core-shell interface, *lies between 0.5H:1V and 0.6H:1V* [69]; *(b)* FEM study for a 150 m high earth core rockfill dam with three locations of impervious core: flat sloping core (upstream slope of 1.6H:1V, downstream slope of 1.3H:1V), moderately sloping core (upstream slope of 0.9H:1V, downstream slope of 0.6H:1V), and central core with both side slopes 0.15H:1V, revealed that under static condition *the moderately sloping core provides the most favorable stress distribution*, whereas under dynamic condition a central core is the most stable when subject to an earthquake with a horizontal acceleration of 0.3g.

It can thus be seen that the opinions differ and studies are inconclusive so far as core position is concerned under seismic loading.

2.6. Core Contact Treatment Design

The contact area between the core and foundations and abutments has to be treated to ensure a leakproof bond and to provide a uniform base on which the fill material can be placed and properly compacted. It is also necessary to treat the foundations to some depth to prevent seepage flow through them just below the contact surface, which could result in erosion of the core material [121].

If the shell were made of a less permeable material than that of the filter, the latter shall extend to the downstream area, in contact with the hillsides, up to the slope protection area, in order to allow an easier drainage of the water which might have run through the core in contact with the hillsides.

Same settlement in the core and shells is desirable but this is practically impossible. In an attempt to achieve it, conditions will improve if transition zones between the core and the filters / and between the latter and the shell shall be provided.

The contact between clay core and abutments requires special care. The abutments should be shaped in such a way that the change in the amount of settlement is gradual. Close control of construction operations, by personnel experienced in this type of work, can not be over emphasized [80].

The core contact area includes the foundation contact for the entire base width of the impervious core, the upstream and downstream filter zones, transitions and the downstream drain. This area is the most important and critical in the foundation treatment of earth-core rockfill dams [109]. The controlling factors are :

a. The rock under the core, including the filling material in faults and joints, must be non-erodible and must be protected from erosion under seepage gradients that will develop under the core.
b. Materials of the core must be prevented from moving down into the foundations.
c. Core contact between the core and the foundation rock surface must remain intact despite distortion that might occur in the dam due to its weight and reservoir loading.

The primary hazards to a high embankment dam are cracking within the core caused by unequal settlement and the development of seepage channels along the contact of the impervious core with the foundation and abutment rock. Either of these defects could lead to failure of dam. It must therefore be ensured that the foundation in the core contact area consists of sound and hard rock reasonably free from joints and fissures which could be the cause of internal erosion.

The objectives are achieved by excavation of the uppermost weathered rock zones to the level of sound rock and by consolidation grouting to reduce the permeability of the rock under the excavated surface. Jointed formation is acceptable, provided the joints do not contain soft materials or clays to an extent that could endanger the stability of the rock [109].

2.7. Filters / Transition Design

Inside a zoned dam where seepage moves from fine material into coarse material, it is possible for the finer particles to be washed into the void of the coarser material. In a zoned dam with an internal core and several upstream and down stream zones of increasingly pervious material, the transition from fine to coarse soil is necessary to prevent piping [113].

While normally the seepage is from core to downstream shell, the flow direction is also towards the upstream after drawdown [121].

2.7.1. Design Criteria [121& 109]

1. <u>Terzaghi's Criteria</u>

This criteria, in use since 1930, for the design of filters for dams and levees and other works, is stated as follows:

a. The 15% size of the filter material, D_{15}, must not be more than 4 or 5 times the 85% size, d_{85}, of the protected soil (to prevent piping), i.e.,

$$(D_{15} \text{ of Filter} / d_{85} \text{ of Protected layer}) \leq 4 \text{ to } 5$$

b. The 15% size of filter material, D_{15}, must be at least 4 or 5 times the 15% size, d_{15}, of the protected soil (to ensure adequate permeability), or (D_{15} of Filter / d_{15} of Protected layer) $\geq$ 4 to 5.

In the above D and d respectively refer to the grain size of the filter and the base soil, and the subscripts 15 and 85 refer to percent finer by weight than the grain size D or d. D_{15} is a representative pore size of the filter and d_{85} is a grain size such that, if the filter retains particles of this size, the whole base soil stabilizes through the quick formation of a self-healing layer. The first criteria safeguards piping, the second thus ensures that the filter is 16 to 25 times more pervious than the protected soil.

Other requirements for a good filter are:

i. Its gradation curve should be approximately parallel to the gradation curve of the protected soil, especially in the finer range.

ii. Filters should not contain more than 5% fines (-0.075 mm) and fines should be cohesionless. This is to ensure that filter remains adequately pervious and does not sustain a crack.

iii. The filler should not have particles larger than 75 mm so as to minimize segregation.

iv. If the base material ranges from gravel (over 10% > 4.75 mm) to silt (over 10% passing 75μ), the base material should be analyzed on the basis of gradation of fraction smaller than 4.75 mm.

2. U.S.B.R. Criteria

In 1955, Karpoff conducted investigations on filter in the U.S.B.R and the criteria evolved is given in Table 2.9. It has been used in U.S.B.R. dams.

The other requirements are the same as in Terzaghi criteria.

Table 2.9. U.S.B.R. Filter Design Criteria

No.	Filter Material Characteristics	Ratio $R_{50} = \frac{D_{50} \text{ of Filter}}{D_{50} \text{ of Base}}$	Ratio $R_{15} = \frac{D_{15} \text{ of Filter}}{D_{15} \text{ of Base}}$
1.	Uniform grain size distribution (uniformity coefficient Cu= D_{60}/D_{10} = 3 to 4	5 to 10	-
2.	Well-graded to poorly graded (non-uniform) sub-rounded grains	12 to 58	12 to 40
3.	Well-graded to poorly graded (non-uniform) angular particles	9 to 30	6 to 18

3. *Sherard's Recommendations Based on Experiments*

Experiments conducted at the Soil Mechanics Laboratory, Midwest National Technical Center, Soil Conservation services, U.S. Department of Agriculture, Lincoln, Nebraska on uniform soils ranging from fine sand to coarse sand (up to 2 mm particle size) as well as non-uniform sandy silts and sandy clays, (D_{85}= 0.1 to 0.6 mm), revealed the following:

1) The filter is successful in its function of arresting particle migration if (D_{15} filter / d_{85} of base) < 9. This shows that Terzaghi's criterion includes a factor of safety of 2 and is conservative.
2) The size of the pore channel which governs permeability is determined by the size of finer filter particles and well represented by D_{15} size.
3) The coefficient of permeability of dense filters is generally in the range of K= 0.2 to 0.6 D^2_{15} with average = 0.35 D^2_{15} (K in cm/s and D_{15} in mm).
4) Since pore size is defined by D_{15} and not D_{50}, the U.S.B.R. criterion in terms of D_{50} is inappropriate.
5) The filter gradation curve need not necessarily be parallel to the base material.
6) Using the same criteria, either angular particles of crushed rock or rounded alluvia particles may be used in filter.

2.7.2. *Filter Thickness and Layers[109, 118 and 121]*

Horizontal filters can easily be placed. Therefore they are placed in thinner layers. The minimum practical thickness for horizontal filter is considered to be 15 cm for sands and 30 cm for gravels. The I.S. Code 9429-1980 recommends a minimum thickness of 1.0 m as desirable from practical considerations.

The minimum thickness of an inclined filter layer is determined from consideration of : (a) filter thickness required for drainage, (b) allowance for intermixing with adjoining zones depending on compaction equipment,

(c) minimum width required for compaction and (d) earthquakes effects. As per I.S. Code 9429-1980 a minimum thickness of 2.0 m is recommended. Where filter materials are scarce, a horizontal filter width of 1.0 to 1.5 m has also been used [121].

According to Sherard [118], upstream filters are made with less width than the downstream filters. The upstream filter is considered a non-critical filter because it is never called upon to act to control a concentrated leak. Also, for an impervious clay core, the amount of water, which seeps out of the core in the upstream direction following draw down is very low. Beside that, the maximum gradient for the seepage in the upstream direction following draw down is less than 1, and the velocity and energy of the seepage entering the upstream filter is too small to erode clay particles and carry them upstream. As part of filter research (referred to Sherard [115 and 118]) at the Soil Conservation Service Soil Mechanics, US Department of Agriculture Laboratory, tests were made with relatively erodible clays of low plasticity (using a gradient of about 20 and very coarse filter). These tests showed clearly that it is not necessary to adhere to filter criteria to upstream non-critical filters [115].

2.8. Summary

1) Core is an important component of a zoned embankment. The safety of a dam is dependent on it.
2) The design parameters of a core are :
   - (i) suitable material,
   - (ii) thickness, and
   - (iii) position in the dam section.

   These are interrelated and depend on the quality and quantity of locally available material.
3) Three important categories of core thickness are thin, moderately thick, and thick. Thickness between 0.3H to 0.5H is generally adopted.

4) Three important categories of core position (geometry) are vertical, moderately sloping, and sloping core. Central (vertical) core type would normally have a symmetrical core or in the central portion of the dam. Sloping core is laid on slope of the sides of about 1.35H:1V. Moderately sloping core is laid in an intermediate position or when its upstream face slope lies between about 0.5H:1V and 1H:1V.
5) The functional requirement of the a core are:
   (i) imperviousness,
   (ii) resistance to cracking,
   (iii) resistance to erosion under seepage head.
6) If the available soil is not suitable, blending is resorted to and moisture adjustment is done by adding water or by drying.
7) Opinions differ on the superiority of inclined core over the central core under seismic loading.
8) The performance of a core is improved by providing adequate filters on both sides and by adequate treatment at core – foundation contact.
9) FEM analysis is helpful in working out stresses and deformations in the core under different loading conditions.

Chapter 3

REVIEW OF LITERATURE OF CORES OF HIGH EMBAKMENT DAMS

3.1. General

The International Commission On Large Dams (ICOLD) has defined an *embankment* dam as 'any dam constructed of excavated materials placed without addition of binding materials other than those inherent in the natural material. The materials are usually obtained at or near the dam site'.

This definition covers both homogeneous dams and those that comprise an impervious (the core) zone with rock support from either side. Embankment dams (maximum head over 50 m) fall under the category of high dams [1, 15, 132 and 150].

This study is limited to the design of core of high embankment dams hence the review of literature is made of high embankment dams with the core i.e. zoned embankments. The different types of cores with different foundation conditions are shown in Fig.3.1 and 3.2.

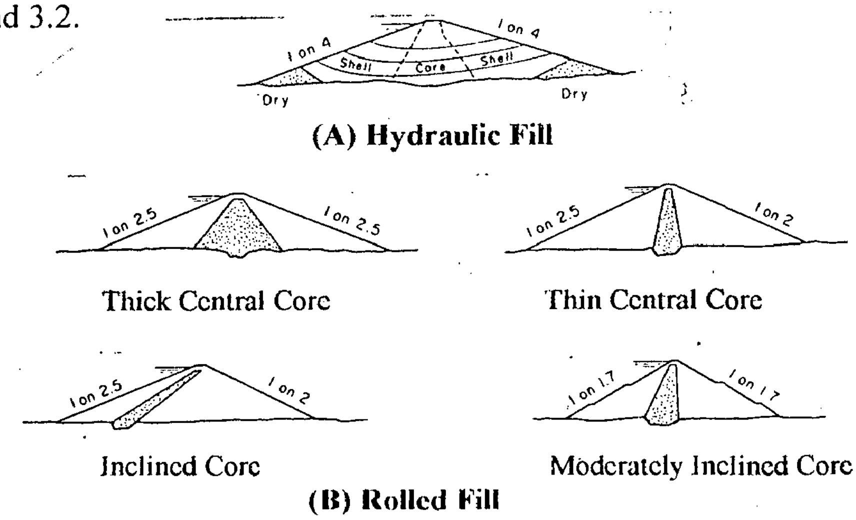


Fig. 3.1. Types of Embankment Dams – Face Slopes indicative only (Slope protection and transition zones not shown)

This review of core of dam in this chapter has been divided into different sub heads depending on shape (vertical, moderately inclined, inclined) and materials (clay or blended). In some cases performance of core is also discussed.

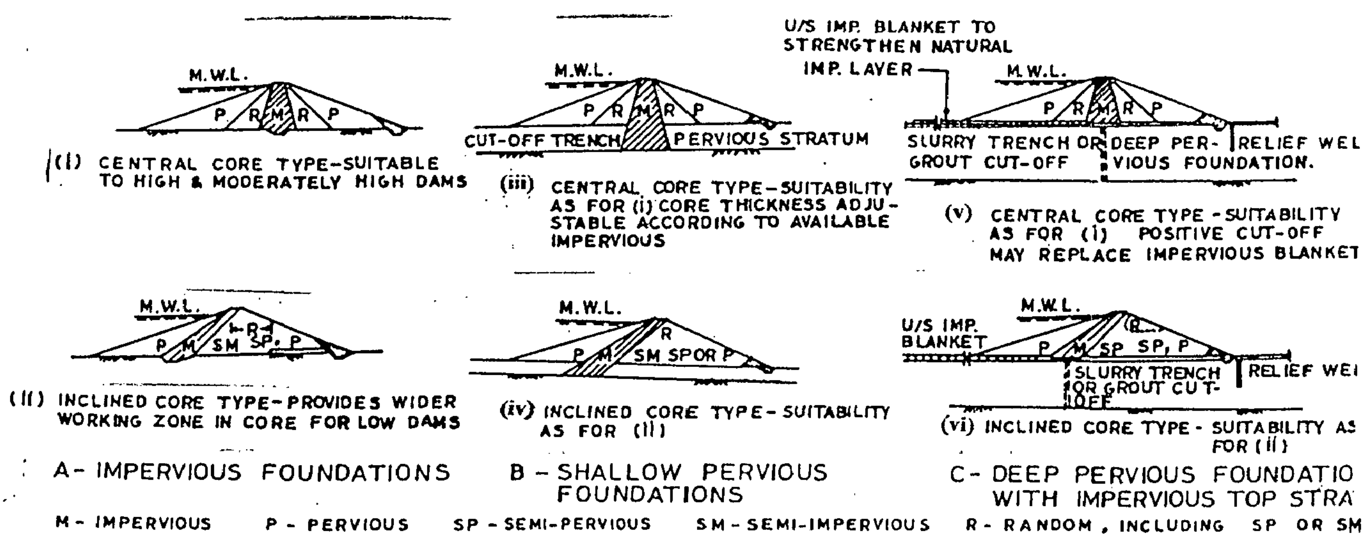


Fig. 3.2. Typical Sections for different foundation condition

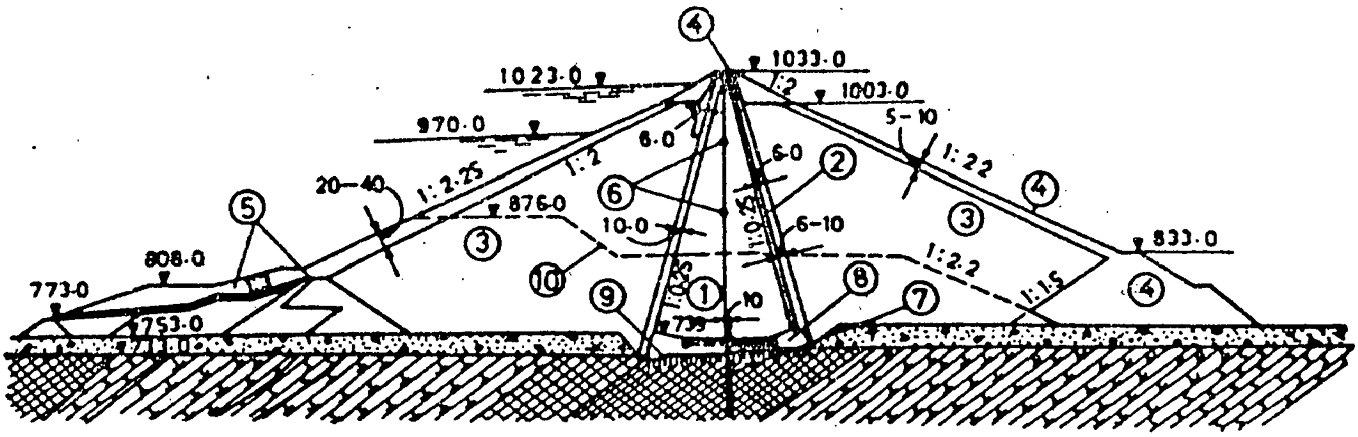
3.2. Central Core Embankment Dams with & without Blended Core Material

3.2.1. Nurek Dam (with Blended Core Material)

The 300 m high Nurek Dam located on the Vakhsh River Tajikistan (USSR) in a narrow mountain canyon composed of hard rock with alternating aleurolites and sandstones, is one of the highest completed dam in the world. The project is a multipurpose project for power, irrigation and navigation. The total volume of the embankment is about 34 x 10^6 m^3. For design maximum recorded flow of about 3,730 m^3 / sec. and a minimum of 120 m^3/sec., a flood discharge of 5,400 m^3/sec for 1:10,000 years, and of 4,220 m^3/sec. for 1:1000 years frequency are taken. The reservoir behind the dam has a gross storage capacity of about 10.5 billion m^3 and a live storage capacity of 4.5 billion m^3. The power station has an installed capacity of 2,700 MW with 9 vertical power units. It is an earth dam with a vertical core, and the embankment cross section and embankment elevations are shown in Fig.3.3. [121,147, 152].

1. Foundation Condition [109, 121, 147].

The dam is constructed in the region where seismic shocks might be of magnitude as high as 8-9. Broken stones form the external parts of the embankments. The canyon is composed of aleurolites which are known to be easily weathered. Therefore the rock surface was protected by guniting after the foundation below the dam core had been cleaned.



(1)Rocky Clay, (2) Filter, (3) Coarse gravel shells, (4) Surcharge of slopes with oversize rock
(5) Upstream coffer-dam, (6) Inspection galleries for instrumentation, (7) Grout curtain.
(8) Concrete block, (9) Surface grouting, (10) Profiles of dam in first stage.

Fig. 3.3. Cross Section of the Nurek Dam

At Nurek Dam, a concrete block (see Fig.3.3) fills the narrow gap cut through the rock by the river and ensures a tight junction between dam core and foundation rock. The block raises the base level of the foundation under the core and has affected considerable reduction in the volume of rock-blasting in the foundation excavation. The rock surface of the bed and banks were also rough with a complex system of hollows, caverns and ledges, and placement of concrete on the base eliminated expensive foundation preparation. Beside providing a base for executing grouting operations, the concrete pad prevents leakage through foundation defects from reservoir coming into direct contact with the core material, thus providing added safety against possible erosion.

Grouting of the canyon walls is carried out from several galleries located at different elevations (Fig.3.4).

2. Core Design and its Material

The dam section has central core with slopes of 0.25H:1V protected by filters on either side. The larger height of this dam with a narrow canyon necessitated extensive laboratory and field studies of the material for the core.

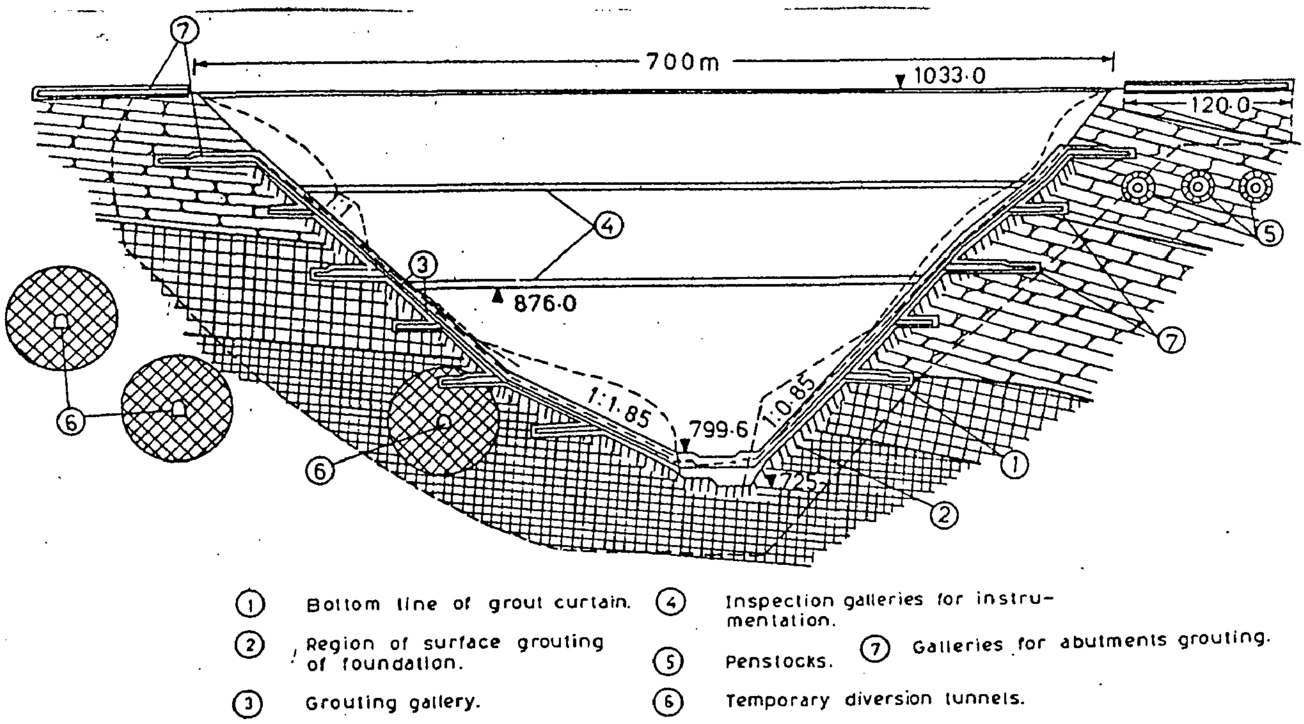


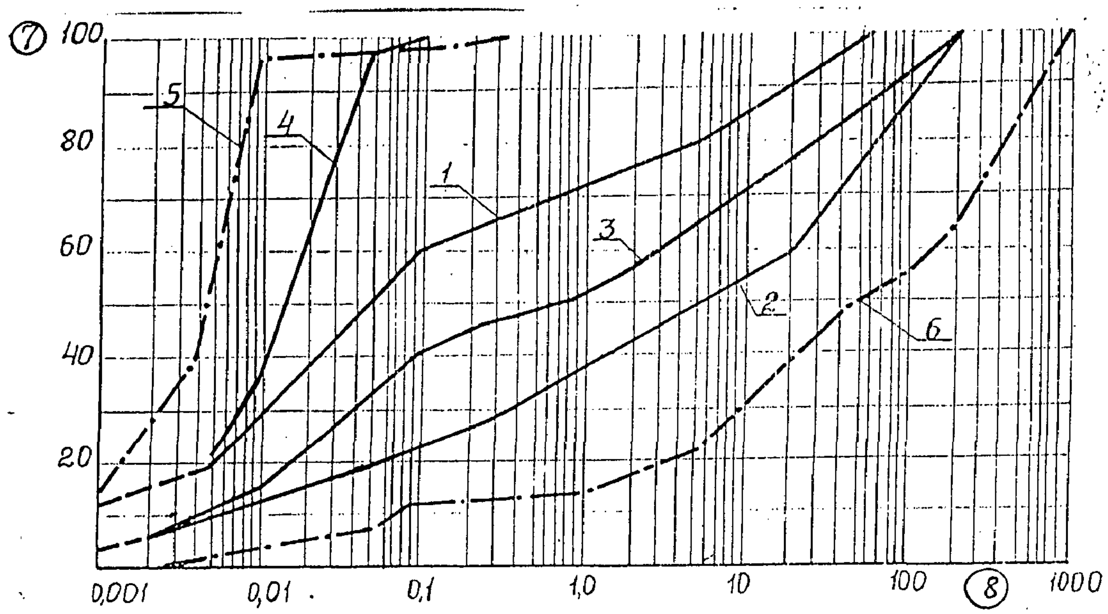
Fig.3.4. Longitudinal section along the abutment grouting galleries of Nurek Dam

According to the original design the dam core was intended to be filled with loess loam depending on its availability in abundance. In the design the compacted dry density of the loess loam was specified within 1.75 – 1.80 t/m^3. Studies revealed that this soil, would have a number of disadvantages such as : (a) relatively high values of settlements in the process of consolidation which, due to the interaction between rigid and easily consolidated upstream and downstream shells and the canyon walls may lead to the arching action in the core, and as a consequence to the stress decrease; (b) insufficient resistance of loess loam to erosion in case of crack formation in the core. As regards the grading, loess loam is a highly uniform fine grain material lacking the necessary quantity of coarse particles which would contribute to stability of the soil in the case of leakage (see Fig.3.5. of Grading curves for the core material).

However in the dam site region there are deposits of *rocky clay* varying widely in plasticity and rock content. Extensive laboratory and field studies of rocky clay yielded convincing evidence of the advantage of its use for the core material in contrast to the loess loam.

Analysis showed that sizes finer than 5 mm constitute from 20 to 100 %. Seepage and piping tests have disclosed that the lower limit of the permissible percent of particles finer than 5 mm should be not less than 50% (Fig.3.5).

That soil density ensuring its piping resistance was found to be somewhat lower than the values corresponding to the relative density of 98%. The total dry density of rocky clay ranged within 2.0 – 2.3 t/m^3.



(1) & (2) Upper & lower limiting curves of the rocky clay grain size distribution
(3) Average curve of the rocky clay, (4) Average curve of the loess loam,
(5) and (6) Range of variation of the clayey gravel grain size distribution in the quarry, (7) Percentage of particles content, (8) Particles dia., mm.

Fig.3.5. Grading Curves for the Core Materials of Nurek Dam, U.S.S.R.

The piping resistance test results are: (a) rocky clay with the given grading and density will possess adequate piping resistance if the sizes of the filter material are within 0.1 – 10 mm; (b) material placed just into the near-the contact zone should comprise sizes smaller than 5 mm not less than 60%, i.e. 20% in excess of that of the main part of the core soil; (c) coefficient of permeability of rocky clay which is within the allowable grain size distribution (Fig.3.5) vary from of about 10^{-5} to 10^{-7} cm/sec.

Prior to placement into the core embankment, the soil preparation followed: (a) mixing of the soils to decrease the variation of grading up to the allowable values; (b) wetting of the soil up to the optimum moisture content (10-12%).

As a result of analysis performed according to the requirements of engineering and economy, the method of mixing in stockpiles has been preferred over the mechanical mixing. With the help of this method the problem of additional wetting of soil has been solved rather easily in the process of placing soil layers of the large stock-piles (100 – 150 thou. cu. m).

The field density tests of "trial fills" showed that the layer thickness should not be in excess of 30 cm to attain the specified density. The soil is compacted by self-propelled rubber-tyred rollers.

As the gorge section has fairly steep slopes, the possibility of tensile stresses resulting in cracking or hydraulic fracturing of the core had to be guarded against. The technique of centrifuge model testing was employed for this purpose.

The test indicated tensile zones in the upper part of the core, adjacent to the canyon walls; while in the center of the core a compressive zone appeared (Fig.3.6.). The cracks in the model were located at a distance of 0.2 to 0.3 of the crest length from the abutments and were 45 to 50 mm deep. The remedial measures adopted are:

- the upper part of the core at the place adjacent to the canyon walls is filled with a more elastic soil;
- a larger surcharge of soil was placed on the core.

Subject to these precautions, studies indicated that the compressive stress in the core would exceed the hydrostatic pressure by 20 to 30% over most of the core heights, and by about 10 % at its contact with concrete block at the base. However the computation did not take the soil creep into consideration and so the stresses would be somewhat higher by the end of the dam construction [121 and 147].

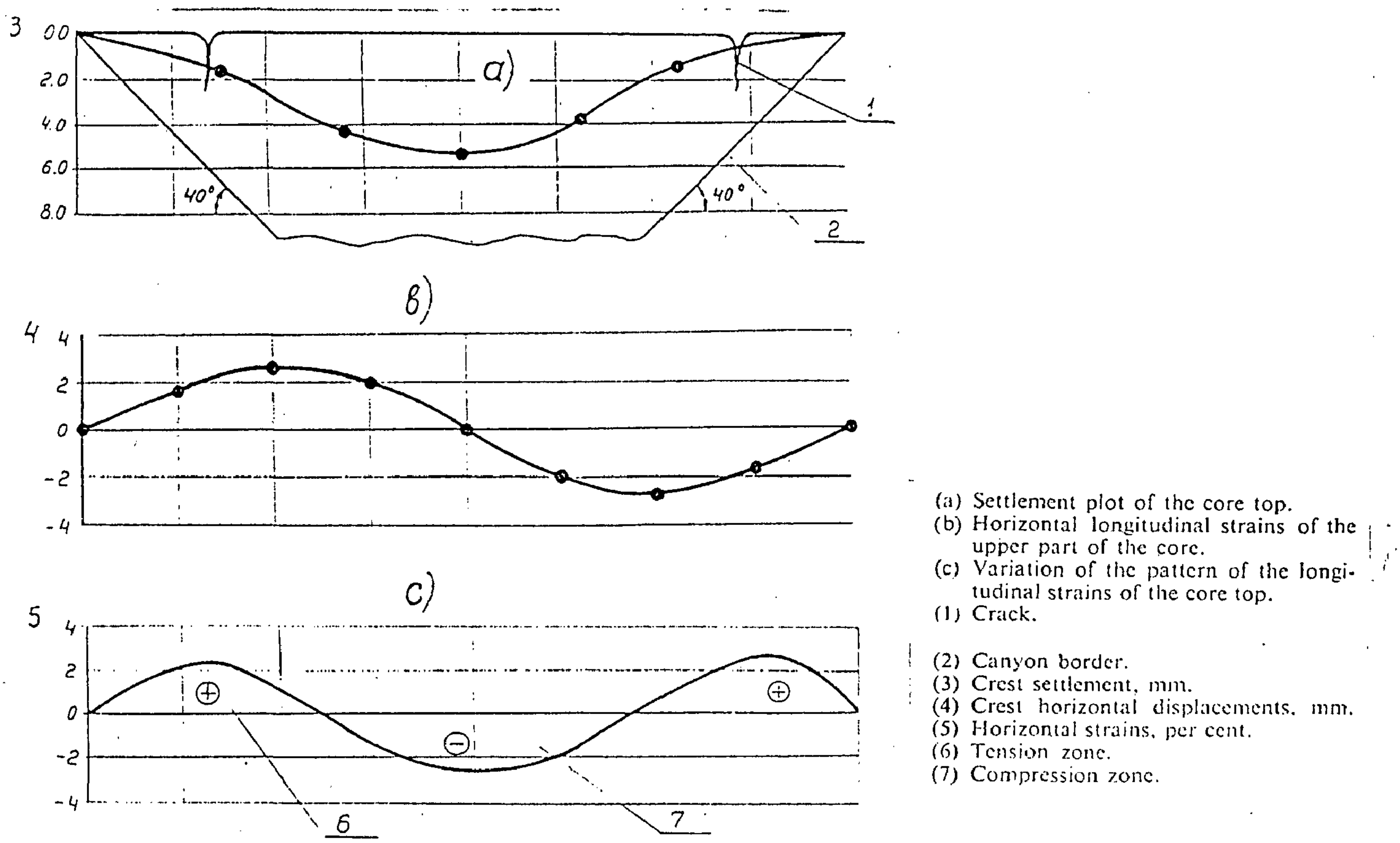


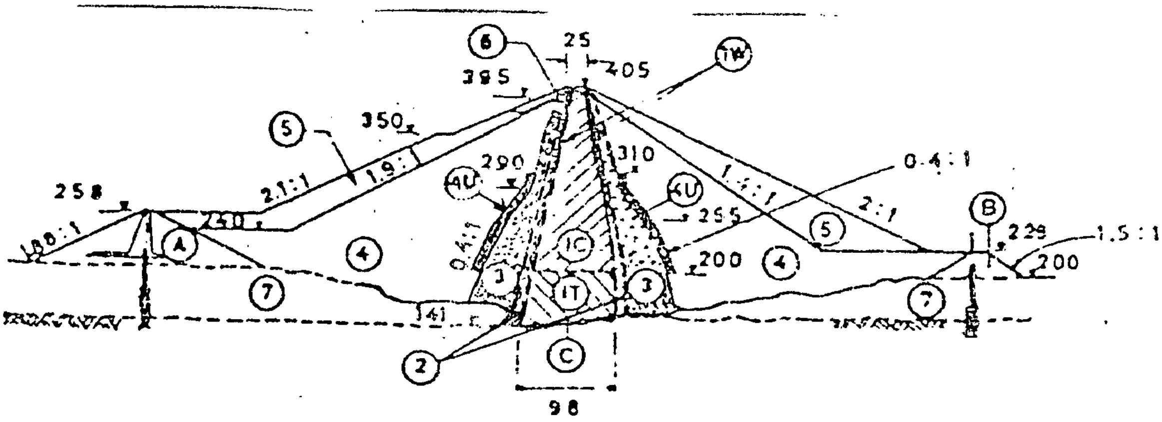
Fig.3.6. Result of Experimental Centrifuge Investigations of the Core

3.2.2. Chicoasen Dam (with Blended Core Material)

The dam is an ECRD (Earth Core Rockfill Dam) 207 m high over the river bed with a maximum height of 261 m above the bed rock. It consists of an impervious core 98 m wide at the base and 15 m at the crest, protected by filters with an average thickness of 7.5 m and ample transitional zones. The maximum section of the dam is given in Fig.3.7. The dam is built across the Grijalva River of Chiapas State, Mexico, for hydro-electric power development with an installed capacity of 2400 MW [2, 78, 79, 121, and 109].

1. Foundation Condition

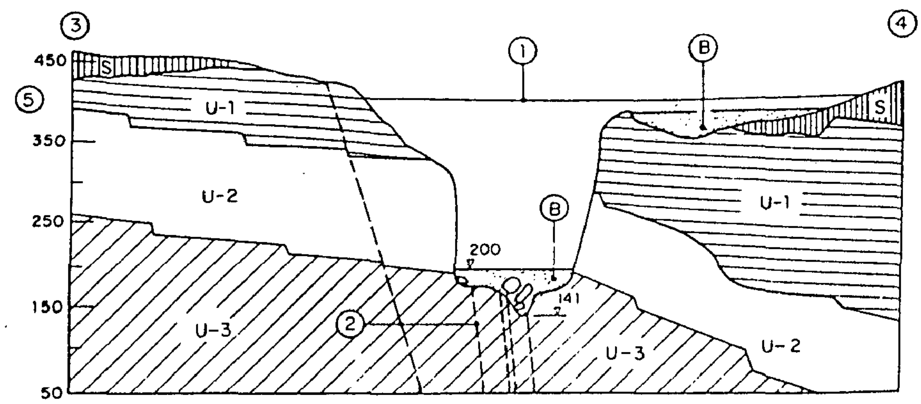
The dam is located in narrow gorge (L/H=1.1), with abutments rising almost vertically up to the mid height, and an abrupt change in the slope of the upper part of the left bank. A regional thrust fault crosses the valley about 5 km upstream from the dam axis, while another normal fault is located 500 m downstream of



(A)Upstream Coffer-dam, (B)Downstream Coffer-dam, (C)Concrete block, (1)Impervious Core, 1T=Tejeira material, 1C=La Costilla material, 1W= 1C material of 2 to 3% above of optimum, (2) Filters, (3)Transition zones, (4)Compacted rockfill (well graded), (4U)Uniform rockfill (15-25 cm), (5) Dumped Rockfill, (6)Selected rockfill (wave protection), (7)Alluvium.

Fig. 3.7. Maximum Section of Chicoasen Dam

the dam axis. The canyon rocks are limestone. The geological profile is shown in Fig.3.8. The base area of the dam is cut by three systems of sub-vertical faults. The seismicity of the region is high.

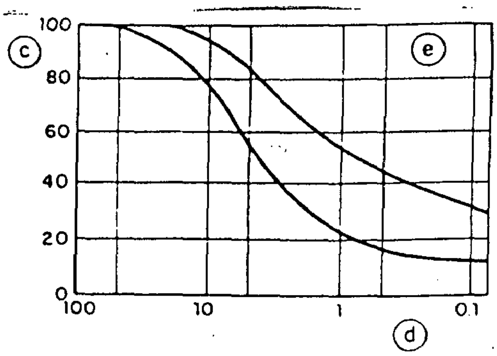


U-1, U-3	Limestone interbedded with clay shales.	(1)	Dam crest, El. 402.
U-2	Massive limestone.	(2)	γ-faults.
(B)	River deposits with big boulders.	(3)	Left bank.
(S)	Clay shales.	(4)	Right bank.
		(5)	Elevations, in m.

Fig.3.8. Geological profile along the dam axis of Chicoasen Dam

2. *Core Material*

The dam core is composed of clayey soil with high gravel content (Fig.3.9a and b) from two borrow areas. Both are residual soils, one from weathered conglo-



(1) Material.
(2) Tejería.
(3) La Costilla.
(a) Fraction retained in mesh No. 200.
(b) Fraction finer than mesh No. 200.

(c) Percent finer.
(d) Grain size, in mm.
(e) Proposed gradation boundaries for the core material.

(1)	(a)			(b)		
	G_s	d_{10}	C_u	G_s	ω_L	I_p
(2)	2.70	<0.074	>100	2.73	37	20
(3)	2.78	$< 10^{-2}$ a to 1.5	> 100 a to 8	2.78	33	12

G_s = Specific gravity

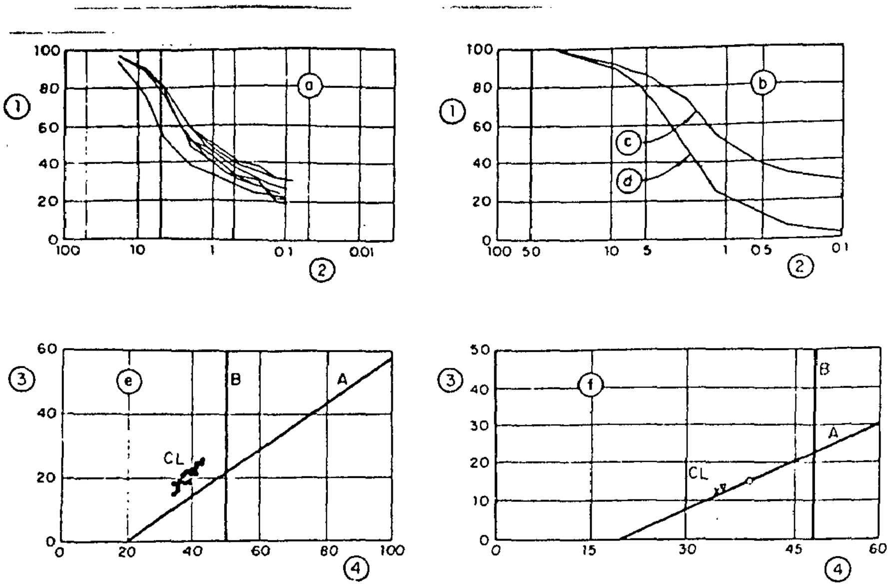
d_{10} = Effective diameter, mm

C_u = Coeff. of uniformity

ω_L = Liquid limit

I_p = Plasticity index

Fig.3.9 a. Index Properties of the Clayey Material of the Core of Chicoasen Dam.



(1) Percent finer.
(2) Grain size, in mm.
(3) Plasticity index, in %.
(4) Liquid limit, %.
(×) Finer than No. 40-mesh (La Costilla upper pit).
(O) Finer than No. 200-mesh (La Costilla upper pit).
(∇) Finer than No. 40-mesh (La Costilla lower pit).

(a) Tejería material.
(b) La Costilla material.
(c) Lower part of borrow pit.
(d) Upper part of borrow pit.
(e) Tejería material finer than No. 40-mesh.
(f) La Costilla material.
(A) Ligne A.
(B) Ligne B.

Fig.3.9b. Index Properties of the Clayey Material of the Core of Chicoasen Dam (continued).

merate (Tejerial materials) and the other from the decomposed surface of clay shale (La Costilla materials). The fine fraction of both soils consists of clays of low plasticity. The natural water content was below optimum. The grain size distribution of the two materials showed that their mixing would be beneficial.

In the core, material with a higher friction angle was used for the lower portion of the core and the material with more cohesion for the upper portion. Both materials were separately processed for homogenization, wetting and curing. The material was hauled to a stockpile area, spread in 50 cm thick layers and wetted with sprinklers. It was mixed with graders and cured for at least one week, then excavated with front loaders in cuts higher than 2.0 m. The material was spread on the embankment in 25 cm thick layers and compacted with six passes of 7 Ton vibratory rollers. The placement was specified at optimum moisture content.

3. Filter Material

The core is protected by filters with an average thickness of 7.5 m and ample transitional zones. The filter material was well-graded sand and gravel with sizes up to 76 mm. The upstream filter was made from processed natural materials while the downstream filter was made from crushed, screened and washed limestone rock. The gradation curve of the filter is shown in the Fig.3.10.

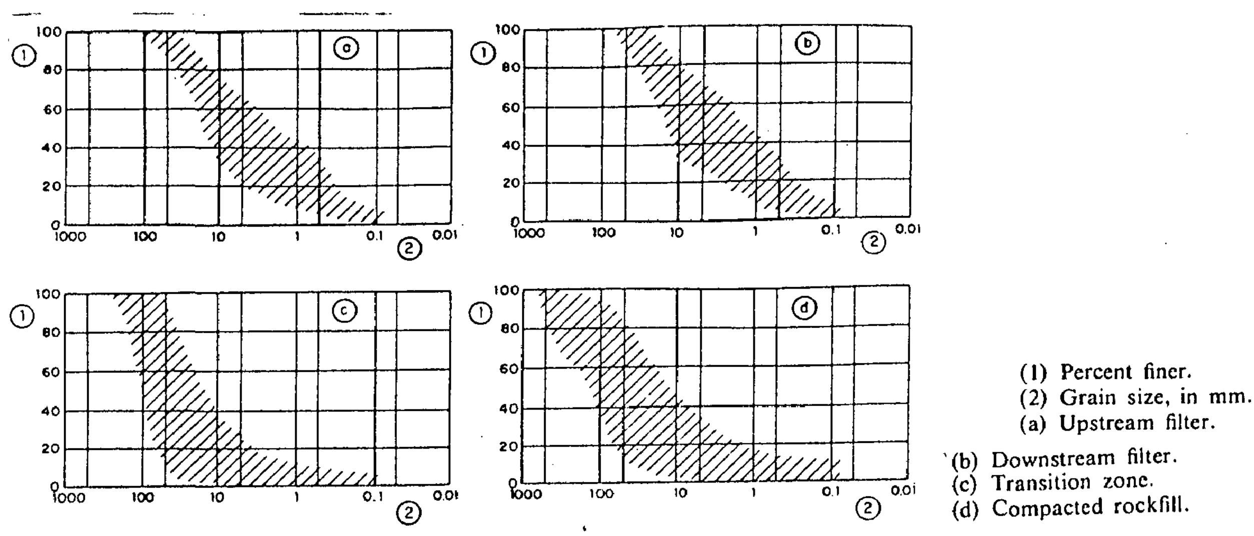


Fig.3.10. Gradation of the filters, transition zones and compacted rockfill materials (Field laboratory Data) of Chicoasen Dam.

4. Core Contact Treatment Design

Three main critical problems identified in the design of the dam that related to core design were: (a) core-abutment interaction, which might lead to the development of zones of low effective stresses in the lower part of the core; (b) core-filter interaction, which might further reduce the stresses deep inside the core; and (c) the possible development of tension zones in the core in the vicinity of the abrupt slope change in the left abutment.

At design stage particular attention was paid to core-abutment interaction. The structure was profusely instrumented, not only to detect possible behavioral anomalies, but also to evaluate the effectiveness of the measures adopted to reduce such interaction.

A 3-D finite element analysis considering the geometry of the canyon profile showed clearly the need to allow easy shear deformations at the core-abutment contact. This was achieved by providing a *4.0 m thick strip of clayey material placed at 2 to 3 % higher moisture content than the optimum adjacent to the canyon walls* to promote the displacement of the core relative to the abutments and thereby minimize arching.

Reduction of vertical stresses in the core by load transfer to the shoulders was overcome by introducing weak zones at the core interfaces. A zone of more compressible material within the rockfill upstream and downstream of the core comprising 10 m wide strips of uniform size rockfill with particles of 15 to 25 cm size was provided. It was expected that the contact crushing and grain breakage would limit the shear strength of the strips. However, detailed field measurements during construction revealed it adequate as the stress level that developed in the uniform rockfill strip was not high enough to induce breakage of the rock fragments. The rockfill zones were, therefore, replaced by two strips each of 3.0 m wide clayey soil placed at moisture content 2 to 3 % more than optimum and located next to the upstream and downstream slopes of the impervious core, as shown in the Fig. 3.11. The result was a reduction in the transition zone, which was kept to a minimum thickness of 10 m from El. 290 upwards.

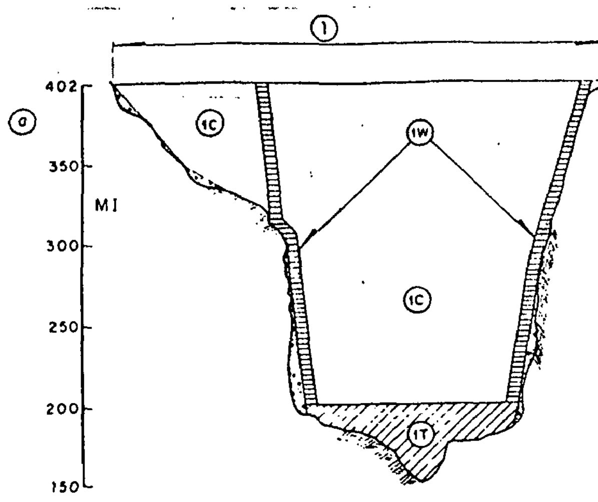
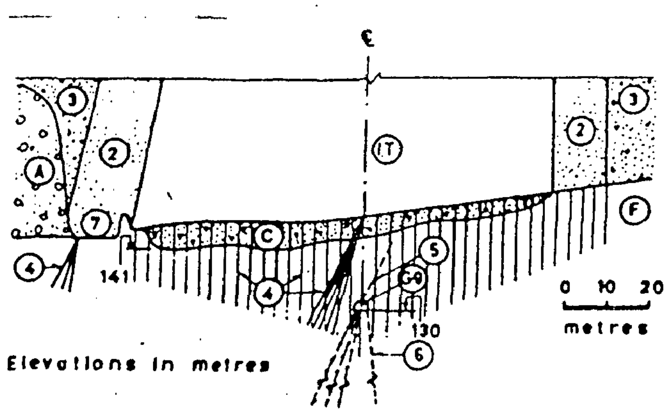


Fig. 3.11. Section along Dam axis showing Treatment for abrupt change in the abutment slope.

The finite element studies indicated that because of abrupt change in the slope of the left abutment, the clayey mass in the zone near the left abutment between El. 280 and 300 would be subjected to tensile stresses. Further studies showed that if the strip of the soft clayey material with moisture content 2 to 3 % wet over optimum was continued up to the top of the dam in the figure, the zone of tensile stress would be eliminated. Hence the section finally adopted is shown in (Fig 3.11).



(IT)Impervious Core (Tejeira material), (2)Filters, (3)Transition zones, (4)Shallow grouting, (5)Grout curtain, (6)Drain holes, (7)Sump, (A)Alluvium, (C)Concrete block, (F)Stratified limestone, (G9)Gallery across the river.

Fig.3.12. Location of Consolidation and Curtain Grouting below The Impervious Core of the Chicoasen Dam

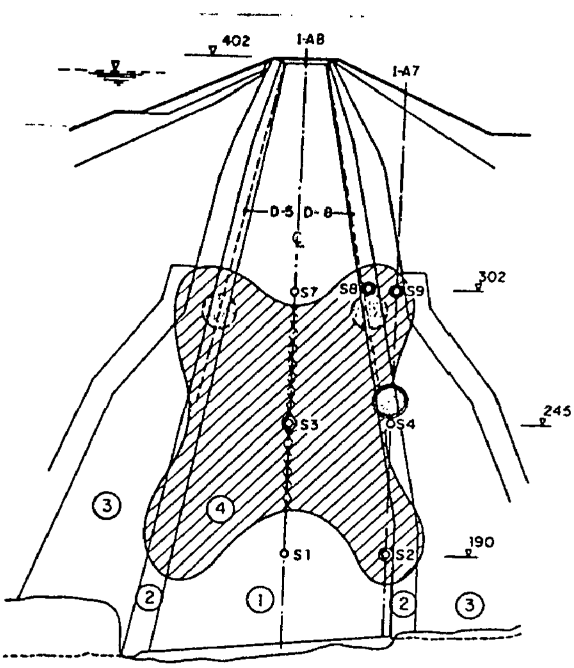
A concrete slab with an average thickness of 5 m was poured under the base of the core. Systematic grouting was done through this slab to consolidate the foundation rock to a depth of about 15 m. Fig. 3.12 shows the location of consolidation and curtain grouting underneath the core-contact area of the dam. A gallery excavated across the river with the invert at elevation 130 m enabled treatment of the foundation at greater depth without interfering with construction of the dam [121,p.533-534].

5. Observations and Dam Performance

Observation and measurements carried out during construction of the dam and first filling of the reservoir from January 1977 to February 1981 revealed satisfactory dam behavior. The maximum settlements recorded in the center of the core are less than 1 % of the height of the dam and differential settlements between the core and the shell are negligible. Furthermore, the deformation both in the direction of the river and normal to it are slight and have caused no cracking at the crest. However, observation data indicated the existence of a plastified zone in the core center at the end of construction, which extended in the upstream rockfill shell during the first filling, causing settlements in the layer of material between El.270 and 320 (Fig.3.13).

During this first filling, settlement within the core along the longitudinal section of the dam are uniform; differential settlements between the core and the walls of gorge are concentrated in the 4 m thick wet zone (1W) close to the abutments. The maximum settlement, in the center of the core, is of the order of 2.30m. Seismicity induced by this first filling was slight.

The maximum horizontal deformations at El.355 and in the direction of the dam axis, are 0.6%, equal to those in the wet zone 1W, close to the left bank. On this basis no significant fissuring is to be expected and, indeed, no cracks have been observed in the crest. At the worst, these deformations may have induced a micro-fissuring of the clay mass between the left bank and the wet zone 1W [79].



Maximum values of vertical displacements in the filters.
Maximum values of vertical and horizontal, parallel to the river, displacements in the filter.
× Maximum values of vertical displacements in the core.
◎ Plastified elements.

D-i Crossarm No. i.
I-Ai Inclinometer Ai.
Si i-cluster.
(1) Core.
(2) Filters.
(3) Transition zones.
(4) Plastified zone.

Fig.3.13. Plastified Zone at the end of Construction.

It was verified by means of numerical analysis with the finite element method that the insertion of two bands of wet clay with a low shearing strength induced an increase of the vertical stresses at the bottom of the gorge. The records of displacements, stresses, and piezometric levels taken at this embankment during its construction and first filling showed that its behavior was satisfactory and that the bands of wet clayey material *fulfilled their objective in the short term.*

But, *long-term behavior*; observation from 1980 to 1990 [2, 78, 79], have proved that the clay bands embedded at the impervious core are not capable of eliminating completely the negative effect of the core-abutment interaction. As a result of the occurrence of an earthquake of magnitude 5.2 on the Richter Scale at the dam site in October 1987 cracking occurred near the right abutment and at the downstream zone of the impervious core. The cracking pattern is shown in Fig.3.14.

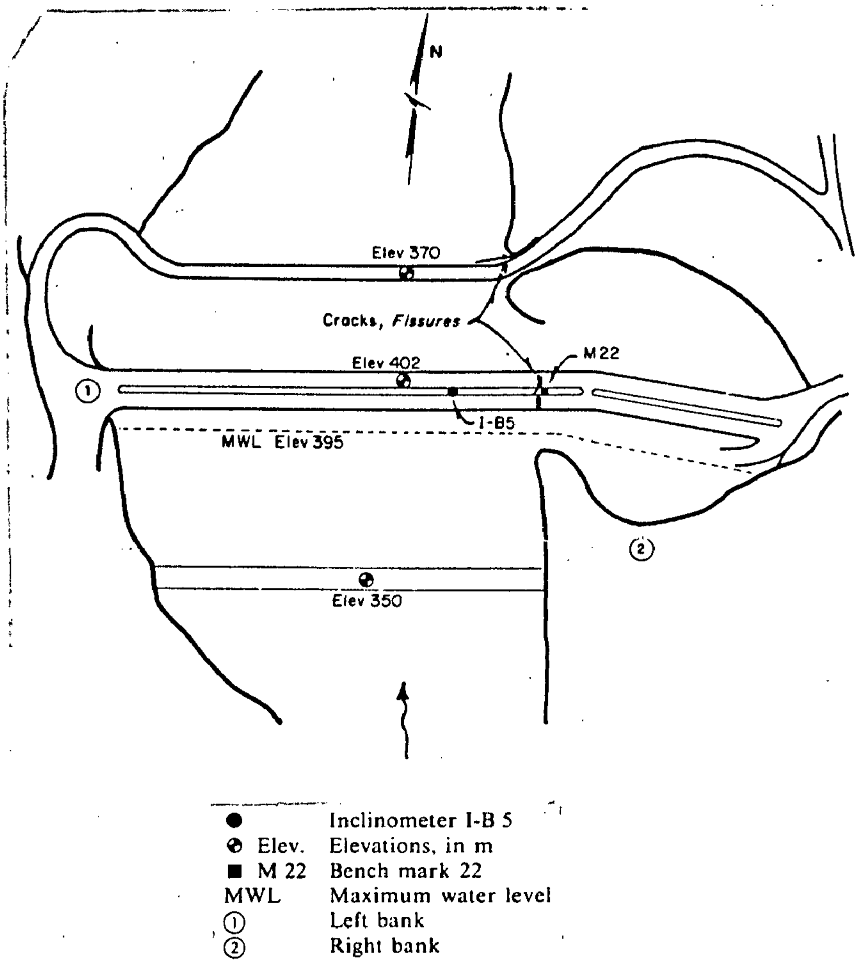


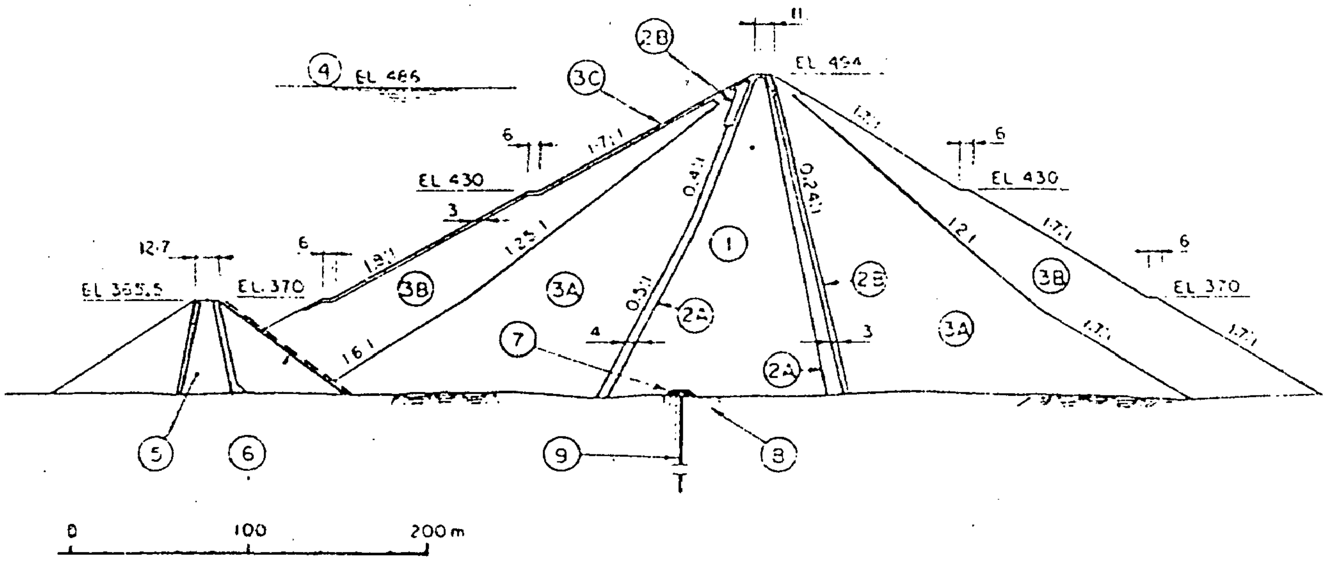
Fig.3.14. Crack crossing the crown and the downstream berm (elev.370)

2.2.3. *Darmouth Dam (with Blended Core Material)* [82]

The 180 m high Darmouth dam is an earth and rockfill dam with a central core of relatively wide base compared with contemporary dams. The dam is located in North-eastern Victoria, Australia. The Mitta River at dam site is deeply incised in a V-shaped gorge with valley slopes averaging 2H:1V but as steep as 1:1 in places. The cross section of the dam is shown in Fig.3.15.

I. Foundation Condition

The dam site is in granitic gneiss, irregularly weathered with massive blocks defined by the jointing pattern, and provided a solid non-compressible rock foundation for the dam.



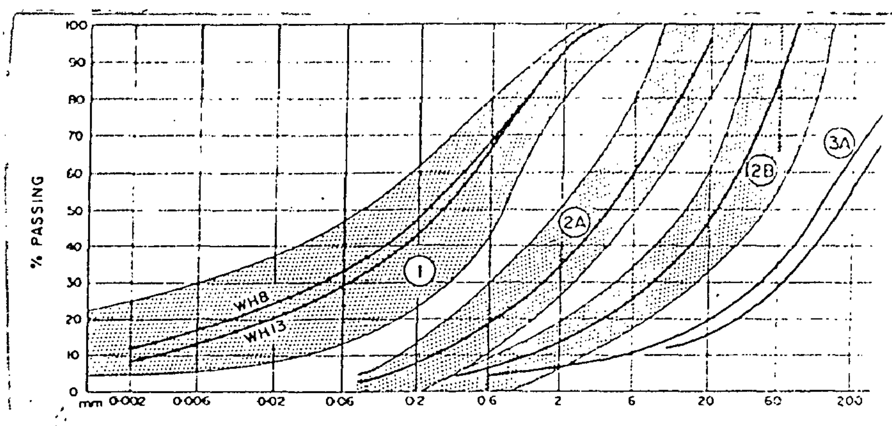
(1) Core, (2A) Fine Filter, (2B) Coarse Filter, (3A) (3B) Rockfill
(3C) Riprap, (4) Full Supply Level, (5) Cofferdam, (6) Mesh Protection
(7) Grout Cap, (8) Blanket Grout Holes, (9) Curtain Grout Holes.

Fig.3.15. Maximum cross section of Darmouth Dam, Australia.

2. Core Design and Blended Core Material

The core material was found to vary with depth in grading and plasticity Index. The four core borrow areas were located in weathered granite in wider valley 2 to 3 km upstream of the dam site. The residual granite soil was of low to medium plasticity, with Plasticity Index (P.I.) ranging from 4 to 18, and averaging about 12. The classification was silty sand to clayey sand. It was therefore excavated by dozing across the soil horizons in depth, while jetting water into it, as it was basically dry (less than optimum), and pushed into large stockpiles for conditioning and to provide a more uniform material. The average grading varied slightly between borrow pits as shown in Fig.3.16.

As there was ample core material available within a short haul distance of the dam site, the dam was designed as a rockfill embankment with central core of generous width (0.7 x height) to provide good stability during drawdown, and resistance to earthquakes, and to give good contact pressures on the foundation without the need for flaring. The width was such that it could be accommodated without unnecessarily flattening the outer rock shell slopes.



(1) Core, (2A) Fine Filter, (2B) Coarse Filter, (3A) (3B) Rockfill

Fig.3.16. Average Gradation Curves & its envelopes for Materials in dam zones, Darmouth Dam.

The moisture content of core materials (relative to Standard Proctor Optimum) were specified as shown in Fig.3.17 to give lower compressibility combined with lower pore pressures in the bottom parts of the core where compressive stresses are high. In the upper section of the dam, increasing relative moisture contents were specified to give more plastic behavior with better ability to take up strains in these areas of lower compressive stress without cracks to occur, particularly transverse cracks in the crest which have occurred in a number of large dams.

The core design also provided for an initial placement of an abutment contact zone on the foundations using the more plastic overlying slopewash material from the borrow pits. This material was placed at a moisture content about 4% more than the optimum and rolled with tyred vehicles to squeeze the material into the irregularities in the abutments.

3. Dam Performance

The behavior of the embankment was monitored by an extensive range of instruments which enabled comparisons to be made between observed and predicted behavior, particularly for the core material. During early construction, monitoring and site control testing indicated higher pore pressures in the core than predicted.

ELEVATION(m) a	RELATIVE MOISTURE CONTENT h: SPECIFIED RANGE b	ADOPTED RANGE c	AVERAGE CONSTRUCTED d	AVERAGE O.M.C. e	BORROW PIT f	AVERAGE P.I. g
480	0 to +2%	0 to +2%	+1.0%	13.5%	WH13	11%
460	-0.5% (dry) to +1.5% (wet)		+0.9%	14.0%		12%
440		-0.5% to +1.5% ±1.0%	+0.4% Opt.	13.6% 14.0%		
420			+0.1%	15.9%	WH8	16%
400		-1.5% to +0.5%	-0.2%	15.1%		14%
380			-0.5%	14.1%	WH10	11%
360		±1.0%	-0.1%	14.6%		8.5%
340	±1.0%		Opt	14.9%	WH12	13.5%
320						22%

(a)Elevation, (b) Specified range, (c) Adopted range, (d)Average as constructed, (e) Average Optimum Moisture Content, (f) Borrow pit, (g) Average Plasticity Index, (h) Relative Moisture content.

Fig.3.17. Embankment construction : Properties of Core Material of Darmouth Dam.

The unusual behavior pattern developed at about 2/3[rd] height of the core with negative pore pressures and low earth pressures at end of construction as shown in Fig.3.18. The unusual pore pressure / earth pressure behavior could be related to: (i) arching of the core due to the stiff filters; (ii) properties of WH8 borrow pit material; (iii) effects of the drier placement of over half the height combined with the narrow core, at this area at about two-thirds height; (iv) change in the upstream slope of cores.

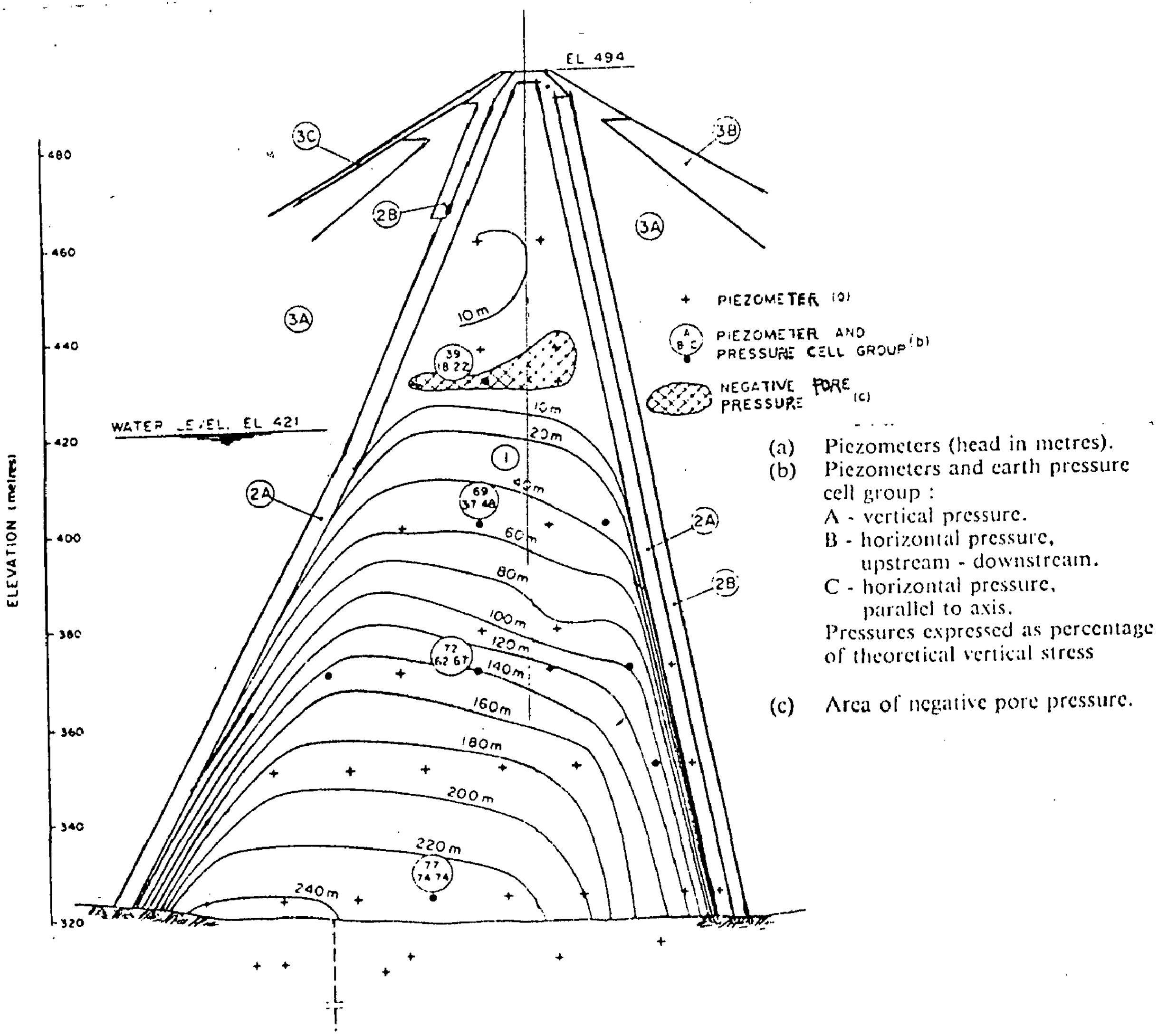


Fig.3.18. Pore pressures and earth presures in the core; End of construction. of Darmouth Dam

3.2.4. Trinity Dam (with Blended Core Material) [148].

The construction of 537 ft (164 m) high Trinity dam was done in 1957. This dam has central impervious core, and is located in the Klamath Mountains of northern California, about 8 miles north of town of Lewiston, and also as part of Central Valley Project, California, USA. The average annual precipitation condition increases with altitude from 100 cm near the dam site to 200 cm in the high country with 75% in all areas occurring in the months of February through June. The maximum cross section of the dam is shown in Fig.3.19.

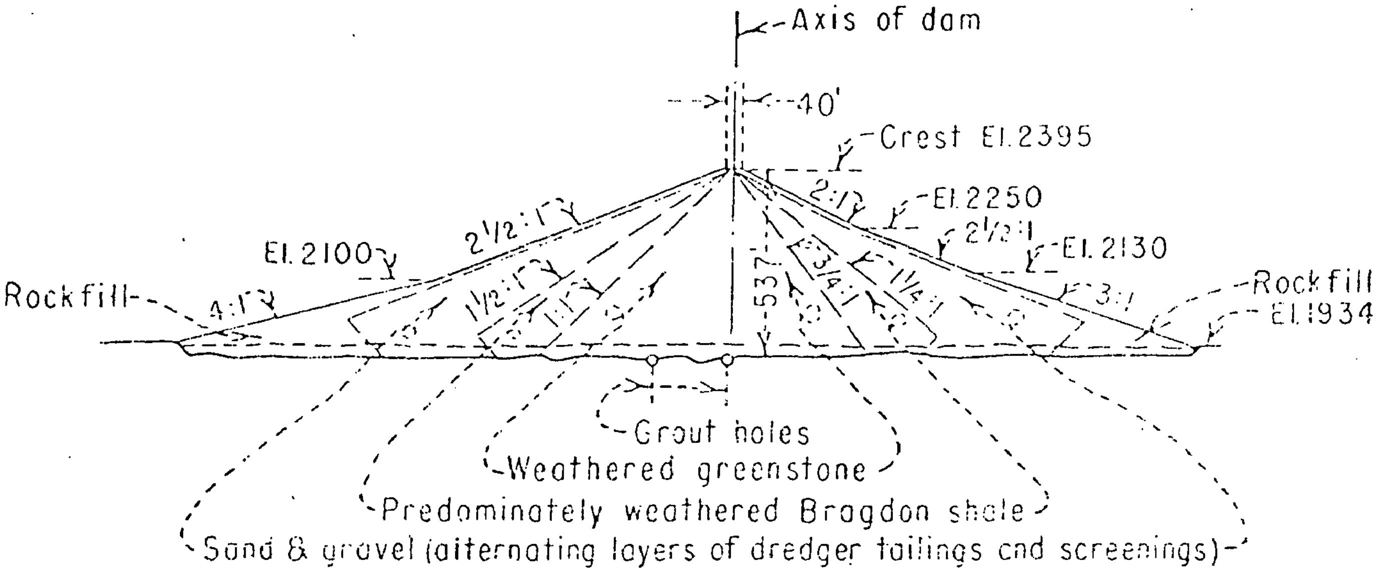


Fig.3.19. The maximum cross section of Trinity Dam, USA.

1. Foundation Condition.

Two bedrock metamorphic formations were found to underlie the site. The riverbed and the major fraction of the canyon walls are cut in the Copley metaandesite formation. The rock is greenstone widely exposed in the Klamath Mountains and in the Trinity River Canyon walls.

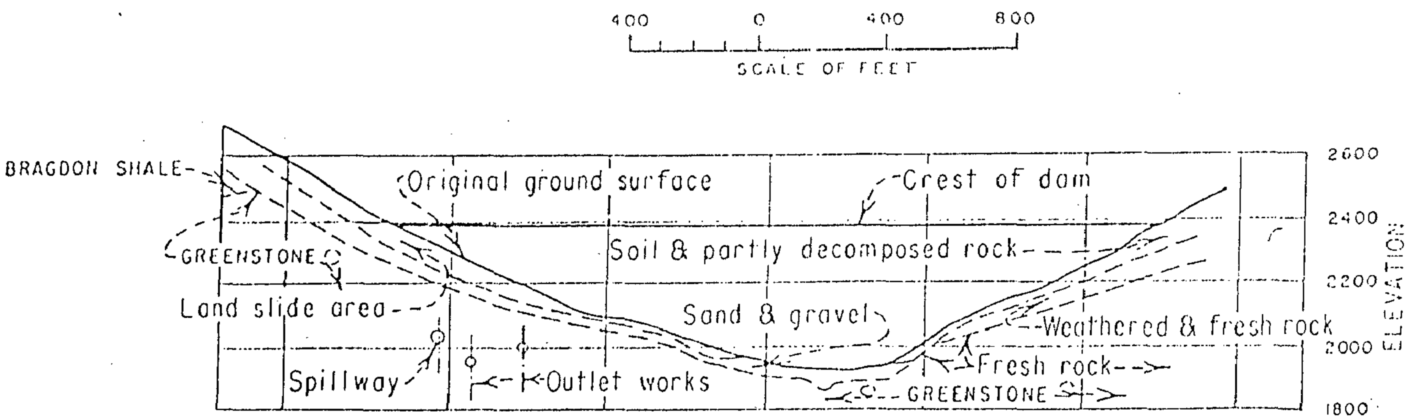


Fig.3.20. Profile on axis of the dam, Trinity Dam, USA.

A fairly good foundation of hard rock in the valley bottom and over the lower quarter of the abutments are available. But a rather broad band of more competent or less fractured rock that forms a ridge does provide a place to anchor the impervious section of the dam on the right abutment. Profile on axis of the dam is shown in Fig.3.20.

2. Core Design and its Material

The thick central core of this dam has slopes 1H:1V and 0.75H:1V for upstream and downstream faces respectively, with approximate maximum hydraulic gradient (H/B) of about 0.55. This core was made after the availability of the construction materials for all the zones of the dam was ensured within a reasonable haul distance, such as residual soils from Weaverville, Bragdon, and Copley formations; dredged and undredged streambed deposits; and quarried rock.

The impervious core material was coming after processing residual Copley metaandesite rocks, which had very similar density, moisture, compressibility, and shear properties on the basis of minus No.4 fractions. The material is excavated from various parts of the pit to obtain a blend with highest rock percentage. Oversize is crushed and fed back into the mixture. The material was conveyed to the dam site by a conveyor belt system. The material arrived at the terminal station of the conveyor system 2 to 5% less than optimum moisture. Here, sufficient water is added to ensure its arrival on the embankment at about 1 % less of optimum.

This core material was placed in embankment in 30 cm lifts and with 12 passes of the specified compaction roller. The core embankment properties are: permeability 1×10^{-7} cm/sec, angle of internal friction 28°, cohesion 0.15 kg/cm^2, compacted dry unit weight 1.74 ton/m^3, and moisture content 15.5% (optimum 16.5%). The general engineering properties of the core material together with other zones are shown in Table.3.1.

3. Dam performance

After completion, piezometer readings in the fill showed materially less pore pressures than values estimated in pre-construction stability analysis.

Table.3.1. General engineering properties of the core material, together with other zones, Trinity Dam.

SOURCE	% of Gradation: - 0.005 mm	- 0.005 mm to No.200	No.200 to No.4"	No.4 to No.3"	Unified Soil Classification	(*) Specific Gravity	(*) Dry Density t/m^3	(*) Optimum Water Content (%)	Natural Water Content (%)	Angle of Internal Friction - (Degree)	Cohesion - Optimum kg/cm^2	(*) Cohesion - Saturated kg/m^2	(*) Permeability - k 1×10^{-7} cm/sec.	(*) Vol. Change-Confined Comp. - % - 300 PSI
Weavarville (decomposed gravels of eocene age)	28	33	20	19	MH	2.88	1.31	33.6	40	24.7	1.1*	0.24	3.87	12
Bragdon Shale (borrow area)	28	47	18	7	ML	2.73	1.68	18	15.2	27.47	1.1*	0.04	3.87	8.8
Bragdon Shale (landslide area)	18	14	18	50	GC	2.72	1.74	16.4	-	32.62	1.03	0.11	3.38	5.7
Copley (greenstone)	7	18	39	36	SM-SC	2.69	1.74	16.5	12.5	28.37	0.9*	0.07	0.97	8.3
Sand-Gravel (undredged)	4	8	31	57	-	-	-	-	-	-	-	-	-	-
Sand-Gravel (dredged)	3	6	21	70	-	-	-	-	-	-	-	-	1334	-

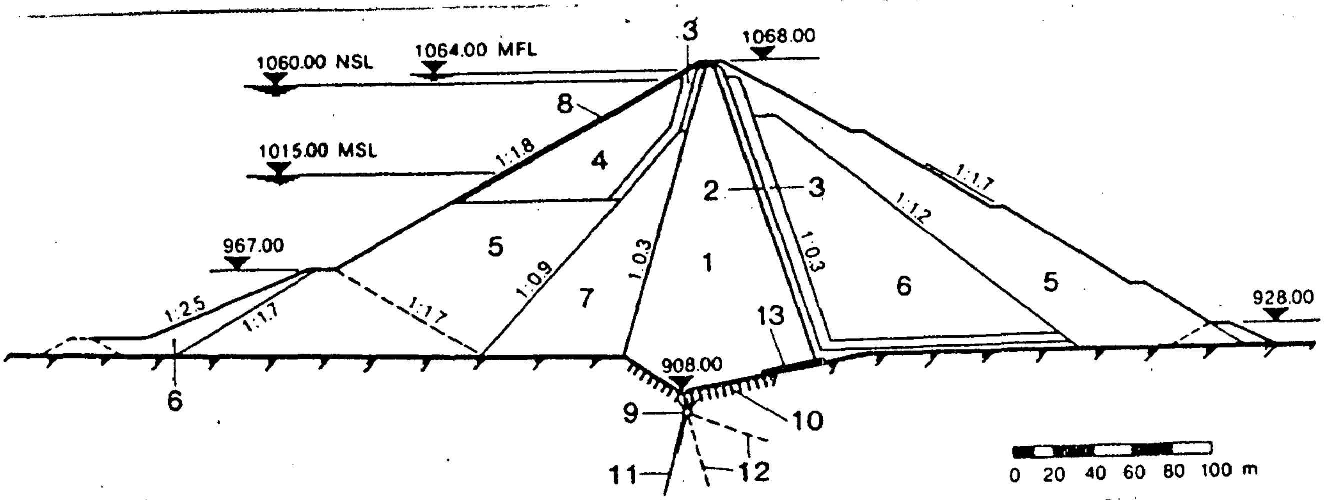
* Based on minus No.4 fraction

3.2.5. *Palo Quemado Dam (without Blended Core Material)* [8].

The Palo Que Mado earth & rockfill of dam is 160 m high, with a vertical core. The dam is located in the western Cordilleras of the Andes (South America) on the Toachi River. The section of the dam is shown in Fig.3.21.

1. Foundation Condition & Seismicity of the sites.

The dam is located in a region of high seismicity belonging to the so-called "fire circle" around the Pacific Ocean. However, earthquakes of magnitude larger than 7.5 on the Richter scale have never been recorded within a radius of 100 km around the site. In this case, it was not possible to define with sufficient accuracy a system of faults. On the other hand, the volcanic activity of the Andean Cordillera contributes to a larger extent to the local seismicity.



(1) Core (volcanic breccia).
(2) Filter.
(3) Drain.
(4) Selected coarse rockfill.
(5) Average quality rockfill.
(6) Weathered rockfill.
(7) Transition zone.
(8) Riprap.
(9) Grouting and drainage gallery.
(10) Contact grout holes.
(11) Grout curtain.
(12) Drainage holes.
(13) Gunite blanket.

Figs.3.21. Palo Quemado Dam, Cross Section.

3. *Core Material*

The core material is volcanic breccia (so-called "lahar"). The main characteristic of this material (volcanic breccia / ML or Lahar), as determined in the laboratory are as below:

a. Grain size: < 100 mm , D_{50} = 2 mm.

b. Atterberg Limits : - Liquid limit (w_L = 33.1%)
- Plastic limit (w_P) = 25.5%
- Plasticity Index (I_P) = 7.5%

c. Effective shear strength parameter (φm) = 36°

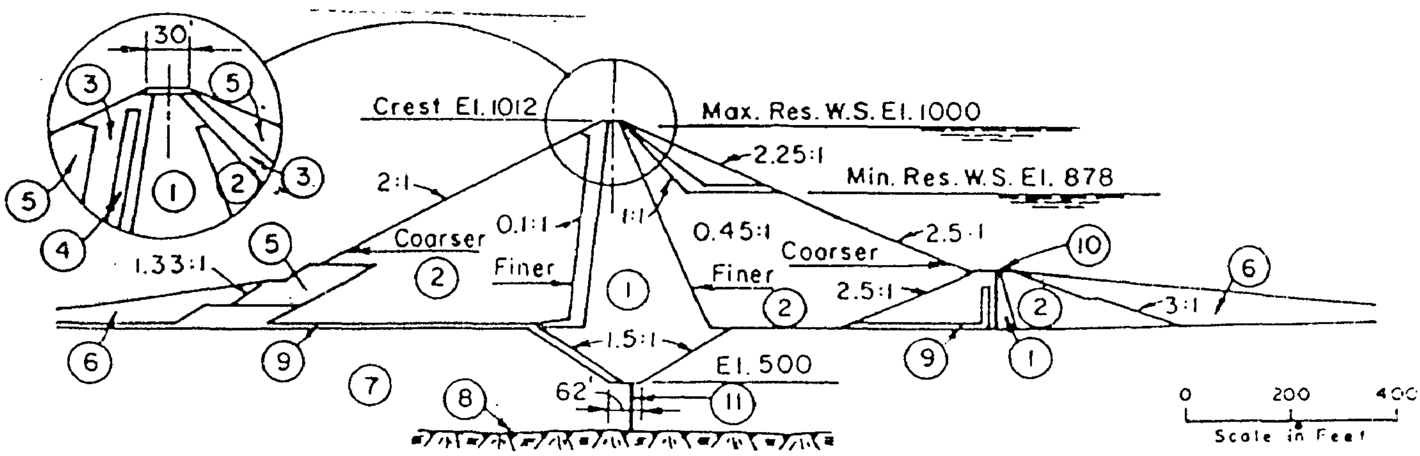
d. γ_d max. = 1.98 t/m^3; $w_{opt.}$ = 12% (Proctor standard)

Field compaction requirement for the core was to compact in 20 cm layers by 8 passes of heavy sheepsfoot roller with an average density of 100 % Proctor standard (PS) and a minimum of 98% PS.

Under dynamic loading condition, a settlement of approximately 50 cm was computed for the core of the Palo Quemado dam, made of volcanic breccia of low plasticity (ML).

3.2.6. ***Swift Dam (with Blended Core Material)*** [33 &101].

The Swift Dam is located on the Lewis River in the southern part of the State of Washington, USA, less than 10 miles from Mouth St. Helens, an active volcano. The height of the dam above foundation is 512 ft (156 m). The typical cross section of the dam as shown in Fig.3.22.



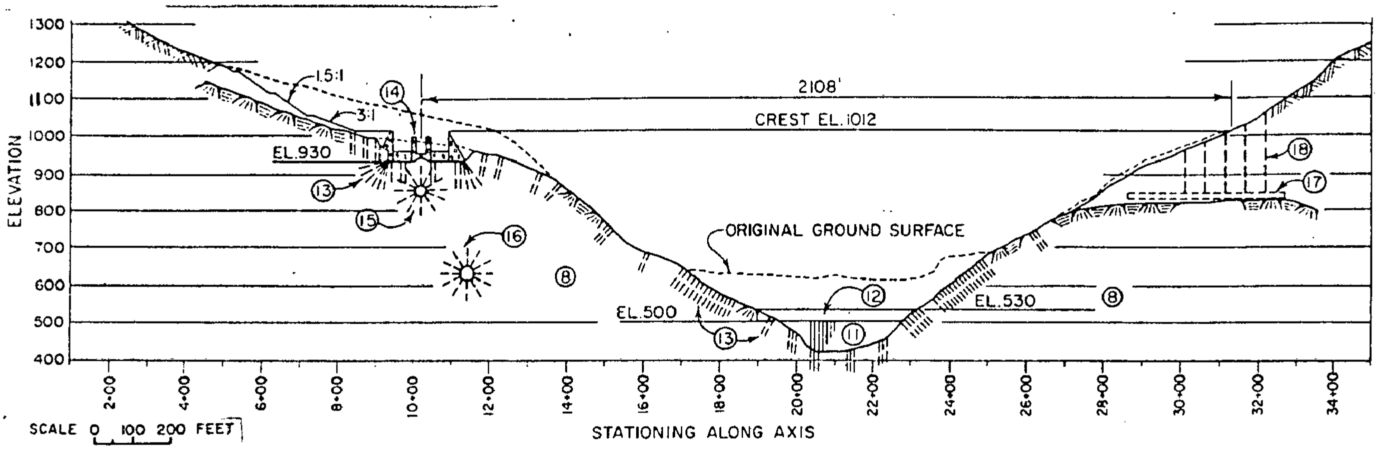
(1)Impervious Core; (2) Random Granular Shells; (3) Sand & Sand Gravel Transition; (4) Processed Drain Material; (5) Rockfill; (6) Spoil material; (7) Streambed materials; (8) Bedrock; (9) Drain; (10) Cofferdam; (11) Pile Cutoff Wall.

Fig.3.22. Typical Cross Section of Swift Dam, USA.

1. *Foundation Condition*

The bedrock at the dam site is dominated by glacial features of volcanic origin composed chiefly of interbedded tuffs, shaly tuffs, and tuffaceous agglomerate which was formed during an early episode of volcanic activity and is a remnant of the ancestral Cascade Mountains.

The foundation was constructed using a combination of open trench excavation, a cutoff wall, and grout curtain. An open cut excavation was made to a depth of approximately 30 m below the streambed. Sheet piles were driven from the bottom of the excavation to bedrock. The profile along the dam axis is shown in Fig.3.23.



(12)Concrete cutoff wall extension; (13)Grout curtain; (14)Spillway; (15) Power Tunnel; (16)Diversion tunnel; (17)Drainage Tunnel; (18)Vertical sand drains.

Fig.3.23. The profile along the dam axis of Swift Dam.

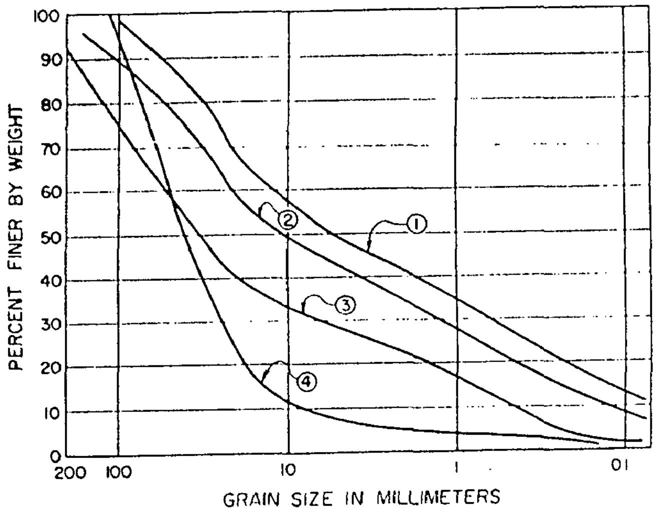
2. *Design of Core and its Material*

The dam has a central impervious core supported by upstream and downstream shell of relatively pervious granular fill.

The most critical item involved in the design of the embankment was the selection of the embankment material which could be placed in the embankment during a period of approximately 18 months under adverse weather conditions. There is no dry period at this site. The driest month July has an average rainfall of over 25 mm.

The mudflow deposits containing 5 to 20% silt from the Swift Creek borrow area was suitable for core in wet weather operations. In soft spots which persisted through the normal embankment operations, it was necessary to manipulate the core fill material by track walking to decrease the water content. If track walking did not ensure satisfactory decrease in water content, stubborn soft spots were removed and the wet material was spread in thin layers on an adjacent area after which coarse grained material was spread and mixed (blending process) into the wet material so that the combined material could be compacted satisfactorily.

The typical gradation curves of the core material and other zones are shown in Fig.3.24.



(1) Impervious Core, (2) Granular Shell, (3)Sandy Gravel Transition
(4) Processed Gravel Drain

Fig.3.24. Typical Gradation Curves of Embankment Materials (including Core Material) of Swift Dam.

The impervious core is compacted with four passes of 50 ton rubber tired roller. During the dry summer months, it was found that adequate compaction could be obtained with as much as four percent below optimum moisture content. During the wet season, excess moisture was removed from the materials so that compaction above the acceptable limit could be obtained easily.

The specification required the material, to be placed in the core section, to contain fines not less than 7% for adequate permeability. The permeability of the material placed in the impervious core ranged from 1×10^{-5} to 5×10^{-5} cm/s.

The average water content of impervious core is 10% at unit weight of 1.834 t/m^3 as compared with a maximum of 1.865 t/m^3 at 12% optimum water content.

Shear strength of the volcanic mudflow material is high. It has no cohesion, due to the extreme angularity of the particles, the angle of internal friction is in the range of 45°.

Use of this extremely high strength material reduced the volume of fill necessary for stability in the dam and lack of plasticity, coupled with the low permeability of material from the mudflow deposits, resulted in a material which is suitable for wet placement in the impervious core section.

3. *Transition & Drain*

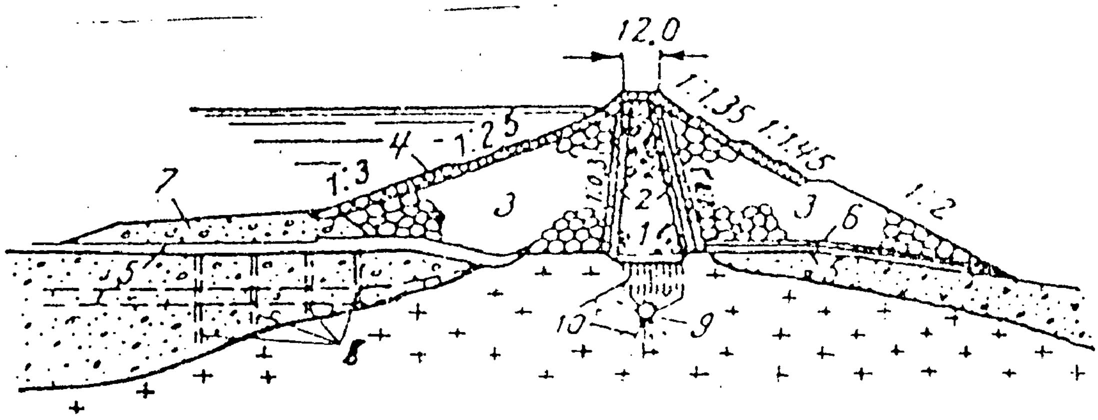
A vertical granular chimney drain placed between the core and downstream shell is connected with a horizontal rock drain blanket constructed on the riverbed in the downstream. The chimney drain is protected both upstream and downstream by transition zone , and the rock drain is also overlain by a transition.

4. *Earthquake / Seismic Condition*

The project lies near the border between the Cowlitz-Willamette area and the middle part of the Cascade Mountains. The largest earthquake within the Cowlitz-Willamette area occurred in 1962 with a maximum intensity of VII on the modified Mercalli (MM) scale. Earthquake associated with eruption of Mount St.Helens occurred in 1980 was located near Marbel Mountain just north of Swift Reservoir. It was of magnitude 5.5. The seismic design of Swift dam is based on intensity VII on MM scale. Horizontal acceleration for pseudo-static stability analysis was taken as 0.07g. A value of 0.1g was used subsequent reevaluation.

3.2.7. Geoscheneralp Dam (with Blended Core Material) [1,70,81,103a]

The Geoscheneralp Dam is 155 m high and was built in 1960 on the Geoschenerreuss River (Switzerland) in the severe climate of the high-mountain Alpine zone. It is a rockfill type dam and has a fairly thin central impervious core (47.0 m thick at the foundation) with hydraulic gradient (H/B) of 3.08. The cross section of the dam is shown in Fig.3.25.



(1)Impervious Core, (2) Filters, (3) Shell from rubble slope, (4) Rip-rap, (5) Drainage layer, (6) Filter layer, (7) Reinforcement fill, (8) Vertical drainage-relief boreholes, (9) Grout Tunnel, (10) Grout Curtain and surface grouting.

Fig.3.25. Cross section of Geoscheneralp Dam, Switzerland

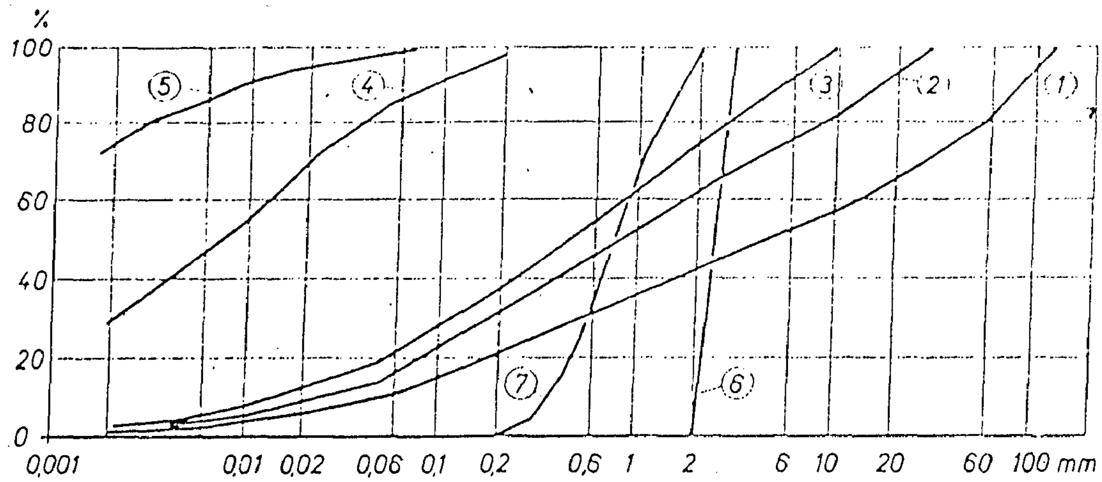
1. Foundation Condition

The foundation is of strong granites, which are, however, permeable along faults and fissures. The granite reach the surface under the central part of the dam cross section, but are covered, on both sides of this part, by alluvial deposits that increase in depth toward both the head and tail waters. Under the core consolidation and curtain grouting was done under the core. The grout consisted of a mixture of Portland cement and Opalinus clay, and grouting was done at a pressure of 5 – 8 atm, from a tunnel in the core foundation.

2. Core design & its materials.

The slope of the fairly thin central impervious core of the Geoscheneralp dam is 0.3H:1V at both upstream and downstream.

Earlier, a thorough investigation of surroundings (transport distance up to 20 km) showed that greater amount of cohesive soil in sufficient quantity were not available. On the basis of soil investigation for grain size analysis in the upper 10 m of the valley floor, it was found that sufficient quantity of material was available. But most of this material was cohesionless and too pervious to be used for the impervious core. This material required mixing with some binders. Therefore it was decided to build the core with sandy gravel (Fig.3.26) from the alluvial deposits of the valley in combination with a binding material. The alternatives of binding material were clay, bentonite and bitumen.



(1) Average curve of the material from borings into the fluvio-glacial deposits of the valley bottom of Geoscheneralp, an extraction of 10.0m.
(2) Sandy gravel, as used for some tests, particle sizes 0 – 30 mm of curve (1)
(3) Standard sand, particle sizes 0 – 10 mm of curve (1)
(4) Opalinus clay , (5) Na-reps. Ca-bentonite from Ponza (220 mesh)
(6) & (7) sand.

Fig.3.26. Grain size distribution curves for Core material

Clay and bentonite are equally suitable as binding agent. Tests were carried out on sandy gravel samples with admixtures of both clay and bentonite to determine the permeability. It appeared that the same degree of impermeability could be achieved with four times clay than bentonite.

Finally core material of Geoscheneralp dam was made by a mixture (blending) of sandy gravel (grain diameter max. 100 mm) and Opalinus clay. Clay binder from Opalinus clay was prepared in a special mixing plant (of the concrete plant type) and mixed with sandy gravel.

The clay binder was 525 kg per m^3 of the mix. The moisture content of the mix was 6 - 8%, and its density 2.2 ton/m^3. The mix was deposited in 30 cm layers and rolled by 35 – 45 ton rollers of pneumatic tires. By meticulous adherence to clay binder production and filling procedures the core could be made sufficiently compact for its mean permeability not to exceed 5 x 10^{-8} cm/sec. The angle of internal friction was 30° - 35°.

3. Filters

Between the core and the shell there are filters, two layers in upstream (6.0 & 4.0m thick) and three in the downstream (4.0, 6.0 & 6.0m thick). The first two layers are of 0 – 100 mm alluvium and gravelly debris with rock waste (100 – 200 mm), the third layer is of crushed rock.

4. Dam performance

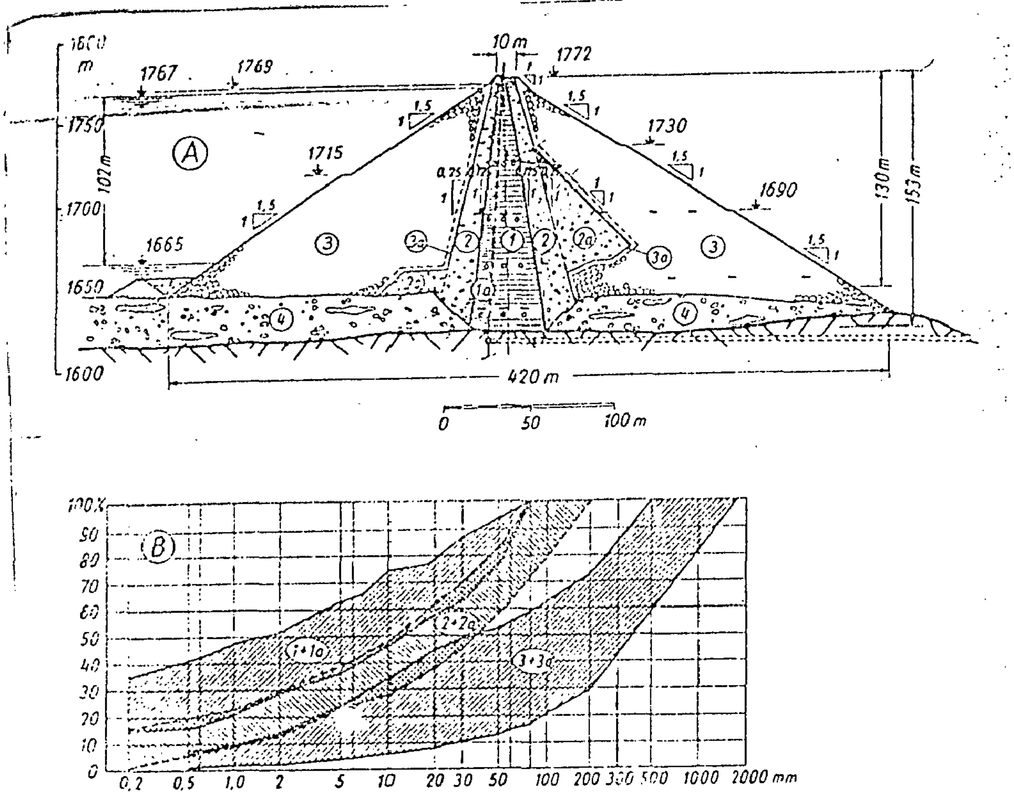
The dam proved satisfactory in service. The settlement at crest and slope are about 1.5 – 2.5%, and in certain sections it reached 5 – 7% with respect to height. After the reservoir had been filled the crest shifted 10 – 14 cm downstream, returning to 5 – 7 cm after draw down. Upon repeated filling and draw down displacements were reduced [1].

3.2.8. Gepatsch Dam (with Blended Core Material) [1, 104, 109, 121,152]

The 153 m high Gepatsch is a thin central core rockfill dam. It was constructed between 1961 and 1964, and it is located on Inn River, in the Kauer valley, Austria. The typical dam cross section and Grain Gradation Curve of the Gepatsch rockfill dam are shown in Fig. 3.27.

1. Foundation Condition

The dam foundation contains gneisses overlain, on the left bank slope, by boulder, clay and talus material reaching a thickness of 20 – 30 cm, and, in the riverbed section, by alluvia up to a thickness of 20 m. The dam core is joined to the bedrock by an entrenched cutoff. A 60 m deep grout curtain is provided in its foundation. For the purpose of reducing contact seepage, there is a 12.5 cm thick grey clay layer at the bottom of this clay cutoff, as well as surface grouting.



(A)Typical Cross-section
(B)Gradation Curves.
(1)+(1a) Impervious core
(talus & moraine material, (1a) enriched with 1% bentonite).
(2)+(2a) Transition zones (gravel)
(3)+(3a) Shell zones (quarry-run rock and oversize, (3a) mixed with gravel).
(4) River deposits and talus material.

Fig. 3.27. Typical dam cross section and Gradation curves of Gepatsch Rockfill Dam

2. Core Design & its Material

The Gepatsch dam is distinguished, among other rockfill dams, by its steep slopes and high core gradient. Beside that, this is an example of arching central core type dam [121]. The core slope is 0.125H:1V for both upstream and downstream.

The dam core was made of graded moraine deposits, which is mixed (blended) with sandy-gravel talus filler (Gradation Curves in Fig. 3.27).

In order to improve the quality of the quarried morainic clayey earth, the fraction above 80 mm were sorted out, and the earth was treated in special rubber-roller mills. In addition, the earth was dried in rotary drying ovens similar to those used for Goscheneralp dam. Four ovens, each having a through output of 100 t/hr were used for drying of the core material to optimum moisture content of 6.5 – 7%. The so treated earth was conveyed to the dumping site in small carts and then spread by dozers. Rolling was effected by 40 ton rollers that passed six times over each 40 cm thick layer. The compacted dry unit weight of the rolled fill reached 2.15 t/m^3.

Also, the imperviousness of the core was improved by an additional adjoining upstream section (zone 1A), where 1% bentonite had been added to the clayey earth. As a result, high imperviousness was achieved in this thin core, its permeability being $2 \times 10^{-7} - 4 \times 10^{-7}$ cm/sec; and in the section with bentonite it is as low as $2 \times 10^{-8} - 4 \times 10^{-8}$. The core is 40.8 m thick at the bottom, the seepage gradient being 3.7. The angle of internal friction of the core fill, 29°; cohesion, 0.1 kg/cm^2.

Measurements during the first year of construction period showed that the placement water content averaged about 2% above the optimum and the compaction attained an average 96% of the optimum density. With the use of drying plant, the placement water content could be kept as low as 0.5 to 1% above the optimum during the second year, then density attained was 99% of the optimum.

3. *Filter / Transition zones*

The transition zones are constituted of filters of natural alluvium and graded earth. Fractions above 80 mm as sorted out of the core earth at the initial treatment stage, were dumped in 40 – 60 cm layers and rolled by vehiculor traffic. At the contacts of filter with the shell there are thin layers of fine rock recovered from excavations and tunnels and rolled in the same manner as the core earth fractions.

4. Dam Performance

Due to reservoir filling, observations showed an increase in pressure in the core and in the transition zones, this led to an increase in pore pressures in the upstream transition zone and in the core [152].

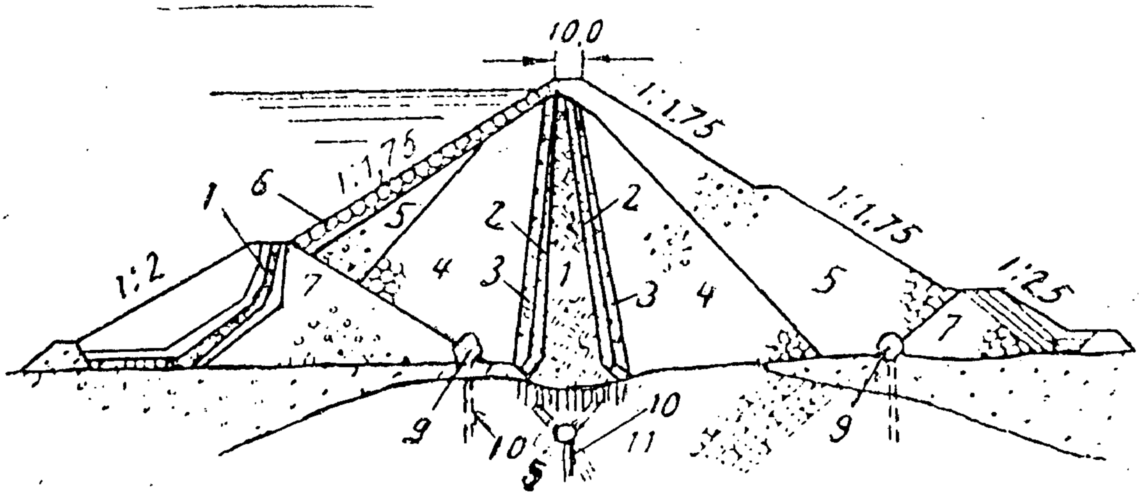
Due to arching of the central core, total pressure measurement indicated that significant stress reduction in the core had taken place [121]. On the horizontal plane 122 m below the crest, the vertical stress in the downstream transition zone adjacent to the core was 27–29 kg/cm^2, while at the adjacent point of the core it was around 11.5 kg/cm^2, indicating an approximate ratio of stresses of about 2.5. An approximate analysis showed that if arching had not taken place, the vertical stress in the core would have been about 22 kg/cm^2. Although the dam has not seriously cracked, it offered very valuable information about the order of magnitude of reduction of vertical stresses in the core due to arching. It was also noted that [121] the cracks did not appreciably affect the performance of the dam.

3.2.9. Infiernillo Dam (without Blended Core Material) [1, 71, 121]

Infiernillo is a rockfill type dam with a thin central core. It intercepts water of the Balsas River 70 km upstream from its out fall in the Pacific Ocean, Mexico. The height of the embankment dam is 148 m and located in a highly seismic zone. The dam was built in 1962 – 1964. The maximum cross section of the dam is shown in Fig.3.28.

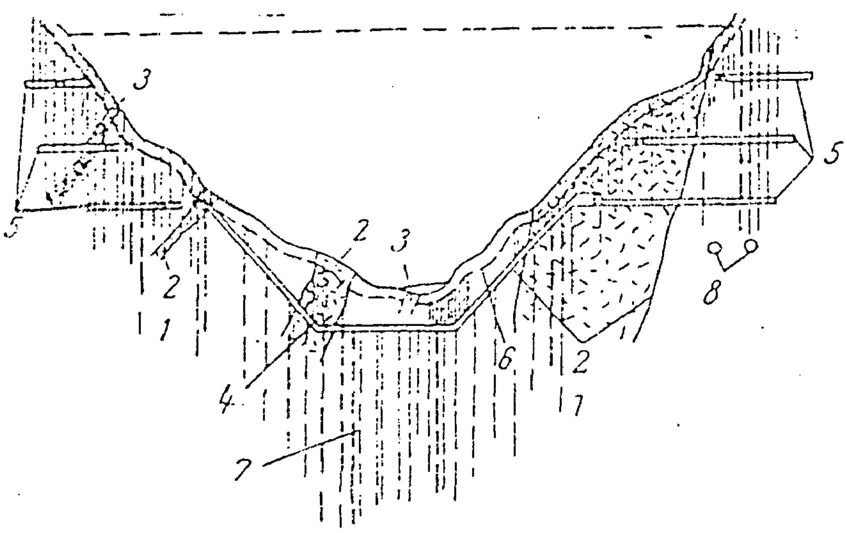
1. Foundation Condition

The dam foundation consists of Tertiary conglomerates, highly modified in some sections and strongly fissured, in particular at the surface, as well as intersected by numerous dikes (Fig.3.29). The fissures are mostly closed up and filled with clay. The conglomerates are overlain, in the riverbed section, by alluvial deposits.



(1) Impervious Core, (2) First Filter, (3) Second Filter, (4) Transition Zone (5) Shell fill of local rock, zonally graded by size, (6) Reinforcement by large rock pieces, (7) Cofferdam, (8) Grout Tunnel, (9) Concrete Plugging, (10) Grout Curtains and surface grouting, (11) Drainage boreholes.

Fig. 3.28. Maximum cross section of Infiernillo dam.



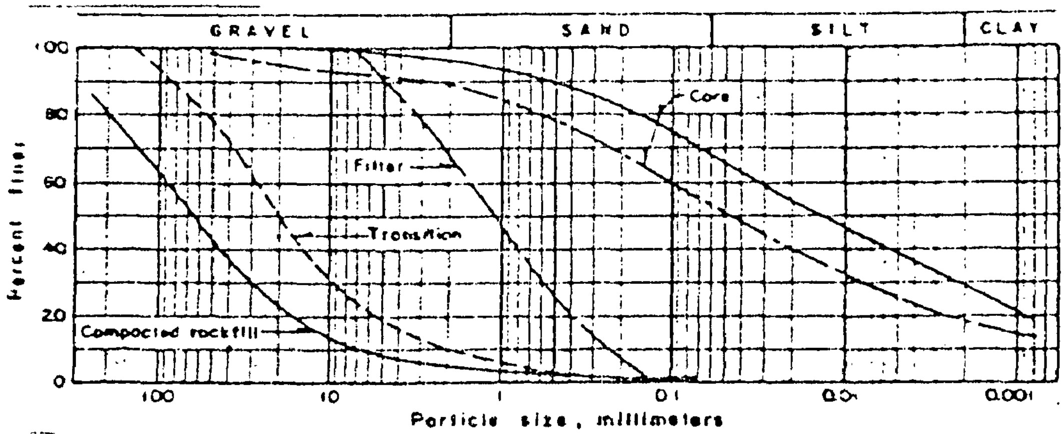
(1) Bedrock-Tertiary conglomerates, (2) Fragmentation zone, (3) Dikes, (4) Grouting & Drainage Tunnels, (5) Drainage Ducts, (6) Limit of surface grouting, (7) Grout curtain, 100m deep, (8) Tunnel.

Fig.3.29. Cross Section of Foundation condition of Infiernillo Dam Site.

At foundation-core contact, all loose material (river bed deposits) and weathered rock were removed, and the foundation was treated by shallow grouting with a grid of drill holes 3 m apart and 8 m deep.

2. *Core Design & its Materials*

The core is 30 m wide in its lower part, but widened to 45 m at the bottom for better junction with the foundation.



Material	Field dry unit weight			Average
	Mean value (kg/m3)	Standard deviation (kg/m3)	Number of determination	k (cm/sec.)
Core	1586	100	1166	5×10^{-8}
Filter	1870	70	112	8×10^{-3}
Transition	2020	140	72	7×10^{-2}
Compacted rockfill	1850	160	65	-
Spread rockfill	1760	120	18	-

Fig.3.30. Index Properties of core and other Materials placed in the dam.

The core was made of sandy clayey silt materials having an average water content 3.7% above the optimum. The specification called for a placement water content from 2 to 4% above optimum, resulting in a degree of compaction of 94% of standard Proctor optimum. Their average liquid and plastic limits were 49 and 24, respectively. The material was excavated from the main borrow pits 18 km from the dam and deposited in a storage pit in layers 50 cm thick, each layer was sprinkled with water. After a minimum period of 15 days, the material was loaded with power shovels and transported to the dam site. The material was placed in layers 15 cm thick before compacted with 15 ton sheepsfoot rollers. Fig.3.30 shows the grain size distribution of core and other zones of dam and the index properties.

3. Filters

The filters, 2.5 m wide, were placed on both sides of the core using a well-graded, washed, and screened sand; its gradation is shown in Fig.3.30. This material was placed in layers 30 cm thick and compacted with a 2 ton vibratory roller; the dry unit weight averaging 1.87 t/m^3.

4. Dam performance.

The filling of the reservoir for the first time caused transverse cracks opened up in the crest, close to both abutments, affecting only the protective cover of the impervious core. No longitudinal cracks were observed.

Fourteen disturbances were recorded from the end of construction of the dam until February 1966. The earthquake of September 1965, had maximum vertical acceleration of 0.08g; recorded at the crest of the dam. The effect of these earthquakes was quite insignificant.

3.2.10. Netzahualcoyotl Dam (without Blended Core Material) [1, 35, 121, 127, & 152]

The 137.5 m high Netzahualcoyotl Dam is located in a narrow gorge of the Grijalva River in the northwest corner of the State of Chiapas, in Mexico. The dam is of rockfill type with moderately thick central core. The region of the dam lies within Mexico's seismic zone. The axis of the dam is arched upstream. The maximum cross section of the dam is shown in Fig.3.31 [35 & 127].

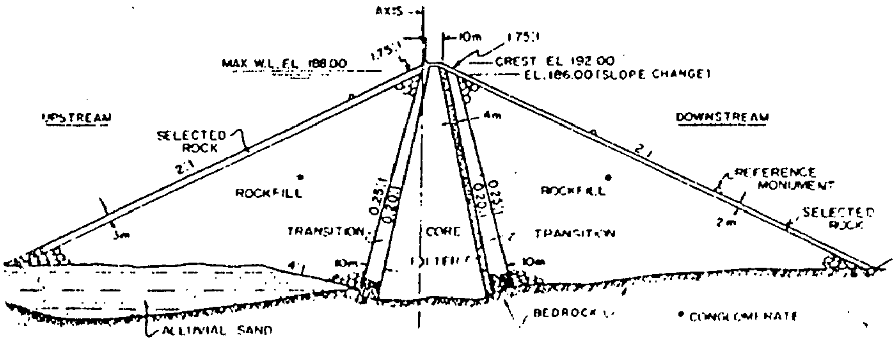


Fig.3.31. Maximum Cross Section of Netzahualcoyotl Dam, Mexico.

1. Foundation Condition

The dam foundation consists of conglomerates and sandstones, river deposits in riverbed section, and other marine sedimentary formations such as shales and limestone. The project rests on a series of folded marine sediments with a well-defined northwest-southeast direction which form part of the Sierra Madre Oriental. The bedrock at the site consists of the Malpao formation which forms thick beds of well cemented conglomerates in an insoluble matrix of sandy clay with lenticular lenses of fine-grained dense sandstone and occasional lenticular layers of compacted shales. Fractures and minor faults resulted from small displacements, were in general well healed and presented no problems [1 and 152].

2. Core Design & its Material

The core material is a sandy silt of low compressibility, and the plasticity index ranging from 10 to 30%. Fig. 3.32 shows the plasticity chart of a representative sample of impervious material and its grain size distribution curve together with other kinds of material for the rockfill embankments. It was placed at a moisture content varying about 7% in excess of the optimum and to an average density of about 94% of the standard Proctor. The optimum water content varied between 16 – 28%

and corresponding compacted dry unit weight between 1.65 to 1.75 t/m^3 [35]. The material was spread in 20 cm loose lifts and compacted with 6 passes of a smooth vibrating roller weighing 4 tons. Appropriate strength tests indicated that the shear strength of the in-place core material was in order of 0.5 kg per cm^2, and internal friction angle of about 22° [35, 121, 127 and 152].

This central impervious core has its width about 7 m at the top and about 60 m at the contact area and slope 0.2H:1V on both sides. The core was founded directly on foundation rock.

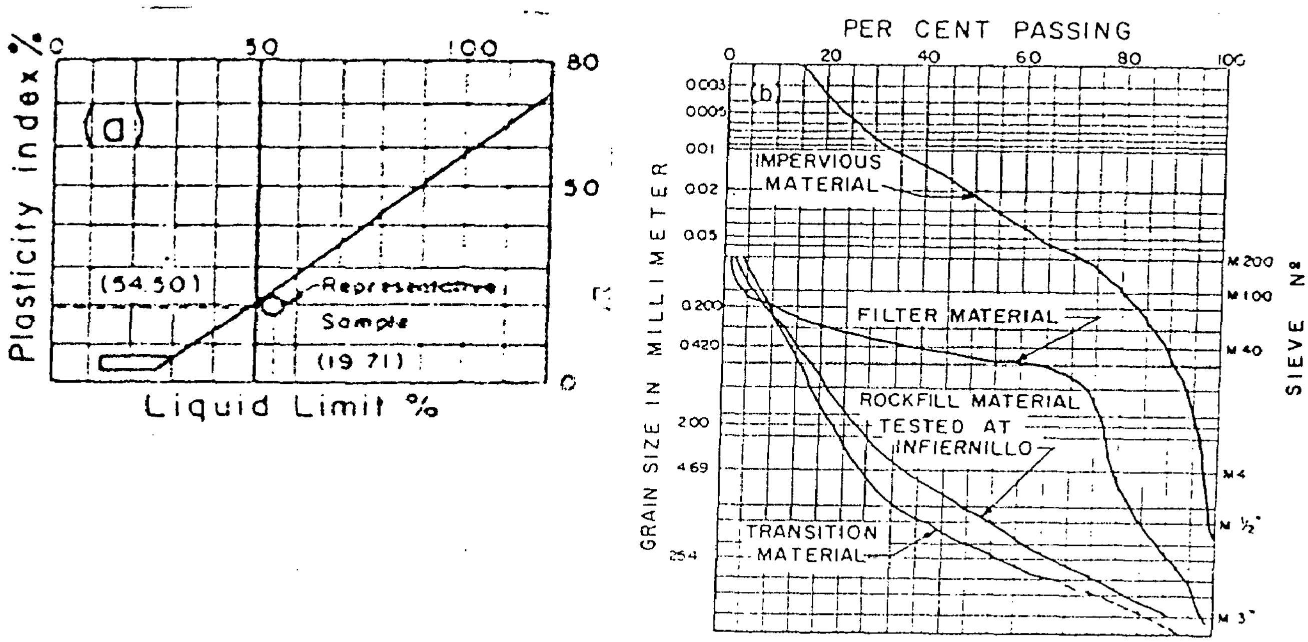


Fig. 3.32 Plasticity chart and grain size distribution curve of Netzahualcoyotl Dam

3. Filters / Transition

The filter was placed in 25 cm layers on downstream side only and compacted with four passes of the vibrating roller of the same specifications as that used to compact the impervious material.

In transition zones on upstream and the downstream, the rock muck obtained from excavations was placed in lifts of 50 cm. Its maximum size was limited to 10 cm. This material was compacted by four passes of a smooth vibrating roller and by routing hauling equipment [35].

4. Dam Performance

The performance of the dam based on observation during 1963 – 1965 showed that deformations did not occur predominantly in vertical direction; and the components of horizontal deformation parallel to the axis and in upstream-downstream direction are of the same order of magnitude as vertical deformations.

Observations showed that pore pressure were not a problem, notwithstanding the fact that the impervious material was placed with a water content 7% in excess of optimum.

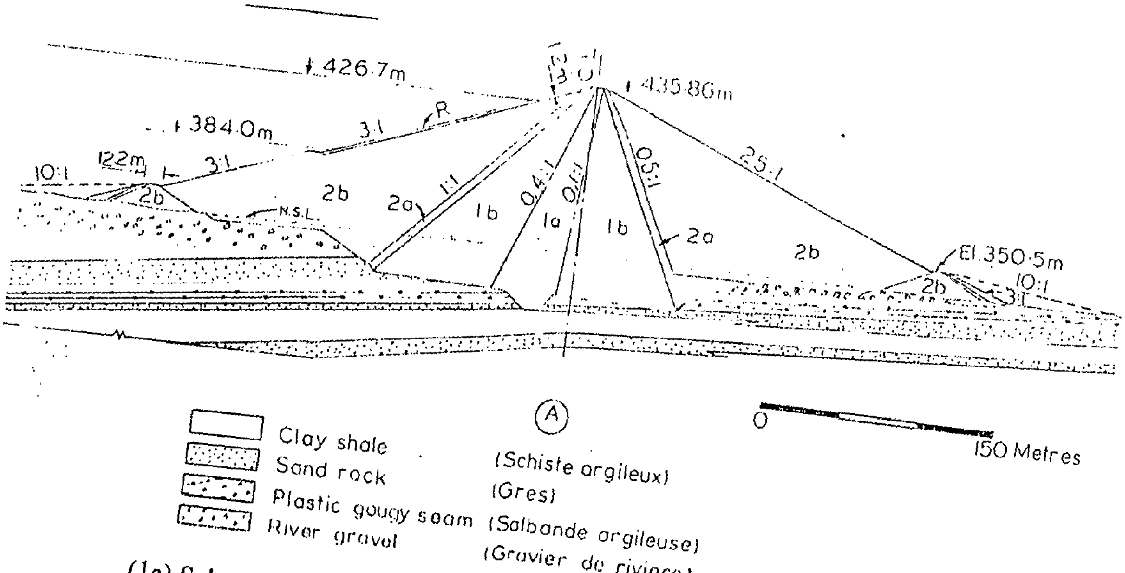
3.2.11. Beas Dam (with Blended Core Material) [87, 109, 121]

Beas Dam, is a multipurpose river valley project, located at a distance of about 483 km North West of Delhi at foothills of the Himalayas in Himachal Pradesh, India. The 132.6 m high dam is an earth core-gravel shell structure for which construction materials being available within economical leads, have been selected from consideration of foundation competency and high seismicity of the region as it would permit deformation without rupture and provide flexibility against unequal settlement in the foundation. The maximum section of the dam is shown in Fig.3.33.

1. *Foundation Condition*

The dam site lies in seismic zone (Magnitude 7 – 8 on Richter Scale). The dam is founded on upper Siwalik formations consisting of sandrock and clayshale / siltstone. These have been subjected to folding.

The clayshale bands are generally massive, well compacted and include silty and sandy portions which show air slacking i.e. breaking into fragments when exposed to air. The sandrock bands are generally coarse grained, massive, friable to firm and are seemingly pervious.



(1a) Selected claystone, (1b) Selected sandrock, (2a) Selected sand & Gravel, (2b) Selected sand, gravel, cobbles and boulders, (R) Rip rap.

Fig.3.33. The maximum section of the Beas Dam, India.

2. *Core Design & its Material*

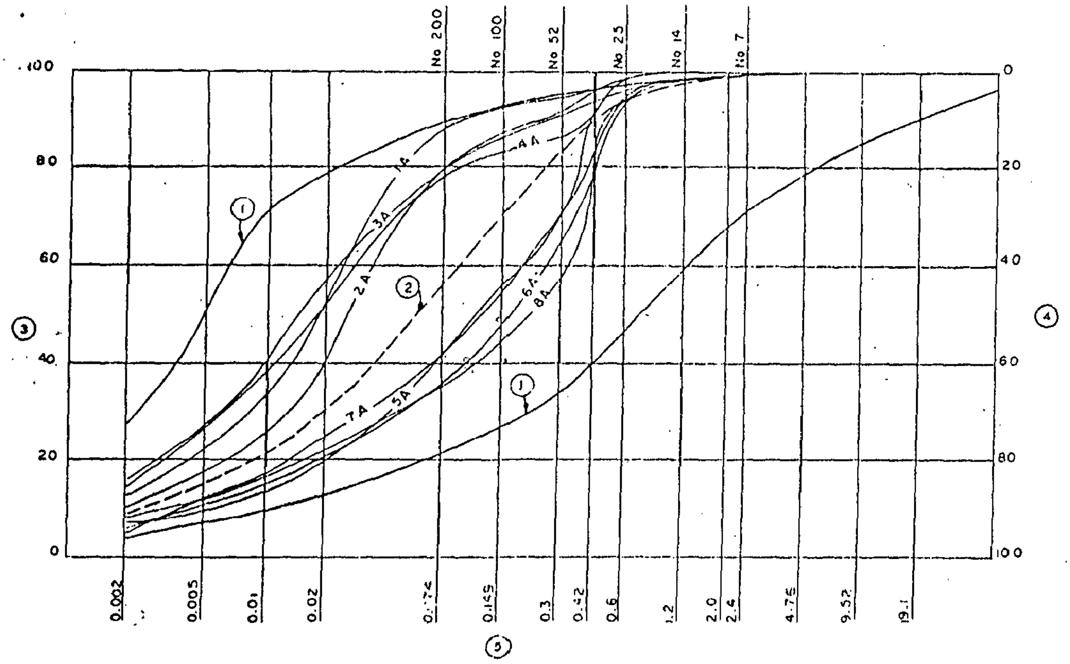
A thick vertical impervious core zone (approximate hydraulic gradient of about 0.7) is made of earth from stock piles of materials excavated from various structures or directly from borrow area. The core is made of two zones, zone 1a and zone 1b. The material available from excavations contained a mixture of sandrock and clayshale in varying proportions. Both the zones of this core are made of mixed (blended) type impervious material.

The material for zone 1a is primarily the claystone containing less than 50% sandrock, and zone 1b is primarily of sandrock containing less than 50% claystone.

Tests were conducted on several mixed (blended) samples and the results showed that material is generally finer than 2 mm; the material is easily compactible and provides an average dry density of 1.94 t/m^3; the permeability is of the order of 10^{-6} cm/s or less; angle of internal friction averages to 32.3° against a design value of 26.5°; average value of cohesion is 0.4 kg/cm^2; average field moisture content was 10.8% (OMC is 11.86%);

and pore pressures were not very high. The material was spread in 20 cm thick layers and compacted to 15 cm with 6 set of sheeps-foot roller moving in lanes parallel to the axis of the dam, to ensure the rolling of layer by 12 coverages.

The grading curves for eight samples (four for zone 1a and four for zone 1b of the dam) are plotted as per grading analysis along with the limiting grading curves in Fig.3.34.

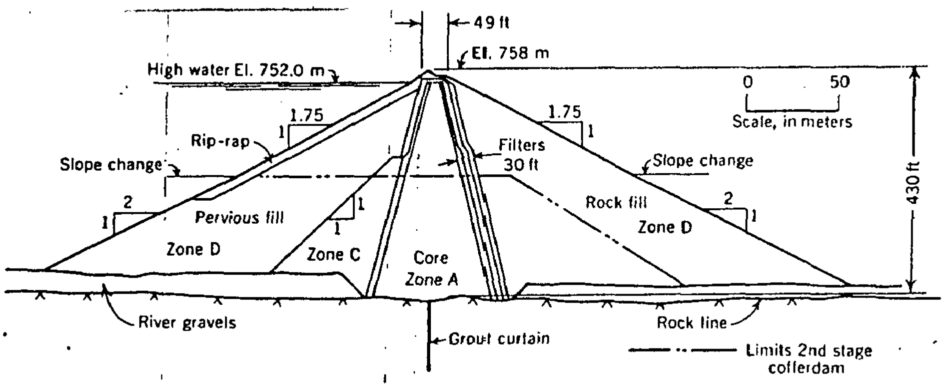


(1) Limiting grading curve, (2) Line of demarcation between 1a and 1b material, (3) Percent finer than, (4) Percent coarser than, (5) Size in millimeters.

Fig.3.34. Grading Curves for the core material of Beas Dam, India.

3.2.12. Ambuklao Dam (without Blended Core Material) [1, 34, 121]

The Ambuklao Dam was built in 1956 in Philippines, on Agno River, Island of Luzon. The earth & rockfill dam is 430 ft (131 m) high with a thin vertical core. The section of the dam is shown in Fig.3.35. The region of the dam has a tropical climate with abundant rainstorm precipitation.



(Zone A): Impervious core; (Zone C): Quarry waste; (Zone D): Pervious rockfill;

Fig.3.35. Maximum Section through the Ambuklao Dam, Philippines.

(1) Foundation Condition

The dam site is a narrow canyon between two ridges; the western ridge being composed of metamorphic rocks and the eastern ridge of both metamorphic rocks and diorite (see Fig.3.36). Above the gorge the river is in a relatively wide valley which will form the reservoir basin. Below the gorge the river flows in a narrow meandering valley.

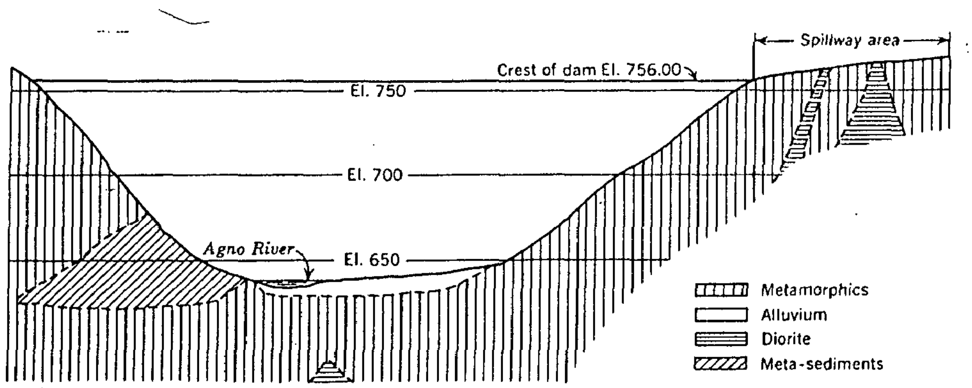


Fig.3.36. Cross section of Ambuklao dam sites, Philippines.

The foundation rock is weathered to a considerable depth and has faults & fissures, one of which diagonally intersects the riverbed. The canyon bottom is covered by pebble-type deposits. Faults & fissures within the foundation were washed, and plugged with concrete. The core cuts through riverbed deposits and joins the bedrock, from which the weathered upper layer has been carefully removed. The grout curtain reaches to a depth of 90 m [1 and 34].

(2) Core Design & its Material

A thin vertical impervious core meeting safety & economic consideration has been adopted.

The core material is of quarried loam. The grain size curve of core material and other zones are shown in Fig. 3.37. The bottom width of the core is 65 m. The angle of internal friction of the core material was found as 15° by triaxial compression test. It was deposited in 45 cm layers and rolled by 50 ton pneumatic-tire rollers (in four passes).

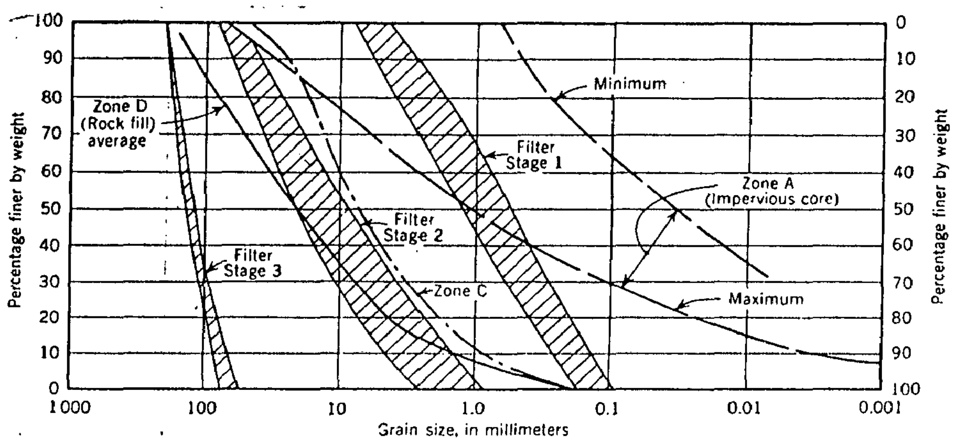


Fig. 3.37. The grain size curve of core material and other zones of Ambuklao Dam

(3) Filter / Transition

There are filters between the core zone and pervious rockfill zone in two layers in upstream and three layers in downstream. The grain size composition complying with the filter requirement is shown in Fig.3.37.

(4) Dam performance

Observations have indicated that the dam performance is satisfactory. Little seepage is observed through the left abutment ridge, indicating that blanket on the upstream face of this ridge is performing well. Meanwhile it was also noted that seepage losses through the core were nominal, the crest settled by only 15 cm, horizontal displacement amounted to only 7.5 cm [1 and 34].

3.2.13. Mornos Dam (without Blended Core Material) [1, 34, 121]

The 130 m high Mornos dam is an earthfill type with central impervious core. The dam is situated in mid-western Greece, 7 km west of Lidorikion, and the purpose is to meet the rapid increase in demand for water in the Athens-Piraeus area of Greece. Its surrounding climate is not typical of a Mediteranian area. The region forms a transition zone between the relatively dry climate of eastern Greece and the more humid climate of the western parts of the country. The medium annual precipitation is 1370 mm and the average discharge in Mornos River is about 11.8 m^3/sec. The typical cross section of the dam is shown in Fig.3.38.

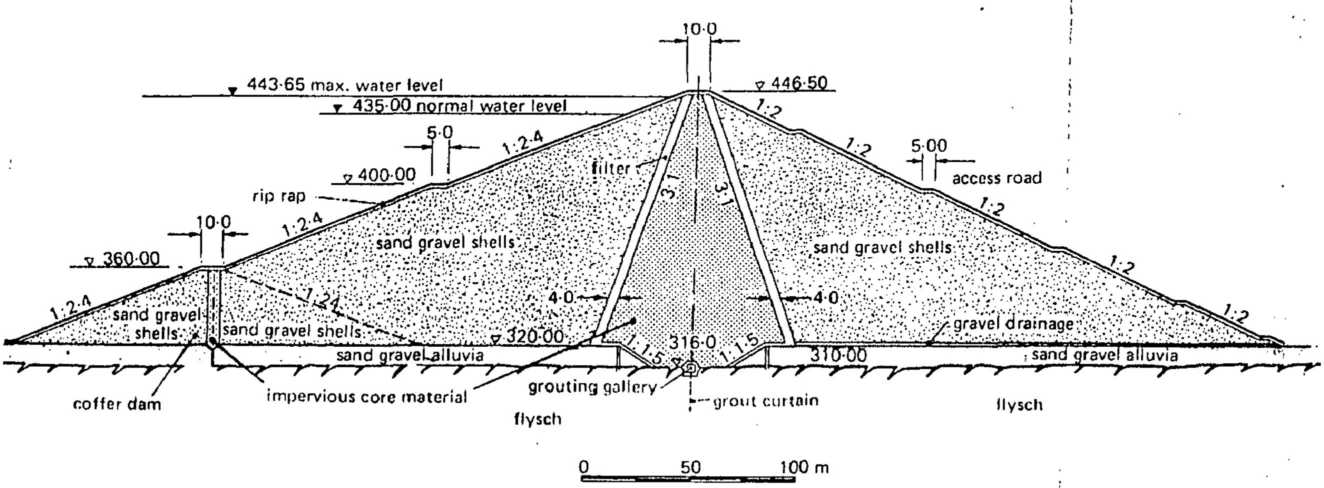


Fig.3.38. The typical cross section of the Mornos Dam, Greece.

1. Foundation Condition

The dam site consists of typical flysch rocks, grey-brown siltstones, and fine to medium-grained sandstones with sporadic conglomerate layers. The whole dam site area is marked by a complex tectonic structure. The tectonic structure has an intensive plate characteristic with strongly folded syncline and anticline structures. Small and medium sized faults in various directions are evident in the rock mass of the dam site area, and in the slopes.

2. Core Design & its Material

The dam has a moderately thick vertical earth core, with a base width of about 95.0 m. The slope of the core is 0.33H:1V both in upstream and downstream side. The core material is taken from primary and secondary deposits of weathered flysch from different borrow areas on the slopes of the Mornos and Kokkinos valleys.

The parameters of the core material were obtained from extensive laboratory tests, and these are as follows, (a) internal friction angle: 26° ; (b) cohesion : 2.4 t/m^2; (c) compacted dry unit weight : 2.09 t/m^3; and (c) permeability : 1x 10^{-9} cm/s. Other parameters are shown in Table 3.2.

Table 3.2. Properties of core material & other materials of Mornos Dam

	Friction angle ρ (°)	Cohesion c^1 (t/m^2)	Unit weight proctor γ_d(t/m^3)	Unit weight saturated γ_{sat}/m^3	Permeability k (m/s)	Factor of pore-water pressure B (–)
Riprap	45	—	2·40	2·40	1×10^{-5}	—
Shell filter mat.	42	—	2·10	2·32	1×10^{-5}	—
Core material	26	2·4	2·09	2·14	1×10^{-9}	0·5
Alluvial layers	37	—	1·90	2·10	1×10^{-5}	—
Flysch weathered in sliding zones	34	—	2·30	2·40	10^{-5}-10^{-6}	0·6
Flysch less weathered outside sliding zones	40	—	2·30	2·40	1×10^{-6}	—

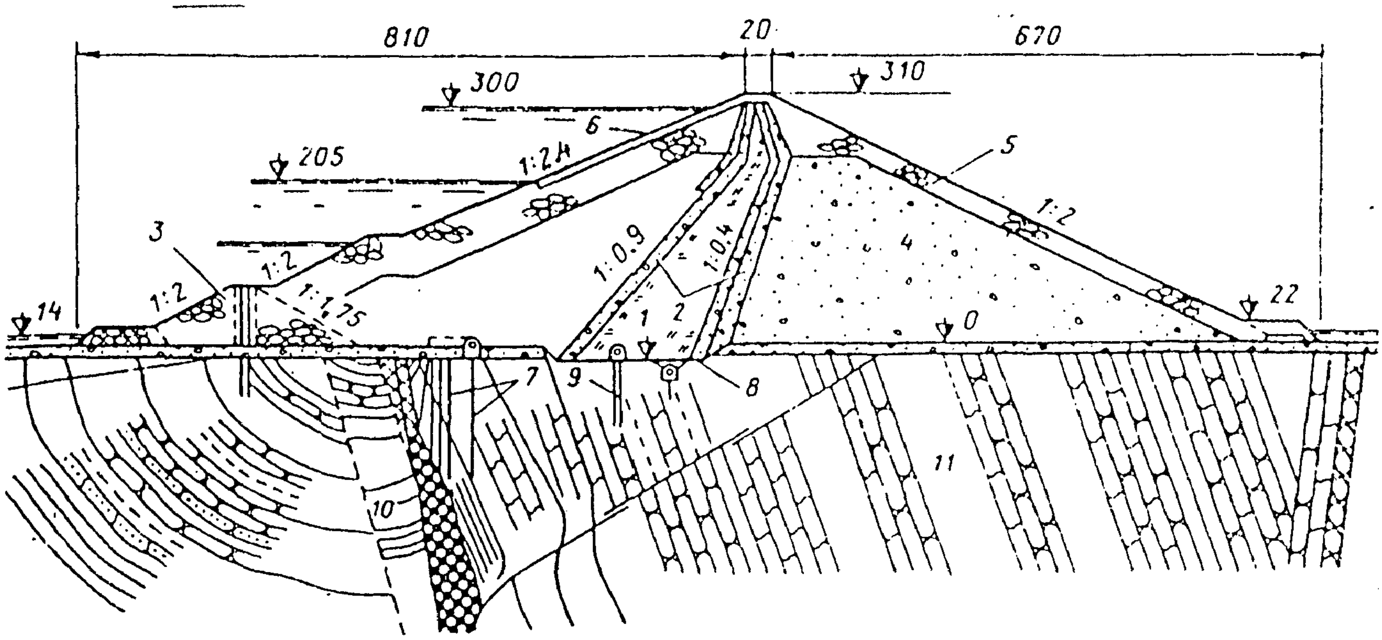
Along the dam axis below the core base an inspection and grouting gallery is provided, branching to horizontal grouting tunnels on the abutments.

Blanket grouting under the core bottom, contact grouting around foundation structures and drainage holes have bee provided. The preparation of core-abutment contact included the covering of foundation surface and the top of the gallery with cement bentonite paste layer.

3.3. Moderately Inclined Core Embankment Dams

3.3.1. Rogun Dam (with Blended Core Material) [36a , 77 and 121].

Rogun the highest in the world is 335 m high earth and rockfill dam with moderately inclined core [77]. It is located on the Vakhsh River in Tadjikstan. The site is in a zone of high seismic and tectonic activity and harshest environments; the temperature ranging from a maximum of 40°C to a minimum of -31°C; annual precipitation reaching 816 mm; and in winter months (December to February), the average depth of snowfall is 61cm. The maximum section of the dam is shown in Fig.3.39.



(1)Core; (2)Transition Filters; (3)Upper Construction Cofferdam with injection Core; (4)Trust Prisms; (5)Stone Revetment; (6)Reinforced-Concrete Revetment; (7)Leaching Prevention Measures; (8)Area Grouting & Concrete Sputtering at Core Base; (9)Deep Grout Curtain; (10)Rock Salt Stratum; (11)Inter-layers of Sandstones and Aleurolites.

Fig.3.39. Design of Rogun Dam, Maximum Section.

1. Foundation Condition

The dam site is a narrow, S-shape canyon of Vakhsh River, where slopes reach 400-700 m above water level at an angle of 50 - 55°. Gorge is composed of sandstone, aleurolite and argillite, dipping steeply in downstream at an angle of 65 - 80°.

The site is characterized by complex geologic and tectonic conditions. Large Ionakhsh fault filled with a salt stratum, is very close to the site, and secondary faults are also present.

A two-row grout curtain of 60 – 80 m maximum depth has been provided along the core.

2. Core Design & its Material

A moderately thick and moderately inclined core has been provided. The core slope is 0.9H:1V at upstream side and -0.4H:1V at downstream. The configuration of core as well as the dam centerline is curvilinear in plan. Its dimensions are dictated by the permissible gradients in the core, and the contact of the core with the foundation. The hydraulic gradient (H/B) is about 2.0. The core inclination and its arch configuration in plan ensure that the greater part of the core and upstream shell is located beyond the salt layer.

The core material is composed of processed mix of natural loam and pebbles. The properties of the loam are: $\varphi=17^{\circ}$, average c=o.20 kg/cm^2, and average $k = 2 \times 10^{-5}$ cm/s.

The natural loam and pebble material are obtained from the borrow pits. The material is conveyed to the dam by a complex material handling system consisting of a receiving plant, conveyor line and delivery end. For the loam, the material is delivered to the core zone from semi-stationary hoppers installed on the downstream slope of the dam after receiving from the 2.6 km long main conveyor. The pebble from its 7.2 km long conveyor lines is stored in a storage hopper within the dam zone. Both materials are hauled to the core zone by 40 ton dump trucks. Then, the Crawler-mounted bulldozer are used for leveling both the materials in such a manner that mixed (blended) composition as per requirement is achieved. The 22 ton vibratory rollers are used for layer by layer compaction.

The main parameters of the conveyor lines for delivering of core materials at construction sites are shown in Table 3.3.

Belt conveyor for earthwork handling is a rational and economically efficient technique for the conditions at the Rogun dam site. It reduced the number of heavy dump trucks, personnel and, hence, the cost; and avoided access tunnel.

Table 3.3. Main parameters of the conveyor lines for delivering of Core Materials.

Parameters	*Unit of mesurement*	*Conveyor lines*		
		From pebble borrow pit to dam.	*From stone quarry to dam.*	*From loam borrow pit to dam.*
Conveyor belt width	(mm)	2000	2000	1200
Conveyor belt speed	(m/s)	2	2	2.5
Size of Material up to	(mm)	500	500	200
Length of line	(km)	7.2	3.6	2.6
Conveying Capacity	(m^3/h)	2000	2000	500
Volume of material to be delivered	($10^9 m^3$)	39.7	16.35	7.5

3. *Filter / Transition*

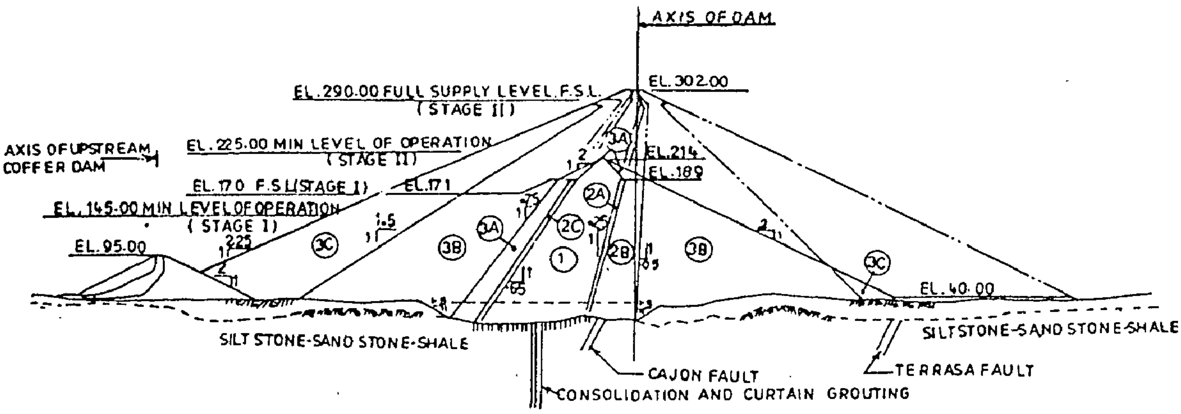
The filter is provided at both sides upstream and downstream of core, but the transition zone is provided only at downstream side.

4. *Progress of the project*

The dam is yet to be completed.

3.3.2. Boruca Dam (with Blended Core Material) [49a, 121].

The Boruca Dam is situated in the southern part of Costa Rica near its border with Panama. A moderately inclined core is adopted for this rockfill embankment dam which is 302 m high, after considering the topographical and geotechnical characteristics of the site and the economic aspects. The maximum cross section of the dam is shown in Fig. 3.40.



(1)Impervious core, (2A) Filter, (2B) Transition, (2C) Filter,
(3A) Transition, (3B) Rockfill shell 1st stage, (3C) Rockfill shell 2nd stage.

Fig.3.40. Maximum Cross Section of Boruca Dam, Costa Rica.

1.Foundation Condition

The dam is located in a narrow gorge cutting through a set of rock formations including interbedded shales and siltstones on upstream and downstream sides with limestones in between. Two active faults pass through the proposed axis of the dam. The site also falls in the zone of high seismicity.

2. Core Design and its Material

A moderately inclined core having slopes of 0.65H:1V and -0.25H:1V on upstream and downstream side respectively is provided. The moderately thick core has been adopted to provide a hydraulic gradient of about 2.0.

The core material is chosen on the considerations of permeability, strength, plasticity and erosion resistance. The core material was produced from mixing (blending) alluvial deposits of silt, sand and gravel with little or no plasticity. The most suitable source was in flood plains of the Rio

Grande de Terraba. This material is normally well graded as it is obtained from the erosion of the volcanic and intrusive rocks of the Talamanca Range. The maximum dry density of the material is 2000 to 2200 kg/m^3. The moisture content varies with the seasons but it is generally slightly lower than the optimum in dry season.

The core was mainly founded on upstream shale complex with small part on the limestone. The alluvium under the core, filter and transition area was excavated and to ensure water-tightness, consolidation grouting under the core and filter was done to a depth of 15 m with holes 3 - 5 m apart. Three rows for grout curtain were also located to a depth about one-third of reservoir head.

3. Filter / Transition

The filters and transitions are provide on both upstream and downstream side of the core. The downstream transition is more thicker than upstream transition. These filters and transitions were made by processing alluvium.

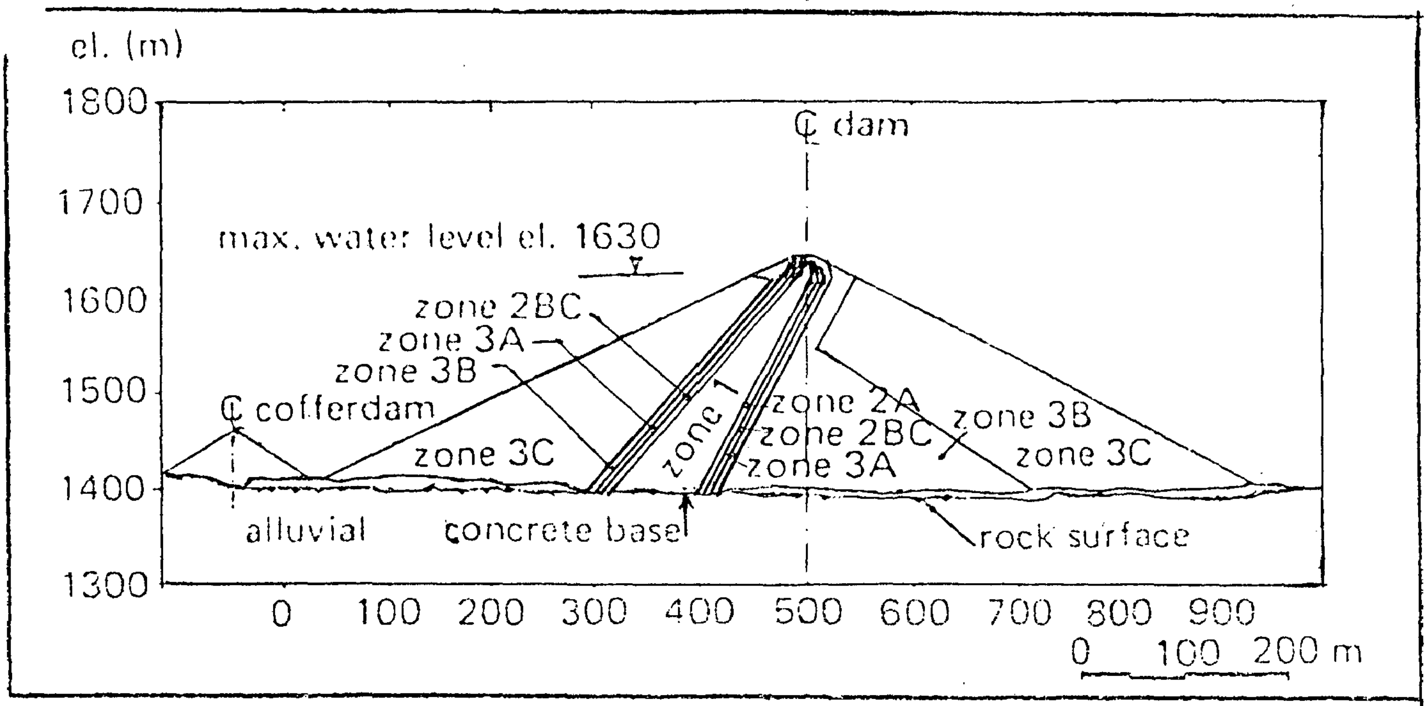
3.3.3. Guavio Dam (with Blended Core Material) [49a, 72 & 109].

Guavio Dam is 245 m high earth and rockfill Dam in Colombia. It was completed in 1988. The main section of the dam is shown in the Fig.3.41.

1. Foundation Condition

The dam is located in a narrow V-shaped valley with L/H=1.6. The canyon is about 600 m deep. The lower half of the canyon comprises Paleozoic rocks, the upper half part comprises Cretaceous rocks which overlay, on a geological unconformity, above the Paleozoic rocks.

This unconformity was found near the surface with a high degree of alteration, filled with silty and clayey soils. These soils of low plasticity and low permeability found in layers 2 m thick, could not be grouted. These were easily erodible under any concentrated water flow. These soils were removed by excavating a gallery following the plane of the deep grout curtain. This gallery was then filled with concrete. No faults exist in the foundation of the dam. Some minor folds have been found at the upper part of the left abutment.



(Zone1): Impervious Core; (Zone2A & 2B): Filter;
(Zone2C): Transition; (Zone 3A, 3B & 3C): Rockfill

Fig. 3.41. Section of the Guavio Dam, Colombia

The core and filters were laid over the rock surface, removing alluvial deposits about 7 to 10 m thick. Along the core length deep grout curtain was located at the upper third of the core width. The grout holes were deeper than 80 m. The foundation / geology of the dam site is shown in Fig. 3.42.

2. *Core Design and its Material*

A thin moderately sloping core has been adopted for this dam with core slopes of 0.85H:1V for the upstream face and -0.5H:1V for the downstream face. Its hydraulic gradient is about 2.85.

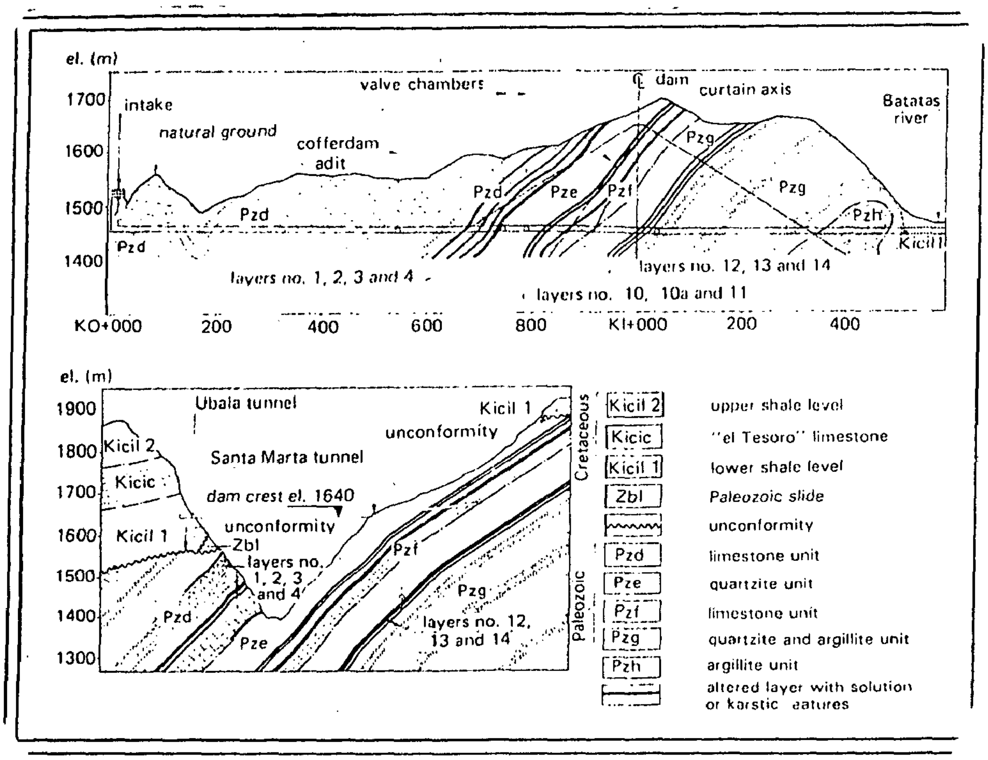


Fig.3.42. Foundation / Geology condition of the dam site, Guavio Dam.

The core material consists of shale fragments within a silty-clayey matrix (blending). The material was stock-piled together for drying purposes in order to reduce its natural water content from about 20% to about 15% (the optimum being about 12%) for achieving specified density. The processing and stock-piling was done before the start of each dry season. The stock-piles were constructed with adequate drainage slopes.

Other steps of the drying process are construction of deep drainages in the borrow area, removal of top 1 m thick layers, harrowing the surfaces of the material to be exposed to atmosphere to reduce moisture content to about 17%, and during hauling and spreading at dam site the material reached a water content of about 15%. In planning the construction program, the rainy and dry seasons of the area were taken into account. The gradation curves of the core material and other zones are presented in Fig.3.43.

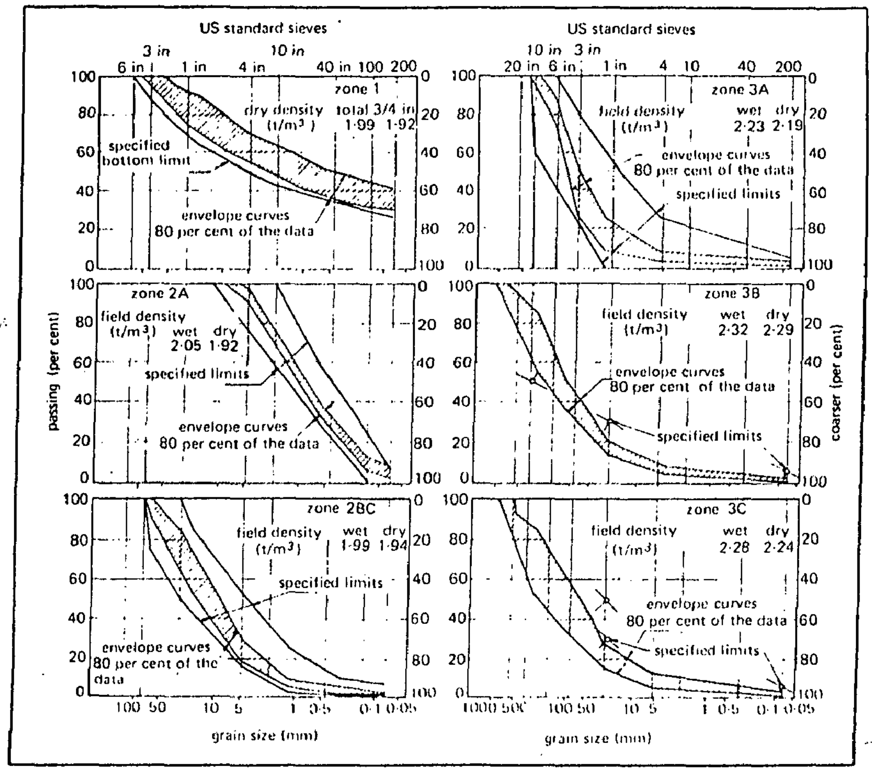


Fig.3.43. Gradation curves of the core material and other zones of Guavio Dam.

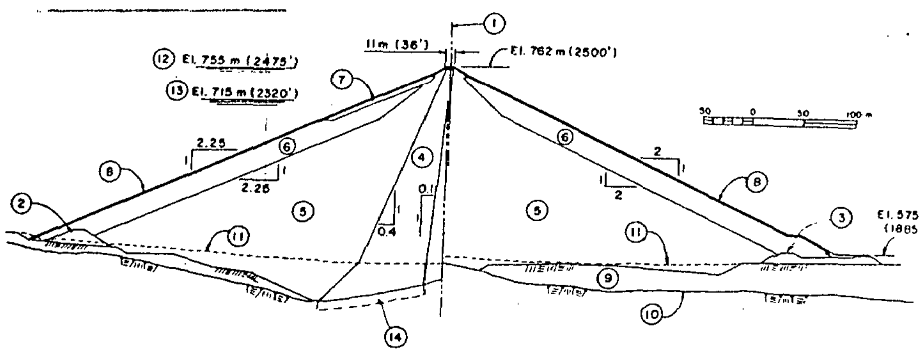
The core was compacted to dry unit weight of 1.96 t/m^3 by making 8 coverages of 10 ton vibratory roller on layers of 20 cm thickness. Compaction requirements (an average of 95% of the maximum of the Modified Proctor test up to el.1600 and 90% up the crest of the dam) led to drying of the core material before placement.

The core is placed on sound rock surface after removing weathered layer. The rock in the area of the core was consolidated by surface grouting and a deep grout curtain is provided.

To prevent cracking of the core measures were taken such as: (a) reduction of the natural water content of the core material before compaction, (b) provision of inclined core which will remain under compression as a result of the weight of the upstream shell, (c) conservative double filters in downstream scope of core, (d) use of wetter and more plastic material in contact with the abutments and near the crest of the dam.

3.3.4. Mica Dam (with Blended Core Material) [30, 36a, 109, 121 and 149].

The 244 m high Mica dam is located on the Colombia River in British Colombia in mountainous terrain immediately west of the Rocky Mountains, Canada. The axis of this earth and rockfill dam is slight curved upstream so that deflection of the crest downstream under reservoir loads will tend to increase the compression parallel to the axis in core, and thereby will reduce the tendency for the formation of transverse cracks. The dam was built between 1969 – 1972. The cross section of the dam is shown in Fig.3.44.

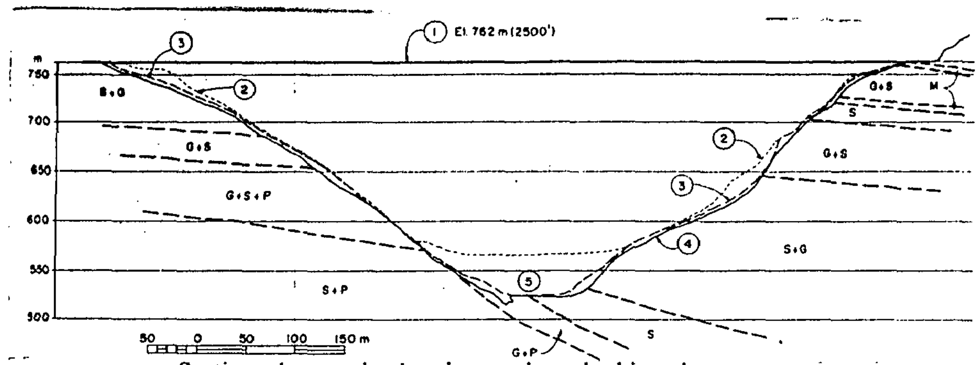


(1) Axis of dam and break in section.
(2) Upstream cofferdam.
(3) Downstream cofferdam.
(4) Core - glacial till in 25 cm (10") layers.
(5) Inner shells — sand and gravel in 30 cm (12") layers.
(6) Outer shells - sand and gravel or rock in 60 cm (24") layers.
(7) Drawdown zone - gravel, cobbles, and boulders or rock in 60 cm (24") layers.
(8) Surface protection.
(9) River overburden.
(10) Bedrock.
(11) Original riverbed.
(12) Normal maximum reservoir level.
(13) Minimum drawdown level.

Fig.3.44. Cross section of Mica Dam, Canada.

1. Foundation Condition

The bedrock in the narrow gorge consists of a Precambrian series of mica schists and granitic gneisses containing discontinuous intrusions of pegmatite. There is no major fault, but numerous minor faults which exist are filled with hard and brittle material, and gouge. Thin overburden on the abutments such as sand, gravel, and talus, existed especially on the right abutment. The cross section along dam axis showing geology looking downstream is shown in Fig.3.45.



Section along axis showing geology looking downstream.
Note: Where more than one rock type shown, first predominates.

(1) Crest of dam.
(2) Original ground surface.
(3) Original bedrock surface.
(4) Final excavated surface.
(5) River overburden.

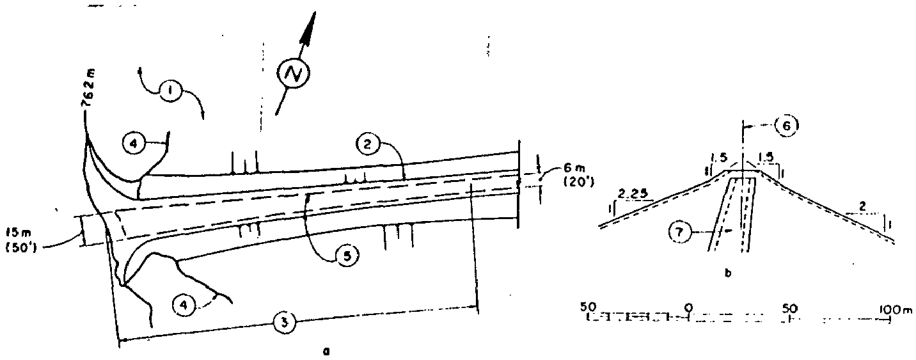
(G) Granitic gneiss.
(M) Marble.
(P) Pegmatite.
(S) Mica schist.

Fig.3.45. The cross section along axis of Mica Dam, showing geology looking downstream.

2. Core Design and its Material

A moderately inclined core [109] was provided to ensure stability and reduction in deformations under earthquake loading condition. The core was made wider than originally planned to reduce the possibility of propagation of any transverse or longitudinal cracks. It was also widened towards each abutment in a length of 168 m, i.e., in the area where longitudinal tensile strains are most likely to occur. It is shown in Fig.3.46.

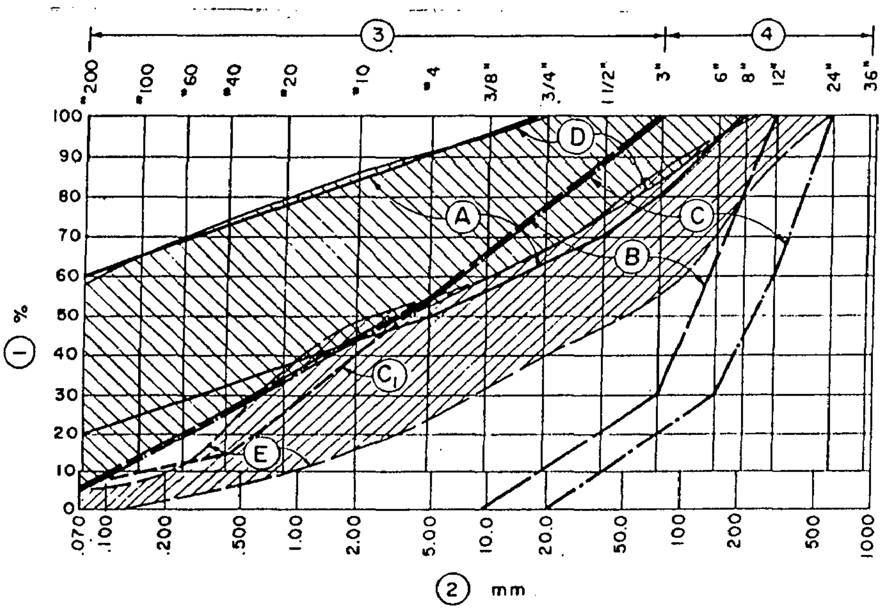
The relatively thin core (hydraulic gradient about 2.3) is provided. It consists of glacial till material from borrow area within 16 km of the dam site. These borrow areas contain large quantities of well-graded materials. Glacial till* was preferred because of its higher shear strength, lower compressibility, good compactability at close to natural moisture content, excellent self-healing characteristics, and relatively good pore pressures dissipation. (*: The term Glacial till [119] is defined as: *a). Glacial* deposits include materials transported and re-deposited by glaciers or by melting of glaciers; *b) Till* or boulder clay matrix is unstratified, heterogeneous mixture of clay, silt, sand, gravel and boulders, directly deposited, without transportation or sorting by water).



(a) Plan at right abutment.
(b) Typical sections.
(1) Right abutment.
(2) Crest of dam.
(3) Core widened over 168 m (550').
(4) Outline of dam.
(5) Top surface of core.
(6) Axis of dam.
(7) Core.

Legend for (b): ———— Section where crest width flares. — — — — Section where crest width maintained constant.

Fig.3.46. Core Widening at abutments, Mica Dam



(1) Percentage by weight finer than.
(2) Particle size in mm.
(3) U.S. standard sieve size.
(4) Screen size square opening.
(A) Specified limits for core.
(B) Specified limits for inner shells.
(C) Specified limits for outer shells-sand and gravel (C_1 = rockfill).
(D) Envelope for 243 tests on minus 3-inch glacial till from Area A.
(E) Envelope for 32 tests on sand and gravel from Area B.

Fig.3.47. Gradation of core material and other zones of Mica Dam.

The properties of the core material (glacial till) were tested in laboratory. On the basis of minus 20 mm size, the coefficient of permeability was found to be about 10^{-7} cm/sec., the average maximum dry density as 2150 kg/cm3, and optimum and natural moisture content around 9% and 12% respectively. The average liquid limit and plastic limits based on the minus 0.5mm size are 26% and 21% respectively. In drained condition, the peak strengths were essentially constant at φ max = 34°, with only minor variations for changes of moisture content within ± 2% of optimum. The core thickness is about 81 m. The core was compacted in the main body of the dam in thickness of 25 cm by 4 passes of rubber-tired rollers and it resulted in an average dry density of 2100 kg/m^3, based on minus 2 cm size. The gradation curves of the core material and other zones are given in Fig.3.47.

3.3.5. Chivor Dam (with Blended Core Material) [3, 104 and 121].

Chivor (Esmeralda) dam, 237 m high, is located about 120 km north east of Bogota, Colombia, South America. This dam, rockfill earth core type, was completed in 1975. The plan and maximum section of the dam as well as three cross sections through the core are shown in Fig. 3.48.

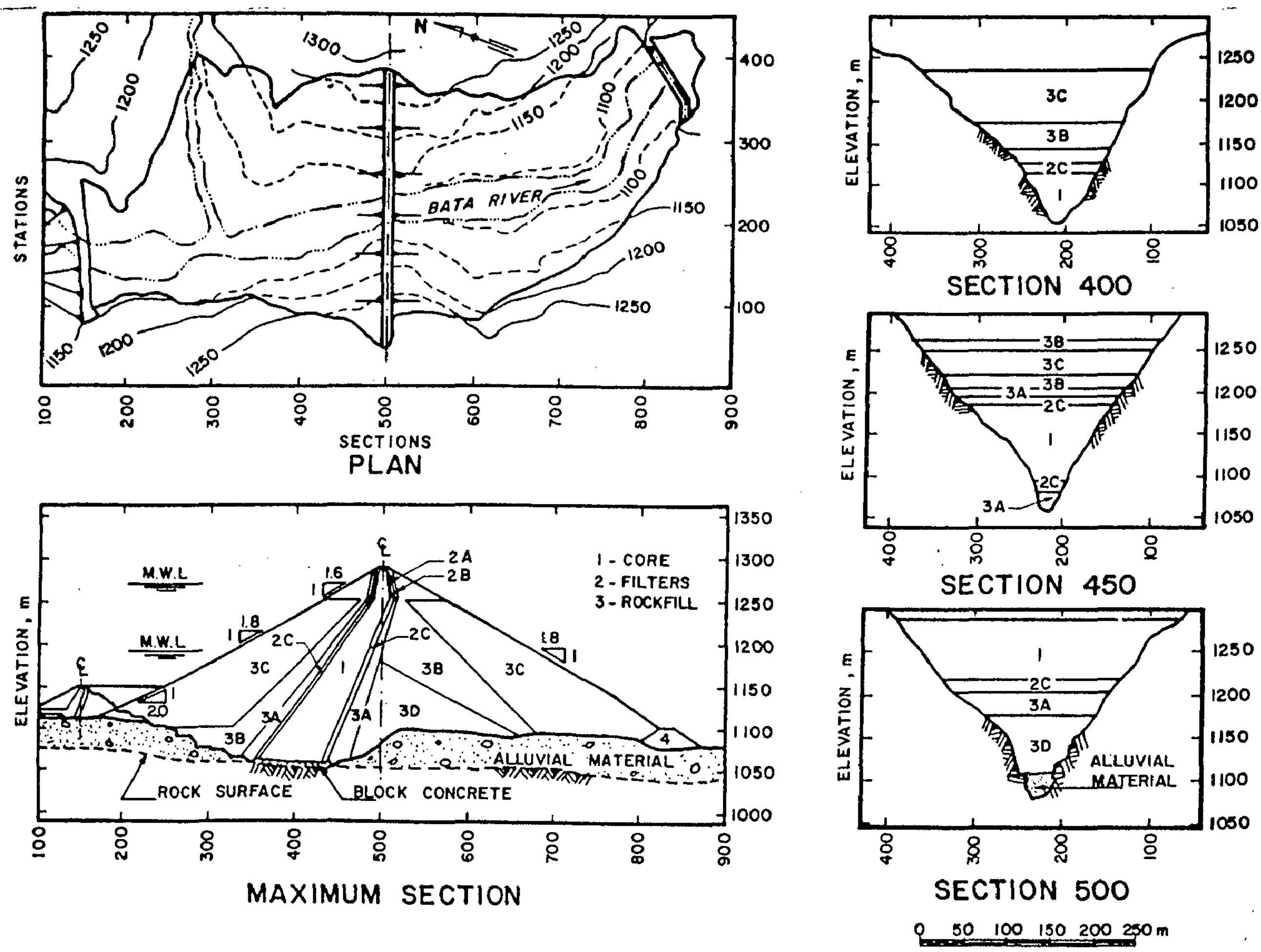


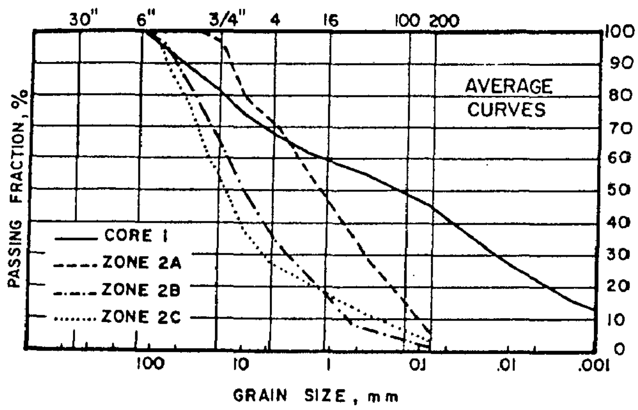
Fig.3.48. Chivor Dam – Plan and Sections (in Colombia).

1. Core Design & its Material

The dam has a moderately inclined earth core. Its maximum hydraulic gradient is about 3.0. The core slope is of about 0.65H:1V at upstream side and -0.38H:1V at the downstream.

The core material is constituted of rock fragments up to 15 cm in size in a silty clayey matrix (blend) of medium plasticity, coming from the old slide. The material was placed and compacted at its natural moisture content which ranged from 17 to 21%, which was on an average 7% above the optimum moisture content determined by the Modified Proctor Compaction Test, for the fraction passing N°4 sieve. Data on moisture content, density, grain size distribution, and characteristics of plasticity of the core material are presented in Fig. 3.49.

The core material was not placed during the rainy seasons due to high precipitation conditions at the dam site (average annual rainfall of 3500 mm).



CORE MATERIAL CHARACTERISTICS

CONSTRUCTION DATA	MOISTURE CONTENT PASSING N°4 SIEVE	DRY DENSITY PASSING N°4 SIEVE	LIQUID LIMIT	PLASTIC. INDEX	% OF COMPACT-ION	OPTIMUM MOISTURE
N° OF TESTS	897	883	392	392	882	391
MEAN VALUES	18.9	1.82	44	20	94	12.9
STANDARD DEVIATION	1.0	0.11	2.5	2.2	5.6	1.0
COEF. OF VARIATION	4.8	6.0	5.7	11.0	6.0	7.7

Fig.3.49. Chivor (Esmeralda) Dam – Core and Filter Material Characteristics.

2. Filters

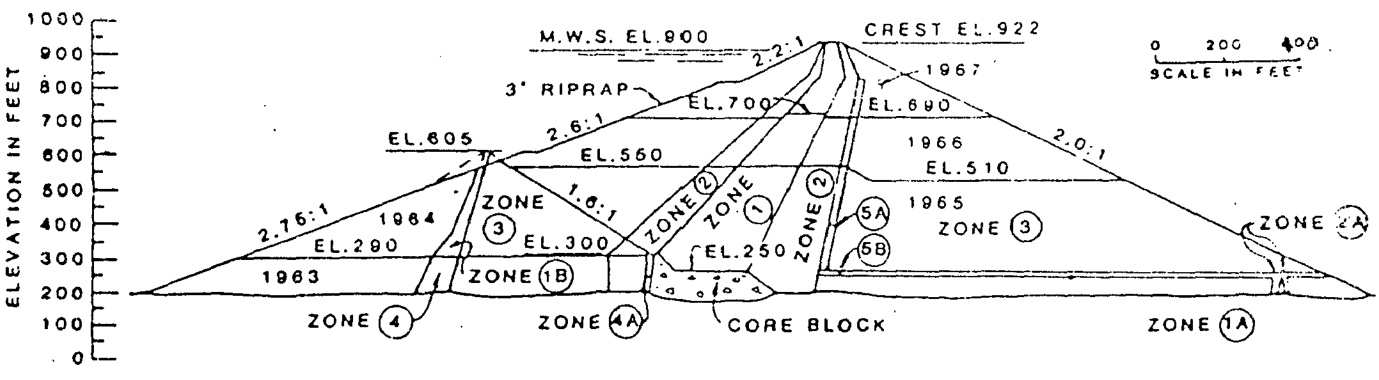
Filters zones were built at both upstream and downstream slopes of the core. Most of the filter is constituted by a well graded material up to 15 cm in size (Zone 2C), with a minimum width of 4 m. In the upper 40 m of the dam, a two layer filter was used (Zone 2A and 2B). Grain size distribution of the filter is also presented in Fig.3.49.

3. Dam Performance

The core of Chivor dam was built with materials compacted at about 7% above the optimum moisture content. Therefore high pore pressures developed during construction, and these could not be fully dissipated even after four years of completion. The maximum pore pressures occurred at about elevation 1150, where materials were placed at a rather high moisture content. Prior to the first reservoir filling, which took place about one year after completion of the dam, the construction pore pressures had dissipated by about 30%.

3.3.6. Oroville Dam (with Blended Core Material) [33,37, and 121]

Oroville dam is located on Feather River, California, USA. This dam with height 770 ft (235 m) is one of the world's highest embankment dams, and was constructed between 1961 – 1968. Fig. 3.50 shows the maximum section of the dam.



Zone(1&1A) Impervious Core; Zone (2) Transition Zones; Zone(3) Pervious Shell; Zone(4&4A) Impervious Core from selected abutment stripping; Zone (5&5A) Drainage zones.

Fig. 3.50. The maximum section of Oroville dam, California, USA.

1. Foundation Condition

The dam is founded on metavolcanic rock formation. The rock is predominantly amphibolite, with abundant veins of calcite, quartz, epidote, asbestos and pyrite. The rock is categorized as fine grained with average specific gravity of 2.96. Stripping to a sound foundation for the embankment required removal of all soil and weathered rock. The average excavation depth was about 5 m, with a maximum depth of the core trench of nearly 30 m in a major shear. Minor shears could be treated satisfactorily.

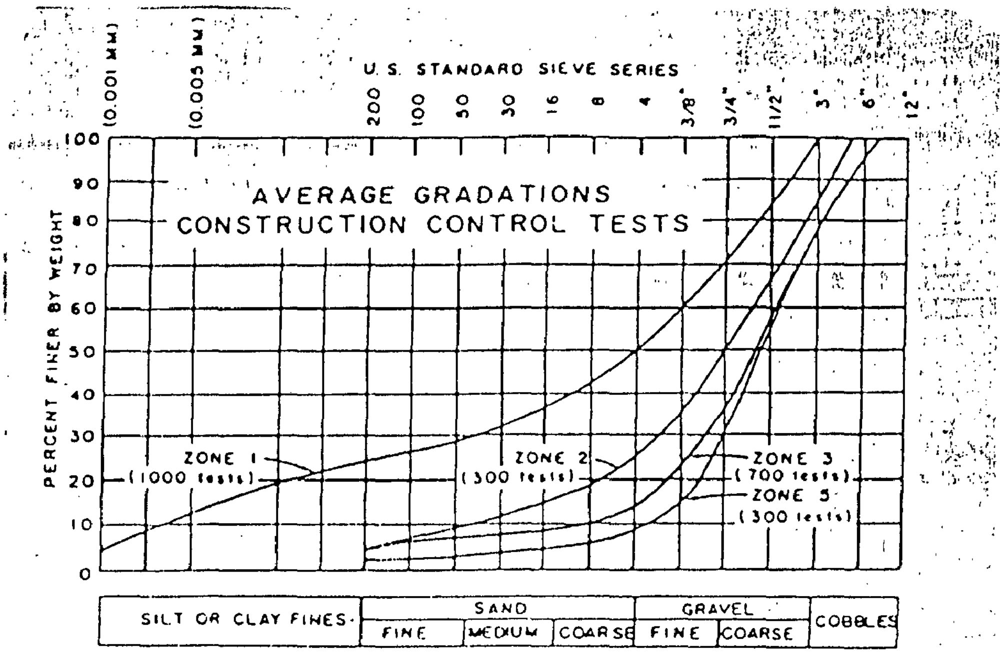
A cement grout curtain, of maximum depth of 60 m is made in the foundation beneath the core. A reinforced concrete gallery is constructed in a trench excavated into the rock foundation for the core to expedite the grouting operation during construction and to permit supplemental grouting after the dam is constructed, if required.

A concrete core block is provided on which the impervious core rests, in order to eliminate differential settlement problems in deeply incised river valley and to provide a uniform surface large enough for placement of the core.

2. Core Design and its Material

The moderately inclined and a thin core section was chosen for the dam. It was estimated to be slightly more expensive than a vertical core section, but it provided more favorable geometry for settlement of the Zone1 material with relation to Zones 2 and 3, which are both considerably less compressive than Zone 1 material under the high loads to be experienced within the dam. The core of the dam has hydraulic gradient (H/B) of about 2.94; and core slope is 0.9H:1V and -0.5:1 on upstream and downstream respectively.

The materials for the core were found in sufficient quantity close to the dam site. This impervious core material is made from the alluvial deposits adjacent to the pervious borrow areas. The core is made of well-graded and well-compacted mixture (blend) of cobble-gravel, and clayey sand and silty clays. The maximum particle size is 75 mm (3"). The core is built in 25 cm lifts compacted by four coverages (eight passes) of a 100 ton pneumatic-tired compactor / roller. The Gradation curve of the core material (Zone1) together with other embankment materials is shown in Fig.3.51. The results of shear testing for the core material and materials for other zones are shown in Table 3.4.



SPECIFICATION GRADING LIMITS

SIZE	PERCENT PASSING ZONE 1	ZONE 2	ZONE 3	ZONE 5
24 INCHES	—	—	100	100
15 INCHES	—	100	—	—
3 INCHES	100	—	—	—
1 1/2 INCHES	—	50 TO 90	—	—
NO. 4 U. S. STANDARD SIEVE	40 TO 70	17 TO 50	0 TO 25	0 TO 12
NO. 200 U. S. STANDARD SIEVE	10 TO 40	0 TO 8*	—	—
5 MICRON	5 TO 20	—	—	—

* 6% MAXIMUM 5 DAY RUNNING AVERAGE

Fig.3.51. Gradation Curve of Core and other Zones Material of Oroville Dam, USA.

3. *Special Seismic Consideration*

The dam is located in a high seismic area. University of California, Berkley, has conducted a series of special studies for the subject, under the direction of Prof.H.Bolton Seed [37], whereas the model of the Oroville dam showed that it can withstand a 0.5g simulated earthquake shock. It was also concluded that this embankment dam will have a high degree of resistance to damage from earthquake loading.

Table.3.4. Shear test properties of core & other zones material of Oroville Dam, USA.

MATERIAL	TEST RESULTS (12-IN DIAMETER SPECIMENS) NO. TESTS	0	C' (TSF)	ADOPTED DESIGN VALUES Ø	C' (TSF)
Zone 1 (clayey gravel)	6	32°–44°6(Avg. 35°)	0.2	34°	0
Zone 2 (silty sandy gravel)	1	44°	0	39.3°	0
Zone 3 (sandy gravel)	5	40°–42°6(Avg. 41°)	0	38°	0
	1*	39°	0		

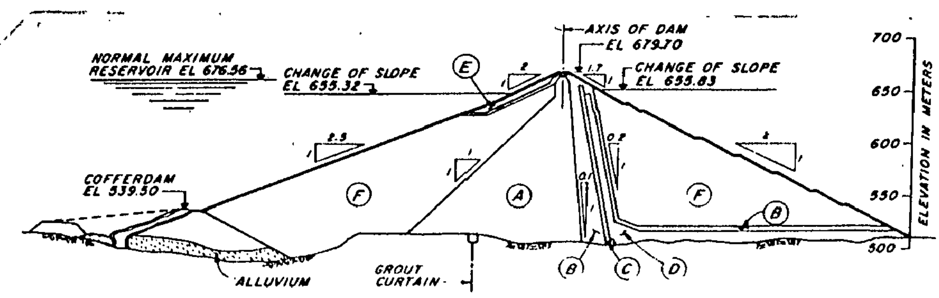
*During construction, one sample was tested in the DWR/UC Berkeley Rockfill Test Facility using 36-inch diameter specimens with 6-inch maximum particle size.

4. Dam Performance

Oroville dam has performed very well, especially during earthquake. An earthquake of magnitude 5.7 on Richter scale which occurred on August 1975 with epicenter about 7.5 miles from the dam produced acceleration at the base of the dam of about 0.1g. The dam behaved satisfactorily. After that the embankment was evaluated for MCE of magnitude 6.5 with 0.6g peak acceleration by the latest techniques, and it was concluded that the dam would perform satisfactorily.

3.3.7. W.A.C. Bennett Dam (with Blended Core Material) [121and 130]

The W.A.C. Bennett dam (formerly Portage Mountain dam) is an earth and rockfill dam 183 metres (600 feet) high, located on Peace River in north central British Colombia, Canada. The dam was completed in 1967 and storage commenced in 1968. The dam section is shown in Fig.3.52.



(A) Core material, (B) Transition, (C) Filter, (D) Drain, (E) Free draining zone, (F) Random shell.

Fig.3.52. Dam section of the W.A.C. Bennett Dam, Canada.

2. *Foundation Condition*

The dam is founded on sedimentary rock in a canyon cut in the rock during post-glacial times by Peace River. The canyon is about 183 m wide at the base.

The steep left rock wall rises about 122 m above the canyon bottom, and the canyon is of wide shape with L/H>5. The sedimentary bedrock consists of a series of interbedded sandstones, shales and thin coal seams. The sandstones in the unwatered bottom exhibited large cracks and frequently found infilled with silt and sand.

In order to avoid any concentration of seepage in the contact area under the core, the foundation rock was blanket grouted to a depth of 18 m. The infill and open cracks portions were filled with grout so that expected settlements are reduced.

Fig.3.53 gives the typical core-contact surface treatment details of W.A.C. Bennett Dam Canada.

3. *Core Design and its Material.*

The moderately inclined and a moderately thick core section was chosen for the dam. The hydraulic gradient is 1.1 and slope is 1H:1V and 0.1H:1V at upstream and downstream side respectively.

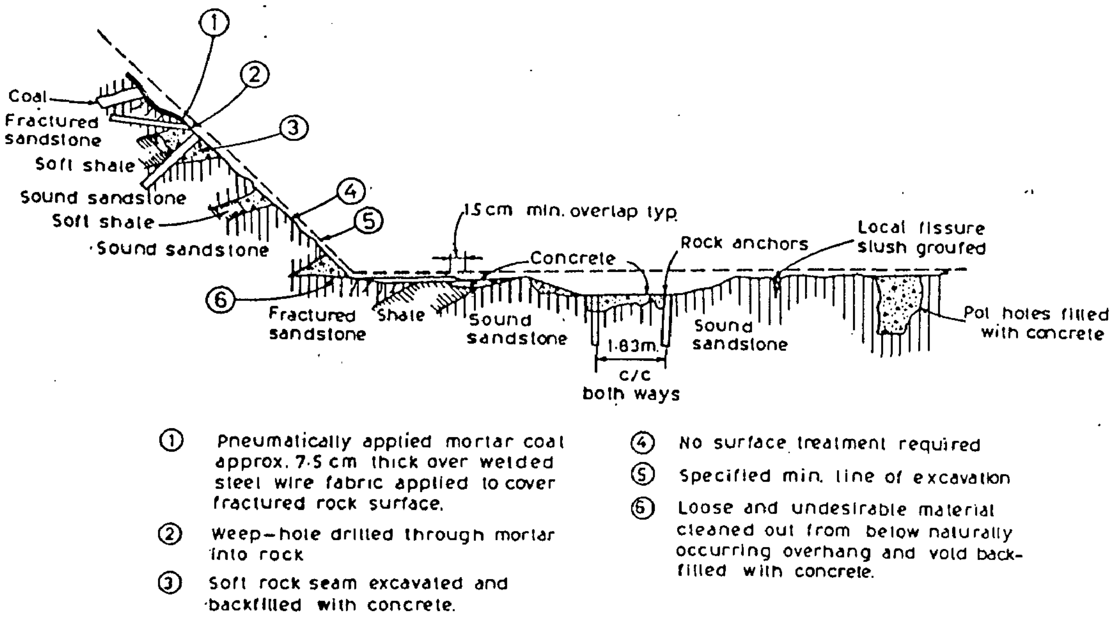


Fig.3.53. The typical core-contact surface treatment details of W.A.C. Bennett Dam Canada.

Table 3.5. Typical "As placed" Densities and Moisture Contents of the W.A.C. Bennett Dam, Canada.

Zone	*Soil Type*	*Field Dry Density (t/m^3)*	*Field Moisture Content (%)*	*Maximum Laboratory Dry Density (t/m^3)*
Core	Silty-sand (SM)	2.01	6.9	2.04
Random Shell	Sandy-Gravel (GW-GP)	2.10	6.1	2.19
Transition	Gravelly-Sand (GW)	2.07	6.1	2.16
Filter	Sandy-Gravel (GP)	2.13	4.1	2.15
Drain	Gravel (GP)	1.8	-	1.79

The core material was well-graded silty sand. The source for material for the dam was moraine deposits consisting of sand and gravel with varying silt content. The cross section was designed making best use of this material. The free draining material, the core, transition, and filter materials were economically produced by large scale processing (mixing various size materials). Approximately 44% of the total quantity of fill placed in the dam was processed.

The properties of the core material and its gradation curves together with other zones are shown in Table 3.5 and Fig.3.54 respectively.

This core material was placed in layers of maximum thickness of 25 cm and compacted with 4 coverages (8 passes) of a 90 ton pneumatic tire roller. Permeability of the core material is about 1 x 10^{-7} cm/sec., and cohesion between 0 – 3.5 kg/cm^2.

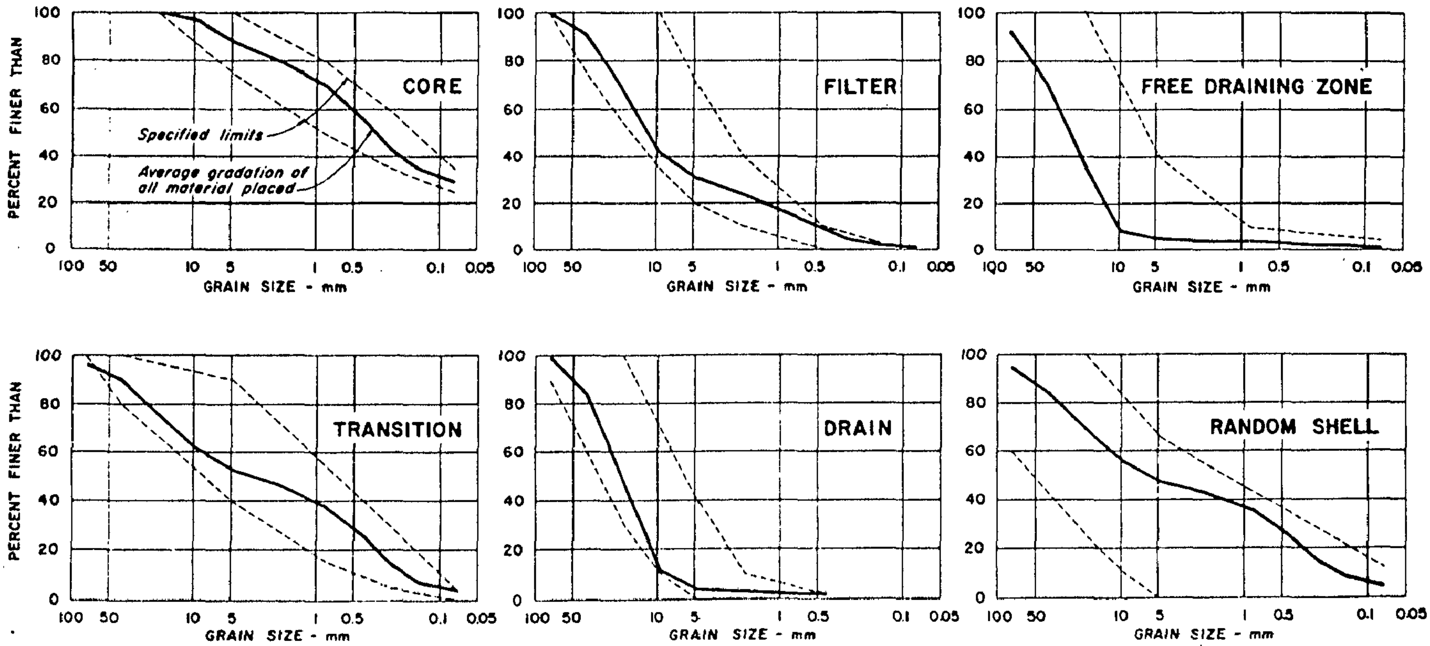


Fig.3.54. Gradation Curves of Core Material & other zones, of the W.A.C. Bennett Dam, Canada

4. *Filter / Transition*

Transition and Filter are provided on downstream side of the dam. The transition zone was placed in maximum layer thickness of 50 cm and compacted with an assembly of 6 ton "Ferguson" vibratory roller. The material in the filter zone was placed in layers of maximum thickness of 50 cm and compacted with 2 pass of the assembly.

5. *Dam Performance*

After completion, the dam has performed better than anticipated and the fill settlement is much less than generally experienced at other dams constructed of similar materials. Besides, no cracks and no evidence of crack development have been observed.

3.3.8. Cougar Dam (with Blended Core Material) [33, 70, 121 and 152]

The Cougar dam is built in USA, on a southern tributary of the Mackenzie River. The dam is 158 m high, and is a rockfill embankment. The typical cross section of the dam is shown in Fig.3.55.

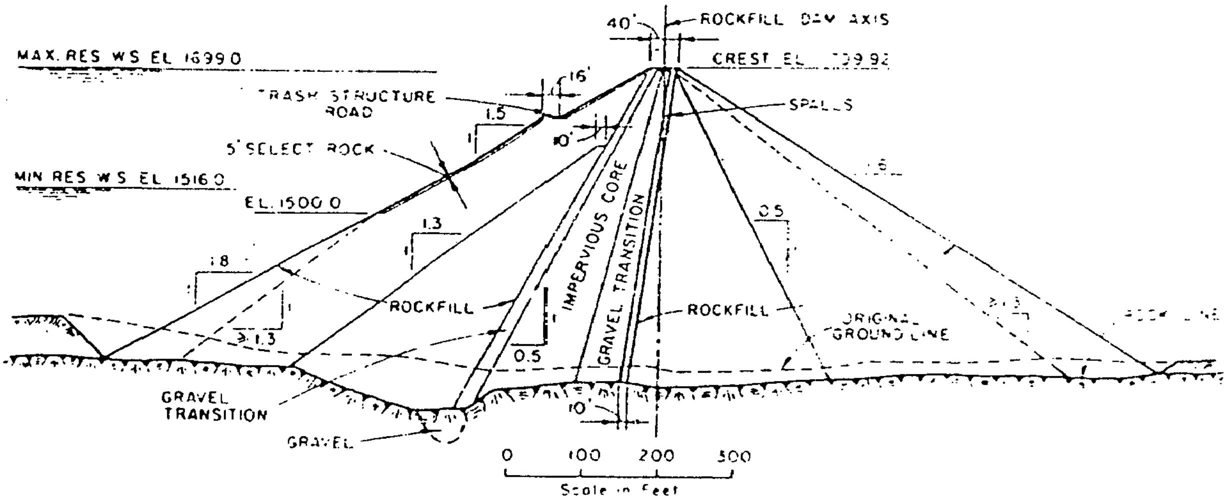


Fig.3.55. The typical cross section of the Cougar Dam, USA.

1. *Foundation Condition*

The dam foundation is made of tuff formations. The basalts are mostly weathered and fissured and covered with an alluvial layer. The latter, in the bank section, represents a mixture of basaltic talus with fine-grain sand and sandy loam, and, in the riverbed section it is pebbles with clay and clays deposited in the ancient river channel.

This old river channel cutting across the embankment in upstream and downstream direction has river bed talus up to a maximum depth of about 26 m below the base level of the excavations with a bottom width in the core-contact area of the order of 6.1 m. This channel was excavated to rock in the core and transition areas and filled with concrete.

Earlier, no seismic activity was recorded in the vicinity of the dam site and the faults did not appear to be active. But later the seismic stability of the embankment was re-evaluated.

2. Core Design and its Material

The moderately inclined core was finally adopted for this dam, after conducting careful studies regarding the central core, inclined core, as well as with respect to material available for the core and other zones of the dam and the time and seasons available for construction. The possibility of future cracks and erosion condition were also considered in the design.

Due to the presence of the projecting basalt ridge on the left abutment, it was considered desirable to place the base of the impervious core upstream of the crest of this ridge. The impervious core was therefore designed to be moderately inclined. The slope of the core is 0.5H:1V and -0.25H:1V at upstream and downstream side respectively. The maximum hydraulic gradient is about 3.7, and it is categorized as a thin core type.

The impervious material was obtained from stripping the right abutment; extending the excavation in the upstream of the foundation area, and from a borrow area 2.5 miles upstream of the dam. The material consisted of a well-graded mixture (blend) of basalt talus and sandy silt with 25% to 55% passing through N° 4 sieve and 10% to 30% passing a N° 200 sieve. The material was excavated, mixed (blended), stockpiled, and re-excavated for placement in the fill. Rock fragments over 15 cm were removed during stockpiling.

The impervious material was placed in 30 cm lifts at water content of about 1 percent above the optimum (standard proctor) and compacted with four passes of a 50 ton rubber tired roller. Other properties of the core material are: permeability 1 x 10^{-7} cm/sec.; average moisture content 15.3% (about 1% above optimum); and dry density 1.9 ton/m^3.

3. Transition

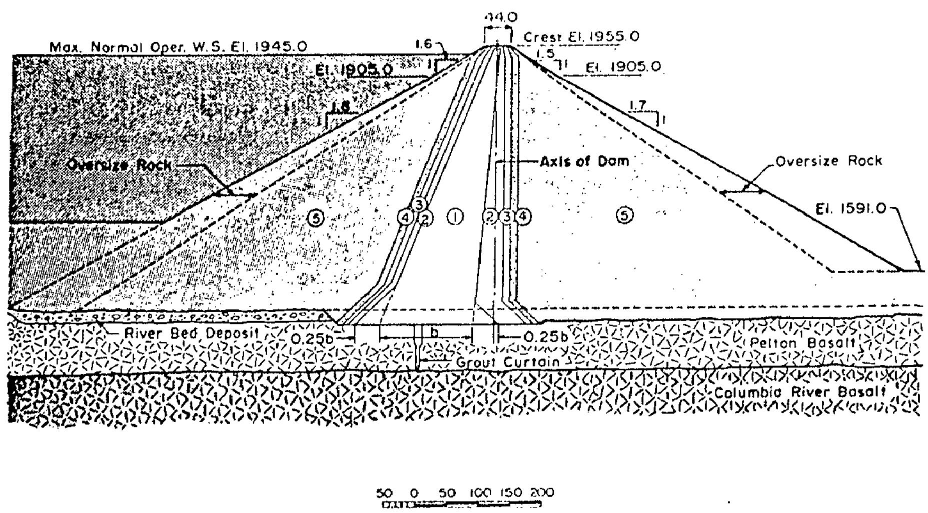
The transition zones were provided on both upstream and downstream sides of the core. The downstream side is thicker than the upstream side. The material for sandy gravel transition were obtained from terrace deposits existing downstream of the dam. This material was spread and compacted to an average dry density of 1.865 t/m^3 with same rollers as used for the core.

4. Dam Performance

The Cougar dam has functioned as designed. No major damages occurred except a transverse crack which occurred along the crest of the dam in 1980.

3.3.9. Round Butte Dam (with Blended Core Material) [121,124 & 152]

The Round Butte Dam with moderately inclined core was completed around 1964. The dam is 440 ft (134 m) high, and located on the Deschutes River in central Oregon, USA. The maximum section of the dam is shown in Fig.3.56.



(1)Impervious core, (2) Filters, (3) Iintermediate filters, (4) Coarse rock material, (5) Rockfill shell from alluvia.

Fig.3.56. The maximum section of Round Butte Dam.

1. Foundation Condition

The canyon is deep at the dam site and has a trapezoidal form with the left bank slope of about 1.7H:1V and the right bank slope of about 1.2H:1V. In the geologic past basalt flows have nearly filled the ancestral canyon up to a depth of approximately 300 m. It consists of lava flows interbedded with horizontally stratified bands of siltstones, sandstone, lightly cemented sand and gravel.

2. Core Design and its Material

A moderately inclined core with curved axis has been provided. The material available for core was silty sand. A slope of 0.38H:1V and -0.087H:1V at upstream and downstream face respectively has been provided. The hydraulic gradient (H/B) is around 2.9, and so it is thin.

The core material was produced by mixing (blending) silty sand with rock components. Originally silty sand was obtained from excavation of foundation. Silty sand was also obtained from processing plant by separation of fines from talus rock. Rock was obtained from quarry sources.

Characteristics of silty sand were superior to the material obtained from quarry sources because of its being well graded.

Then, both materials were brought to stockpiling. The core material obtained from rock talus was moisture conditioned at processing plant prior to stockpiling. The material from quarried rock was moisture conditioned in the borrow area.

The impervious core was placed in 30 cm layers, moisture conditioned as required, and compacted with a 50 ton pneumatic rubber tire roller.

4. Transition

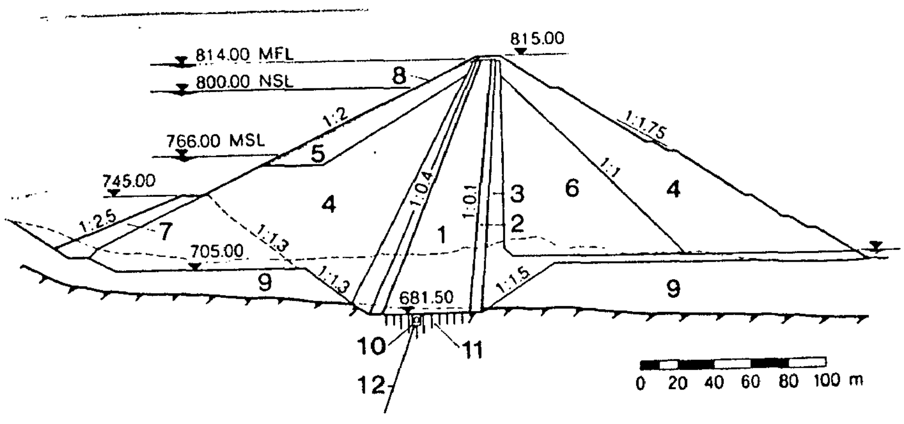
The transition was provided both on upstream and downstream side of the core. The material was taken from rejected material of processing plant and concrete aggregate from basalt rock. It was placed in 30 cm layers and compacted with a 10 ton vibratory roller.

6. Dam Performance

The transverse cracks were observed in the core very near from its contact with the right abutment, approximately one month after the reservoir had been filled in January 1965. The cause of the crack was the tension in the brittle core material resulting from differential settlement between the maximum section of the dam and minimun section of the abutment. The abrupt change in core contact topography caused differential settlement and cracking of the non-plastic material.

3.4.10. Pueblo Viejo Dam (without Blended Core Material) [8, 8a & 93].

The 130 m high Pueblo Viejo dam was designed to withstand the effects of an earthquake of magnitude 8+ whose epicenter lies only 5.5 km from the site. The dam is a moderately inclined core rockfill embankment and located at north side of Guatemala City on the Chixoy River, Central America. The section of the dam is shown in Fig.3.57.



(1) Silty clay.
(2) Fine filter.
(3) Coarse filter and drain.
(4) Good quality rockfill.
(5) Selected, coarse rockfill.
(6) Poor to medium quality rockfill.
(7) Colluvial material.
(8) Riprap.
(9) Alluvium (partly cemented).
(10) Grounting and drainage gallery.
(11) Contact grout holes.
(12) Grout curtain.

Fig.3.57. The section of Pueblo Viejo Dam, Guatemala.

1. Foundation condition

The dam site is located in a region of very high seismicity belonging also to the Pacific "fire circle". The seismicity of the region is influenced by the strong earthquake zone running parallel to the Pacific Ocean, subjected by the volcanic activity in the adjacent mountain range and by the fault resulting from the movement of tectonic plates. The dam site is close to two active faults the Cuilco-Chixoy-Polochic (CCP) and the Motagua Faults forming the limit between the North American Plate and the Caribbean Plate.

The core and filter zones are taken down to bedrock in a trench which will be 60 m wide at its lowest point.

2. Core Design and its Material

A well compacted moderately inclined core embankment was considered to be the safest structure for this difficult seismic environment. The maximum hydraulic gradient (H/B) of this thin core of rock fill dam is 2.8. The core slopes are 0.4H : 1V and -0.1H:1V at upstream and downstream respectively.

The core material is silty clay (CL), to be compacted in 30 cm layers by 8 passes of sheepfoot roller, to achieve an average density of 100% Proctor Standard (PS) and a minimum of 98% PS. Other properties of the core material are: (i) angle of internal friction 29.5°, (ii) compacted dry unit weight 1.61 t/m3, (iii) placement of moisture content about 25.5% or about 2% above optimum (OMC 23.5%).

3. *Filter*

Filters are provided on both sides upstream and downstream of core. These consist of fine filter and coarse filter. Coarse filter at downstream is also connected to horizontal drain. This filter and drain are compacted in 20 to 30 cm layers (drain exceptionally 60 cm) with 4 passes of 10 ton vibratory roller to achieve minimum density equal to 95% of the reference density .

4. Dam Performance

The dam performance has been very satisfactory during the reservoir filling, because no critical events occurred.

3.4. Inclined Core Embankment Dams

3.4.1. Tarbela Dam (with Blended Core Material) [49a, 55a, 109,121 and 135].

The 145 m high Tarbela dam (in Pakistan) is located on River Indus, which rises in the high glaciated Himalayan Mountains in Western Tibet at elevation higher than 5,000 m above sea level. The typical cross section of the dam is shown in Fig.3.58.

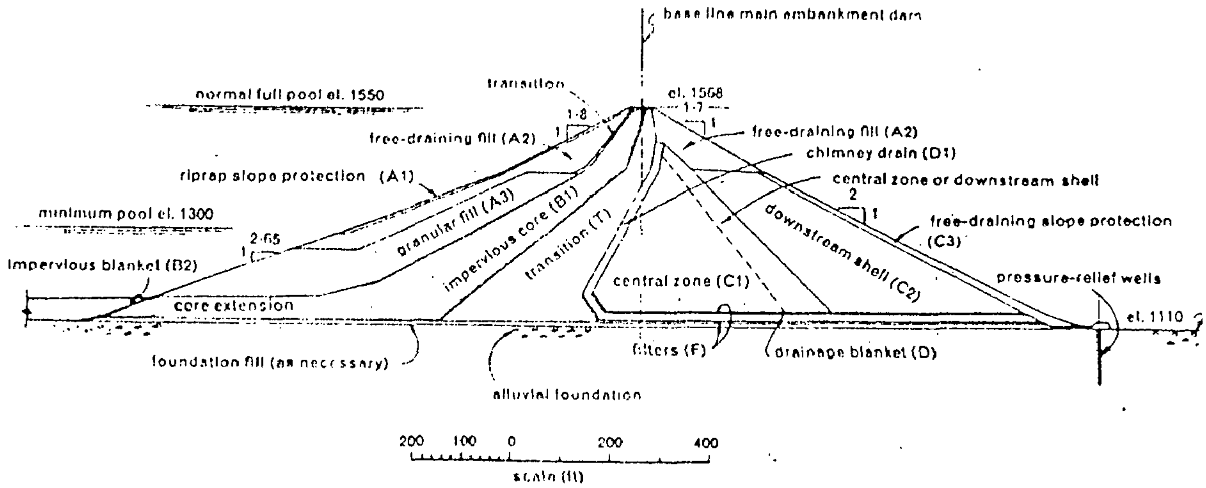


Fig.3.58. The typical cross section of Tarbela Dam, Pakistan

1. Foundation Condition

The dam is located in high seismicity area with a minimum seismic acceleration of 0.1g.

The dam was built on a very difficult foundation. The abutments comprise of quartzite, limestone, phyllite, schists and basic igneous rocks. Several faults lie around the region and Darabund fault lies under main dam.

The valley was filled with deep alluvium comprising of boulders, cobbles, gravels and sand. The depth of alluvium exceeds 215 m under the main dam. The pervious nature of the alluvium necessitated an upstream blanket which extends 2,130 m upstream. In addition to an upstream blanket, an extensive drainage well and collector system is provided at the downstream toe to control seepage under the dam.

The Indus river valley in the project area is wide and flat with river flowing as braided stream. A profile across the river valley at the dam axis is shown in Fig.3.59. On the left side hard white quartzite rock is present. Along the Dal Darra valley, a near vertical slide fault system is present. The limestone and phyllite are generally thin-bedded. Many faults and folds occur in the limestone and phyllite and generally the rock is of poorly quality.

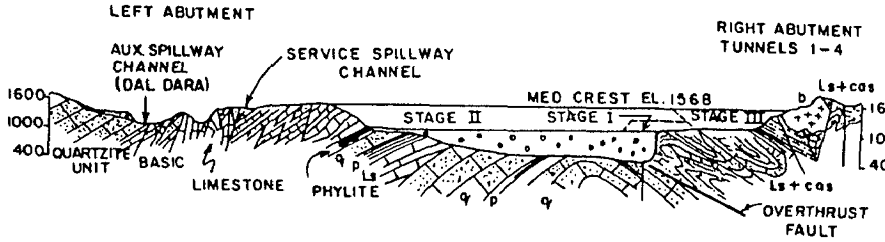


Fig.3.59. Profile across the river valley at the dam axis, Tarbela Dam

2. *Core Design and its Material*

An inclined core has been adopted to control seepage, including an impervious blanket in upstream. The thickness of inclined core gives an average seepage gradient of about 1.9. The slope of inclined core is 1.8H:1V and -1.22H:1V for upstream and downstream face respectively.

The impervious core material was made of gravel-sand-mixture (blend) with 15 cm maximum size and 20 – 40% silt content. The material was made to satisfy the requirements in respect of strength, permeability, flexibility and self healing. In this dam the blending of material was done as part of a highly complex system including excavation, crushing, screening, conveying, stockpiling and mixing. The permeability of the core material was found to be about 1 x 10^{-6} cm/sec.

*3.4.2. **Miboro Dam (with Blended Core Material)*** [70, 81, 109 , 121 and 152].

This 130m high sloping core rockfill dam is located on the Sho River in Honshu Island, Japan. The construction of the dam was started in 1958 and completed by 1960. The typical section of the dam as shown in Fig.3.60.

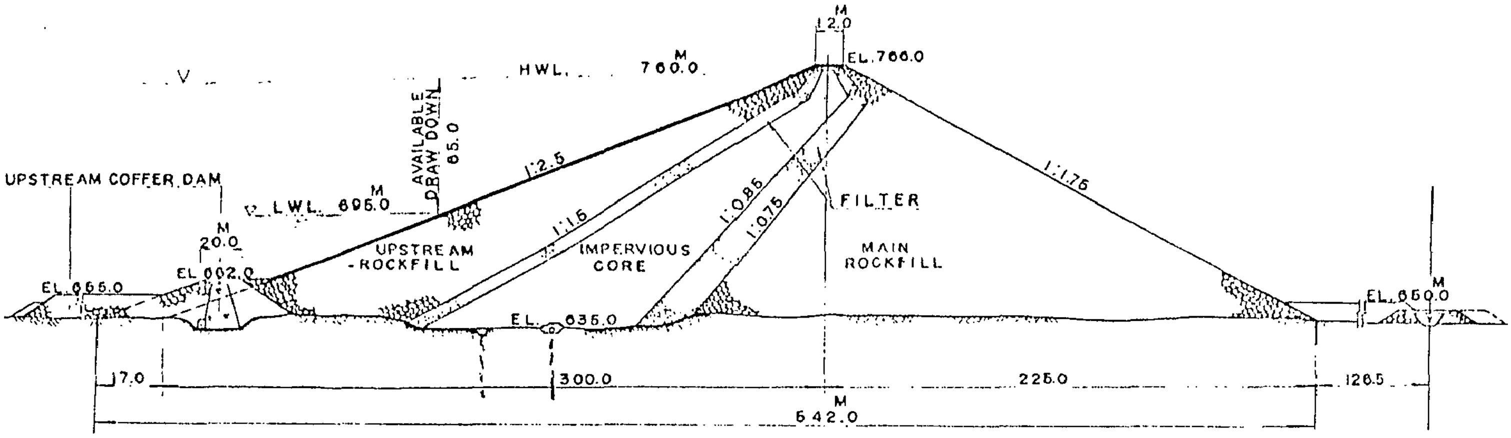


Fig.3.60. Typical section of Miboro Dam, Japan.

1. Foundation Condition

The dam is located in a high seismic region. The bed rock at the dam site consists of quartz porphyry and granite porphyry. The bed rock of the right abutment is rather poor as talus deposits are found along the lower slope. The porphyritic rocks mentioned above was highly cracked and jointed. In the left abutment, quartz porphyry of relatively good quality was found exposed at several spots, but thick talus deposits covered the upstream and downstream sides of this abutment. Thickness of deposits in the river bed was relatively thin and was less than 8 m.

A conspicuous fault was found at the toe of the right abutment of the dam, parallel to the river. The width of the fault portion coming in contact with impervious core zone was about 10 m, out of which 2 m was well compacted clay and the rest consisted of crushed rock.

2. *Core Design and its Material*

Due to unfavorable geological conditions especially due to fault formations on the right abutment, a sloping core rockfill embankment dam was chosen as compared to earlier proposed concrete gravity dam. The impervious core material could be found in areas not very far from the dam site.

The slopes of inclined core are 1.5H:1V and -0.85H:1V for upstream and downstream face respectively. Its maximum hydraulic gradient is about 1.33, so it is categorized as moderately thick core.

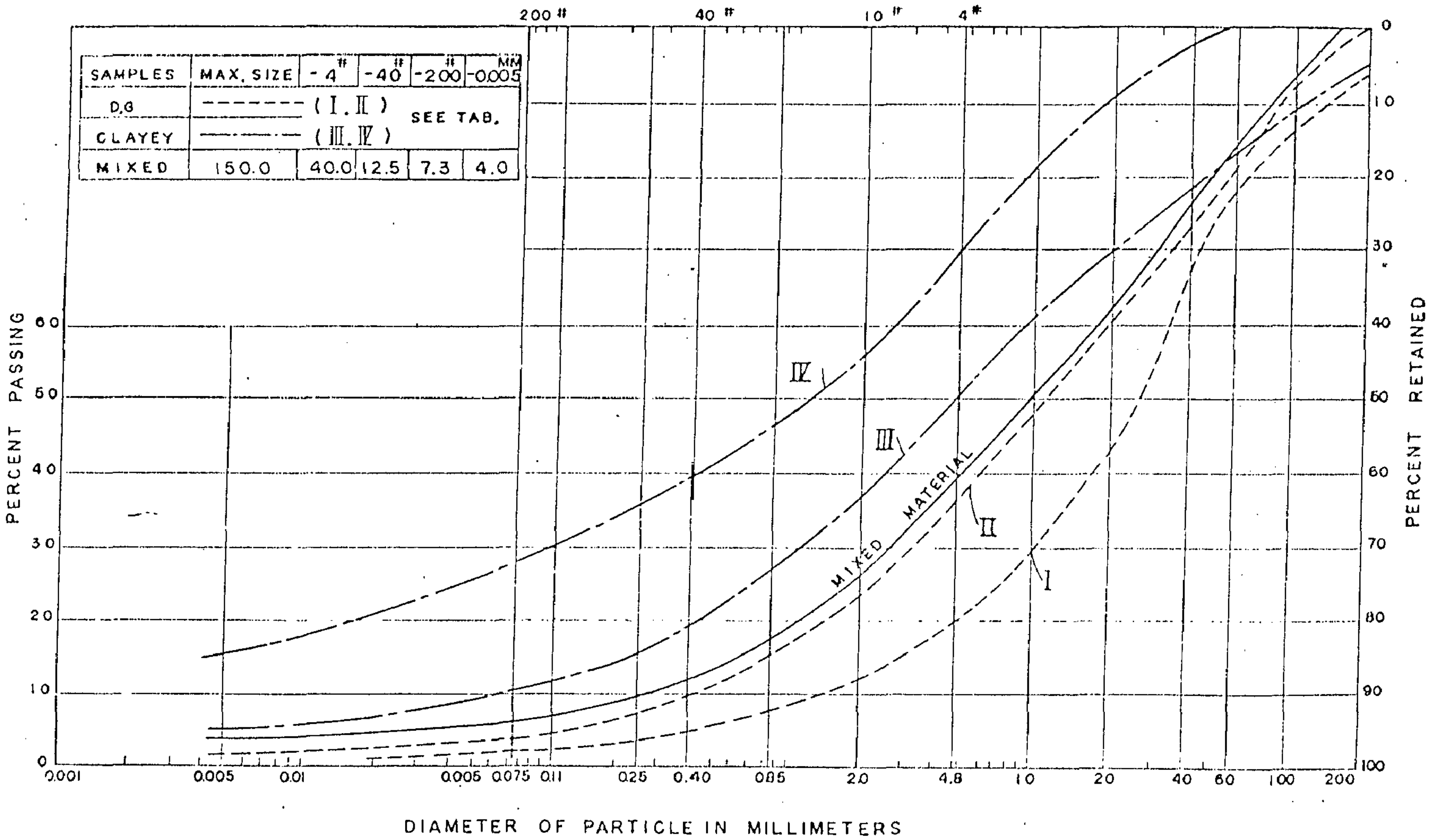


Fig.3.61. Gradation curves of the core material of Miboro Dam

The core material was obtained by mixing (blending) two types of material i.e. clay and disintegrated granite (D.G) material from the Akimachi area which is located about 3 km upstream of the dam. Blending was essential because clay material contained high moisture content (between 30 to 60%.) than its optimum, and had low density. Blending was done successfully by making stockpiles on the flatter slopes. The moisture content of clay could be lowered by mixing disintegrated granite (D.G.). During the stockpiling operation oversized rocks and boulders could be removed. The horizontally laid layers were excavated vertically by shovels. The gradation curves of the core material is shown in Fig.3.61.

The properties of the blended core material are: (i) permeability less than 1x 10^{-5} cm/sec., (ii) angle of internal friction 35 – 40°, (iii) cohesion 2.8 kg/cm^2, (iv) compacted dry unit weight 1.87 t/m^3, (v) moisture content 10 – 14%. Core was laid in layers of 20 cm and compacted by 6 coverages with sheepfoot roller.

3. *Dam Performance*

The Miboro Dam has proved satisfactory during an earthquake of magnitude 7 producing $a_{max} \simeq 0.2g$.

After two years of operation, small amount of settlement occured. For the first year of construction period, the pore pressures within the core were found less than expected and amounted to less than 10% of the superimposed load, and less than 5% when the dam was completed. This may be attributed to the placement at moisture content on the dry side of optimum.

3.5. Study and the inferences

The important characteristics of these high embankment dams (25 nos) in respect to core design are summarized in Table 3.6.

This review of high embankment dams reveals :

1. High dams up to a height of 335 m in narrow valleys in high seismic areas have been successfully built with special care in design and construction of the impervious core.
2. The thickness of core and its position is decided depending on the availability of material, its properties (gradation, c, φ, permeability, moisture content, compressibility, etc.) and the functional requirement such as

seepage barrier, resistance to cracking, hydraulic fracturing and concentrated leaks; settlement and erosion.

3. In high dams above 200 m height a thickness of 0.3 to 0.5 times the water head is found satisfactory.
4. All types of cores vertical, inclined and moderately inclined have been adopted in high dams. It has been found that for 13 nos of vertical core dams, the slopes are ranging from 0.089H: 1V to 1.03H:1V on both upstream and downstream face. For 10 nos moderately inclined core dams, the slope is ranging from 0.38H: 1V to 1H:1V (u/s face) and –0.5H:1V to 0.1H:1V (d/s face). For 2 nos inclined core dams, the slope is 1.5H: 1V and 1.8H:1V (u/s face), and –1.22H:1V and -0.85H:1V (d/s face). This slope of –1.22H:1V (d/s face) which is more than usually adopted limit (1H:1V, see Chapter – 2 point 2.2) is adopted in Tarbela Dam.
5. In many of these high dams blending is done to increase erosion resistance and reduce cracking. Clayey material is mixed with sand & gravel. The core material, therefore, has small cohesion and greater angle of internal friction. This, however, increases permeability but the proportion is made such that required permeability less than 1 x 10^{-5} cm/sec is ensured.
6. In most cases compaction is preferred to be done at a moisture content slightly above the optimum moisture content in order to reduce the possibility of cracking but in some cases this has resulted in high pore pressures.
7. In some cases inclined core is preferred to reduce differential settlement and cracking and also to expedite the construction.
8. In all case core is protected by suitably designed fillers from both upstream and downstream side.
9. The design of core in all cases is site specific and guided by precedence, requirement and characteristics of available material. The proposed section in some cases is analyzed by FEM for stresses and deformations under seismic conditions.

10. In some cases, a concrete block is provided on which the core rests in order to eliminate differential settlement, to provide a uniform surface for placement of core, as base for grouting operations, and to prevent leakage through foundation.
11. In all cases the foundation under the core is consolidated by grouting, so that fines from core may not move in foundation rock under seepage head. In some cases it is done through a foundation gallery while dam construction is in progress.

Tabel 3.6. List of High Embankment Dams with its Core Properties & its Important Characteristics.

1	2	3	4	5			6			7									
No.	Name of Dam	Country	Height of Dam (m)	Type of Embankment Dam			Core Geometry			Core Material									
				Earth Dams (E)	Rockfill Dams (RF)	Mixed Type (E & R)	Vertical	Moderately Inclined	Inclined	Impervious Layer	Blended or Indicated Blended Material	Description of Gradation	Permeability (cm / sec.)	φ (degree)	Cohesion c (kg/cm2)	Compacted Dry Unit Weight (t / m3)	Moisture Content (%)	Compaction: Layer Thickness (cm)	Compaction: Method
1	Rogun [15,77,104,121] [36a,p.259]	USSR	335			✓ [77]		✓ [121] p.52			Processed Natural-Loam & Pebbles [121]	max. 200 mm diameter	2 x 10^4		0.2				22 t - VB
2	Nurek [109,121, 147,152] [36a,p.107, 261]	USSR	300		✓ [147]		✓				Natural Mixed of Rocky Clay [147,p.562]	Min. of 50% Fines Less than 5 mm,	2 x 10^4 [152]			2.0 - 2.3 [147, p.556]	13 - 15	30	50-R.T.R.
3	Boruca Dam [49a]	Costa Rica	302		✓ [49a]			✓			Alluvial Deposits of Silt+Sand+Gravel					Maximum 2.0 to 2.2			
4	Chicoasen [2, 78, 79, 109,121]	Mexico	261			✓ [121, p.439]	✓ [121, p.240]				Clay - Gravel Mixtures [150, p.15]			24-31	0	1.9	- 0.8 to 3% of OMC	25	7t - VR 6 passes
5	Guavio [72 & 109]	Colombia	245		✓ [72]			✓	[109]		Shales fragments within a Silty - Clayey Matrix [72]	Max Size 150 mm				1.95	14 - 19	20	8 Cov. 10t - VR
6	Mica [50 & 149] & [36a,p.257]	Canada	244			✓ [121, p.439]		✓		Glacial-till	Glacial-Till of Well-graded with little or no processing	Max. 8"	1 x 10^7	34	0	2.1	9 ± 2%	25	2 Cov. (4 passes) RTR
7	Chivor Dam (Esmeralda) [3, 49a, 104 & 121]	Colombia	237			✓ [3]		✓			Gravels in Clay - Silty Matrix of medial plasticity	Rocks fragments up to 15 cm in size				1.82	17 - 21 (≃7 % above OMC)		
8	Oroville [33, 37,106, & 146]	USA	235	✓ [33]				✓ [121]			Mixture of Cobble, Gravel + Clayey Sand and silts [121, p.142]	Max 3" or 76 mm [33]	7x 10^1	34	0.2	2.24	8	30	4 Cov; 100t - PR [37]
9	W.A.C. Bennett (Portage Mountain) [118a, 130, 150]	Canada	183			✓		✓ [121, p.142]			Silty sand of well graded and well compacted (blend) [118a]	Well graded non plastic material (SM)				2.01 (128 lb/cu.ft)	6.9	25	4 Cov - 90t - PTR
10	Dartmouth [82]	Australia	180			✓ [82]	✓				Residual granite soil (Silty sand to clayey-sand) and pushed to large stockpiles to provide uniform material	Plasticity from low to medium ranging from 4 to 18 and averaging 12.					12 - 18% (OMC: 13.5 to 15.9 & adopted -1.5 to 2% of OMC)		

1	2	8			9			10	11	12	13	14	15	16	17	18
No.	Name of Dam	Core Slope (Horizontal : Vertical)		Core Thickness at the bottom (m)	Approximate Core Categorised [121]			Approximate Maximum Hydraulic gradient	Foundation condition / Geological Structure of Ground	Type of Canyon	Climate indication	Remarks on Seismicity Condition (E=Earthquake)	Filter /Transition Adjacent to the core Characteristic (F=Filter; T=Transition; D=drain)	Axis of the dam	Important Performance through Observation	Remarks or special interest
		U/S	D/S		Thin (Tn)	Thick (Tk)	Composite (Cp) or Moderately Thick (Mt)									
1	Rogun	0.9 :1 [121, p.42]	-0.4 :1	≃167.5			Mt [109]	≃2 :1	Rock Salt layer, Soft rocks(Sandstone & Siltstone) and two Fault Zones.	Narrow, S-Shape slopes of 50° - 55° L/H = ≃1.5	Temp.40° to - 31°, Ann.Prep. 816 mm, Av.Depth Snow 61 cm, Max. River Q=3730 m3/s	High Seismicity & Tectonic Activity, Intensity Grade IX Design = 0.35g	Cobbles & stone overburden [36a,p.259]	Curvature		
2	Nurek	0.25 :1	0.25 :1	150			✓ [36a]	2 :1 [49a & 36a]	Hard Rock with Alternating Aleurolites & Sandstones	Narrow Mountain Canyon. L/H = ≃3	Assumed Ditto No.1* beacause [77] or d/s of No.1	Seismic Shocks might be of magnitude as high as 8-9	Size 0.1 - 10 mm, size size < 5mm is not less than 60% [121]. u/s 10 m & d/s 6 m thick	Straight		Provision of Alternative Loess Loam & Rocky Clay as Core Mat.
3	Boruca Dam	0.65 :1	-0.25 :1	≃127	✓			2 :1	Rock Formation, Interbedded shales, Silstones and Limestone.	Narrow Gorge (L/H = 2.99)	Coastal Range Area	Seismic wave Velocities vary: 1800-2600 m/sec	Made by Processing Alluvium, as loosely as possible			
4	Chicoasen	0.35 :1 [121]	0.2 :1 Vertical in nearly 60 m lower part	98 [109]	✓ [109, p.286]			2.38 :1	Large masses of Limestone packed with sand & gravel. [121, p.83]	Narrow gorge-carved, nearly vertical [78, 121]. L/H =1.1 [109,p.287]		Highly Seismic	Average thickness 7.5 m, from well-graded sand, max.size 76mm [70 & 121]	Straight		Counter Measure of effect Core-Abutment Interaction
5	Guavio	≃0.85 :1 [109]	-0.5 :1 [109]	≃83	✓			2.94 :1 [49a]	Rocks Paleozoic, metasozoic, and no faults.	A narrow V-shaped valley, L/H =1.6[109] & deep 600 m high			u/s & d/s with T & F from crushed rock fragment.			
6	Mica	0.4 :1	-0.1 :1	≃81	✓			2.3 :1 [49a]	Bedrock: Mica-schist, granitic-gneiss, pegmatite, and marble	Steep walled valley L/H = 4	Most mean falls snow max. ann. Flow = 3170 m3/s	Core slightly-inclined to counter E	No F & T	Curved slightly u/s		The core widening at abutment (flaring)
7	Chivor Dam (Esmeralda)	0.7 :1 [49a, p.20]	-0.4 :1	≃79	✓			≃3 :1	Rock Layers & alluvial material	L/H = 1.3 Ravine-like valleys	High precipitation. Av. Ann. Rainfall = 3500 mm	Highly seismic	Well graded material both u/s & d/s up to 15 cm in size, minimum width 4.0 m. The upper 40 m of dam a two layer are provided u/s & d/s.	Straight	Pore pressure 50% of the load imposed by dam-fill, but dissipate 30% prior to first filling	High water content problem to pore water pressure to dissipate.
8	Oroville	0.90 :1 [49a, p.20]	- 0.5 :1	≃80	✓			2.94 :1	Meta-volcanic rock (av. Gs 2.96) fine grained texture, weathered rock & shear zone.	Relatively narrow L/H = 8.99	Flood flows reach 6,500 m3/s. Av. Ann. Precipitation about 1770 mm	Safety Horiz. E = 0.1g. High degree of resistance to damge from E until 0.5g E shock [37]	Well graded mixture of sands, gravels, cobbles and boulders to max. 40 cm isze at #200 sieve	curvature	Pore pressure problem in the core due to earquake 5.7 magnitude in 1975	Effect of earthquake and pore pressure
9	W.A.C. Bennett (Portage Mountain)	≃ 1.0 :1	≃ 0.1 :1	155 [118a]		✓		1.1 :1 :1 [118a]	Lower cretaceous sediment bedrock	L/H > 5 (widely valley)			D/s Filter, wide Transition & internal drain d/s [150]	Straight	Well graded material is reduced develop of different movement of high earthfill dam.	Well grade material is very important for high earthfill dam.
10	Dartmouth	0.4 :1 at upper, & 0.5 :1 at lower	0.24 :1	129		✓		1.4 :1 [121]	Granitic gnessis, irregularly weathered with massive blocks	Deeply incised in a V-shaped gorge with valley slopes averaging 1V:2H but as steep as 1:1 in places L/H = 4		Within Seismic Area	Well graded with dry density 2.2 t/m3. U/s & d/s with fine filter, coarse F only at d/s. Fine-F=4 m., Coarse=3m	Straight	High pore pressure than predicted	wide central core from residual soil

Tabel 3.6. List of High Embankment Dams with its Core Properties & its Important Characteristics *(Continued)*

1 No.	2 Name of Dam	3 Country	4 Height of Dam (m)	5 Type of Embankment Dam Earth Dam	5 Rockfill Dam	5 Mixwd Type	6 Core Geometry Vertical	6 Moderately	6 Inclined	7 Core Material Impervious Layer	7 Blended or Indicated Blended Material	7 Description of Gradation	7 Permeability (cm / sec.)	7 Φ (degree)	7 Cohesion c (kg/cm2)	7 Compacted Dry Unit Weight (t / m3)	7 Moisture Content (%)	7 Compaction Layer Thickness (cm)	7 Compaction Method	8 Core Slope (Horizontal : Vertical) U/S	8 D/S	Core Thickness at the bottom (m)	9 Approximate Core Categorised [121] Thin (Tn)	9 Thick (Tk)	9 Composite (Cp) or Moderately Thick (Mt)	10 Approximate Maximum Hydraulic gradient	11 Foundation condition / Geological Structure of Ground	12 Type of Canyon	13 Climate indication	14 Remarks on Seismicity Condition. (E=Earthquake)	15 Filter /Transition Adjacent to the core Characteristic (F=filter; T=Transition D=drain)	16 Axis of the dam	17 Important Performance through Observation	18 Remarks or special interest
11	Trinity [148]	USA	164	✓ [74a]			✓				Weathered greenstone (copley); various part of the pit to blend with the highest practical rock percentage.	SM - SC High strength of core type material	1×10^{-7}	28	0.15	1.74	15.5	30	12 Cov - Comp.R	1 :1	0.75 :1	298		✓		0.55 :1	Metamorphic crystalline bed-rocks complex of deeply weathered & altered rock.	L/H = 4.6	Ann. Precip. Increse with altitude from 40 in. near dam site to 80 in. in high country, 75% on Feb. - June.	The dam in Zone 3 (1973 Seismic Zone Map)	T at u/s & d/s from Bragdon shale (ML)	Straight		Searching high strength core - type material
12	Palo Que Mado [8]	South Africa	160			✓	✓			Volcanic brecia or "Tahar" (ML)		< 100 mm D_{50} = 2 mm		36		1.98	12% (Opt.)	20	8 Cov - SFR	0.3 :1	0.3 :1	≃105			Mt	1.52 :1	"Fire system" - fault	L/H = 2.3		High seismic condition	Inclined T at u/s and F at d/s	Slightly Arched		Material chosen in purpose more earthquake resistant
13	Cougar [33, 70, 96, 121]	USA	158		✓ [33]			✓ [70]			Well graded mixture of basalt talus + Sandy Silt		2×10^{-7}			1.9	15.3 (OMC + 1%) [121]	30	2 Cov 50t - RTR	0.5 :1	0.25 :1	≃43	✓			3.7 :1	Tuff, sandstone[70] and basalts, covered by aluvl. deposit	Parabolic shape, ≃35° to Horizontal. L/H = 3.1	Ann. Av. Precipitation ≃2310 mm (7 months / yr)	Seismic Stability of the embankment was evaluated [152]	Sandy gravel F at u/s & d/s, but d/s more thicker	Slightly curved [70]		
14	Swift [101, 33]	USA	156	✓			✓				Coarse grained material was spread & mixed into wet material from mudflow & slopewash materials, so the combined lift could compacted satisfactorily.	Not less than minus 200 to provide adequate control permeability & lack of plasticity.	2.5×10^{-5}	45	0	1.83	Wide range Approx. 10% (OMC 12%)	40	4 Cov - 50t-Heavy RTR	0.1 :1	0.45 :1	≃74		✓		2.11 :1	Volcanic & Glacial features, bedrock mudflow & stream-bed deposit.	L/H=5	Max. flow 1540 m3/s., Av.Ann. Precip. About 305 cm (heavy-rainfall) & under adverse weather condition (18 months)	Max. Earthquake of magnitude 5.5, happened	Vertical granular chimney drain u/s & d/s protected by transition		No pore pressure are being developed in the unusual embankment even when placed under extremely wet cond., (due to rapidly dissip.)	Volcanic mudflow was suitable for placement in wet weather or adverse.
15	Geoscheneralp [1, 29, 70, 81, 103a, 70, 81, 103a, 109, 152]	Switzerland	155		✓ [1]		✓				Mixture of Sandy-Gravel, Opalinus Clay,	Max. dia. 100 mm	5×10^{-8}	30 - 35	0	2.15	6 - 8%	30 [1]	30 - 45 t, PTR	1.03 :1	1.03 :1	47 [1]				3.08 :1	Good granite for core foundation, gneiss with boulder clay [70]	L/H = 2.3	Severe [1]	Special Seismic Survey in Fondation Area	2 layers F u/s & 3 layers d/s., 1st. 2 lyrs. 0-100mm alluvium+gravelly debris, 3th. Lyrs. 100-200mm of crushed rock.			
16	Gepatsch [1, 104, 109, 121, and 152]	Austria	153		✓ [104]		✓				Nature moraine clay consliluted the mixed sandy gravel talus filler, at adjoining section u/s side+1% bentonite (zone 1a)		3×10^{-7}	29	0.1	2.15	8 (+2 % Opt.) [121, p496]	30	6 Cov. 40 t - RTR	0.25 :1	0.25 :1	40.8	✓			3.7 :1	Gneiss overlain by Alluvia, clay boulder & talus material.	L/H = 4	Severe climatic-condition.		Filter of natural alluvia & graded earth fractions > 80 mm, with processed material, 40 - 60 cm in layer & rolled by vehicle	Slightly concave toward u/s		
17	Infiernillo [1, 36a, 109, 121, 127, 152]	Mexico	148		✓ [71]		✓			Sandy-Clayey Silt Medium plasticity(PI 10-20 [121,p233]		High Plastic Clay (Varies : CH -CL) [109,p.294 & 127]	5×10^{-8} [121]	12 - 14 [121]	0.35 - 0.5 [121]	1.586 ±94%of Std. Proctor Optimum.[127 & 121, p.141]	23 (Wet side of Optimum (w + 4%)) [121,p.233&471]	15 [124]	15t - SFR [121]	0.089 :1	0.089 :1 [121, p.140]	0.2H [109, p.70] = 30 m	✓ [121, p.141]			≃4.93 :1	Rock(sedimentary, and alluvium) [121,p233]	Steep (40° - 45°) L/H = ≃2.3		Frequent and intensive earthquakes, Seismic Coeff. Assumed = 0.15. [1 & 152] & High Sismic zone [124]	Backed by compacted Rockfill Grad.0.1-10mm & 5 - 150 mm [1] &[121,p.376]		Narrow Transverse cracks, up 1.3cm < 3.0 m at right abutment [121, p.377]	
18	Tarbela [49a, 55a & 135]	Pakistan	145			✓ [49a]			✓		Gravel-Sand-Silt mixture	Max. 150 mm silt content 20 - 40 %	1×10^{-6}							1.8 :1	-1.22 :1	≃76	✓			≃1.9 :1	Hard white quartzite rock, a near vertical slide slip fault.	Very difficult deep alluvial foundation & broad alluvial valley L/H = 10	Rises in the high glaciated Himalayas Mountain	Very High seismicity & active seismic area	u/s Granular fil, d/s T Horiz. drainage zone		Sinkhole formation on first impounding	

Tabel 3.6. List of High Embankment Dams with its Core Properties & its Important Characteristics (Continued)

1	2	3	4	5			6			7										8			9			10	11	12	13	14	15	16	17	18
			Height of Dam (m)	Type of Embankment Dam			Core Geometry			Core Material								Compaction		Core Slope (Horizontal : Vertical)		Core Thickness at the bottom (m)	Approximate Core Categorised [121]			Approximate Maximum Hydraulic gradient	Foundation condition / Geological Structure of Ground	Type of Canyon	Climate Indication	Remarks on Seismicity Condition (E=Earthquake)	Filter /Transition Adjacent to the core Characteristic (F=filter, T=Transition, D=drain)	Axis of the dam	Important Performance through Observation	Remark or special interest
No.	Name of Dam	Country		Earth Dam	Rockfill Dam	Mixed Type	Vertical	Moderately	Inclined	Impervious Layer	Blended or Indicated Blended Material	Description of Gradation	Permeability (cm / sec.)	φ (degree)	Cohesion c (kg/cm2)	Compacted Dry Unit Weight (t / m3)	Moisture Content (%)	Layer Thickness (cm)	Method	U/S	D/S		Thin (Tn)	Thick (Tk)	Composite (Cp) or Moderately Thick (Mt)									
19	Neltzahualcoyotl [35, 127]	Mexico	137.5		✓ [35]		✓			Sandy silt [35]		ML to MH [127]		22	0.4 - 2.0 [152]	1.75 [121,p.142]	About 1% above OMC (OMC varies 6-28 % [35])	20	41 - VR 6 pass [127]	0.2 : 1	0.2 : 1	about 60 m [152]	✓			≈2 : 1	Folded marine sediment, fault & bed rock [152]	Narrow Gorge [127] L/H=≈3	high rainfall	Within seismic zone.	Filter of 4 m thick d/s , Transition of 10 m thick u/s [127]		deformation not occur predominantly [152]	
20	Round Butte [121, 124, 152]	USA	134		✓ [87]			✓			Mixed Silty sand and Rock components							30	50t - PTR	0.39 : 1	0.087 : 1	≈46	✓			2.9 : 1	The deschutes formation approx. 1000 ft thick with lava flows interbedded	Deep canyon area at site with Trapezoidal form, slope 1.7 & 1.2:1 left & right side bank. L/H=≈3.3			T at u/s & d/s of rejected material processing of concrete aggregate of basalt rock.	Arch-shaped	Transverse cracks was observed in core contact at right abutment.	
21	Beas [87]	India	132.6	✓ [87]			✓				Clay-Stone* & Sand-Rock** *: less than 50% in 1b **: less than 50% in 1a		1×10^{-6}	26.5	0.4	1.94	10.8	20	8 Cov - SFR	1 : 1	0.6 : 1	≈190		✓		≈0.7 : 1	Upper siwalik formations, sandrock & clayshale, siltstone	L/H=≈14.5	Ann. Precipitation in short period of about 3.5 months per year.	high seismic zone region.	Selected sand & gravel F at u/s & d/s		Pore pressure not very high	
22	Ambuklao [1, 34, & 121]	Philipines	131		✓ [1]		✓			Quarried Loam				15	2.5			45 [1]	4 Cov 50t - RTR	0.25 : 1	0.25 : 1	65 [1]	✓			≈2 : 1	Metamorphic rocks & diorite & meta sediment	Narrow Gorge widen u/s. Narrowing canyon within steep scarps of andesites tuffs & ancient lavas. L/H = 2.5	Max. Flood peak daily flow =5950 m3/s, high run off, tropical climate	Safety Factor due to Earthquake has been considered	2 layers u/s F, 3 Layers d/s F at (d15/d85) < 5		Little seepage through left abutment	Core geome is decided a discussed P Inc.&Vert. C
23	Pueblo Viejo [8, 93]	Guatemala	130		✓ [8a]				✓	Silty Clay (CL)		< 100 mm D_{50} = 0.04mm		29.5		1.61	25.50% (± 2% of Opt.)	25	4 Cov SFR	0.4 : 1	-0.1 : 1	≈48	✓			≈2.8 : 1	Fault system cuilco-chixoy-polochic	very steep for abutment L/H = ≈1.9	Mean River flow = 50 m3/s	design for 7.7 to 8.0 Magnitude & peak acceleration 0.65g. And Very high seismicity	Coarse & Fine F at U/s & D/s. D coarse gravel u/s & Fine gravel d/s			Material chos in purpose m earthquake resistant
24	Mornos [103]	Greece	130	✓ [103]			✓			Secondary deposit of weathered flysch			1×10^{-7}	26	0.24	2.09				0.33 : 1	0.33 : 1	≈95			✓	≈1.37 : 1	Typical flysch rocks, siltstone, sandstone & conglomerate	Left abutment steep, ride side likely to slide L/H = ≈4	Transition region from dry climate at eastern and humid at western	Seismic action region. Complex tectonic structure, no fault.	4 m thick at u/s & d/s by separating grain size 50 mm. Gravel & sand drainage at d/s		leakage through karstified limestone	
25	Miboro [70, 81, 121]	Japan	130		✓ [81]				✓		Mixing Disintegrated granite + clayey soil [81,p.555]	Max.Size 4#=40%, 40#=12.5%, 200#=7.3%	$< 1 \times 10^{-5}$	35 - 40	2.8	1.87	10 - 14	20	8 Cov, 20t-SFR	1.5 : 1	-0.85 : 1	87.50			Mt	1.33 : 1	Quartz porphyry & granite porphyry, fault about 70° NE.	L/H = ≈3.3		High Seismicity	Pit run sand and gravel is screened			

Chapter 4

DESIGN OF CORE FOR TEHRI DAM

4.1. General

Tehri Dam, under construction in India is one of the highest earth & rockfill dams of the world. The design of the core of this dam has been examined in the light of the inferences drawn on this aspect in Chapter - 3.

4.2. General Description of Tehri Dam Project

4.2.1. Earth and Rockfill Dam

The dam is located across Bhagirathi River, about 1.5 km downstream of its confluence with Bhillangana River near Tehri Town, Uttaranchal, India (Fig.4.1). The river flows down from the Himalayas. The Himalayas are a seismically active zone.

In view of topography, geology, physical properties of foundation, seismicity, availability of material at Tehri Dam site and economy of construction, an Earth and Rockfill Dam has been provided. The height above the deepest foundation level is 260.5 m. The upstream and downstream slopes of dam are 2.5H:1V and 2.0H:1V respectively. The top of dam is at El. 839.5 m where it is 20 m wide and 575 m long. The cross section of Tehri Dam is shown in Fig.4.2 [49a, 131, and 136].

4.2.2. Foundation Condition

The rocks at Tehri Dam Site comprise phyllitic quartzites and quartzitic phyllite of Chandpur formation. These are banded in appearance, the bands being constituted of argillaceous and arenaceous materials.

On the basis of extent of argillaceous and arenaceous materials present and the varying magnitude of tectonic deformation suffered by them, the rocks are classified into the following categories:

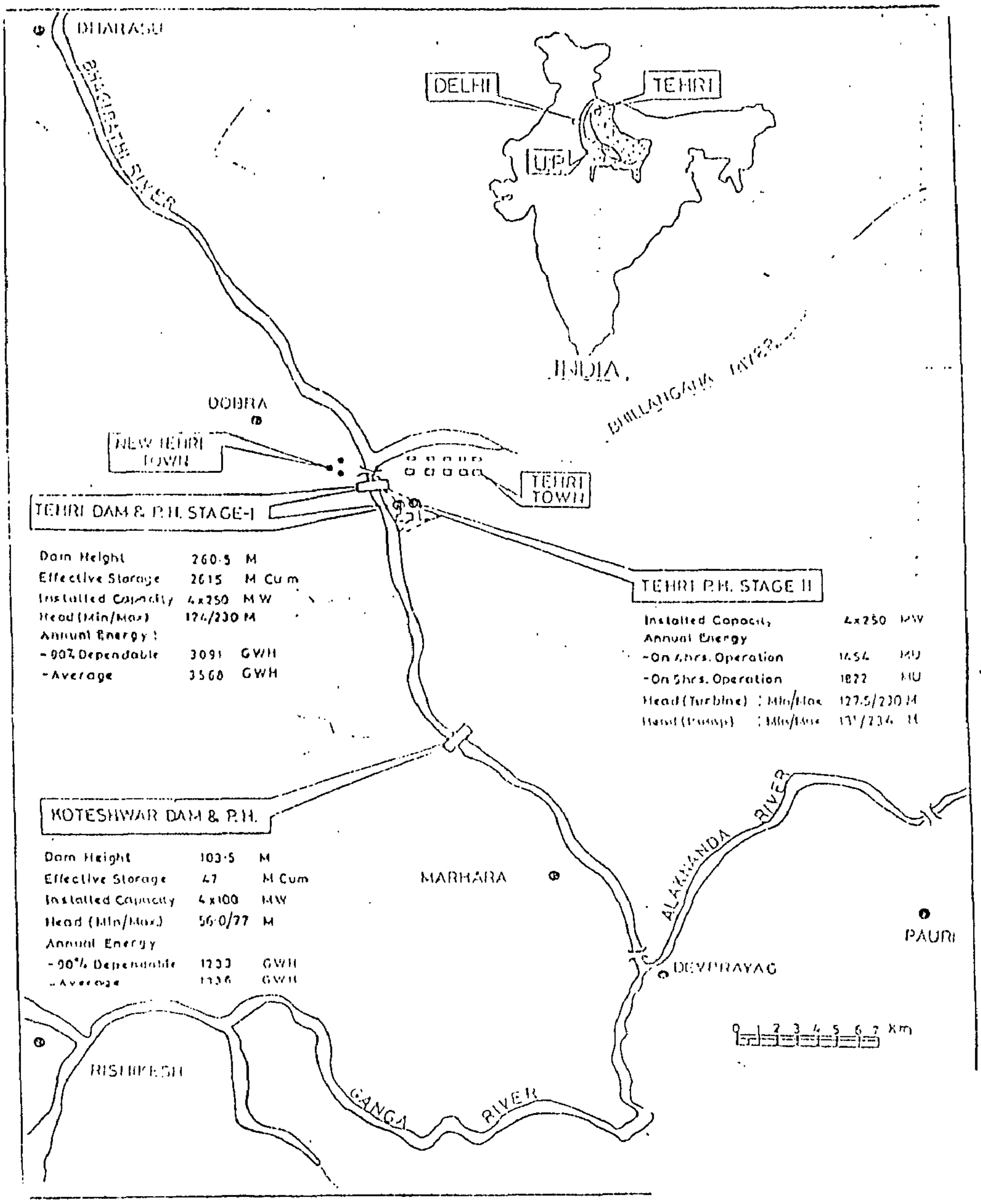


Fig.4.1. Location Map of Tehri Complex, India.

a. Phyllite Grade – I (PQM & PQT).

These are phyllitic quartzite in composition and predominantly arenaceous in nature. These are available in massive form or in thin bedding i.e: phyllitic quartzite massive (PQM) and phyllitic quartzite thinly bedded (PQT). This rock type constitutes about 45 % of the total rocks in the gorge.

b. Phyllites Grade – II (QP).

These are quartzitic phyllite (QP). These are made of alternate bands of arenaceous and argillaceous rocks. This type of rock constitutes about 25 % of the total rock in the gorge.

Grade I and Grade II are nearly of the same quality as far as modulus of deformation and permeability are concerned.

c. Phyllites Grade – III, Sheared Phyllites (SP).

These are composed mainly of argillaceous material and are schistose in nature. The foliation planes, cleavages and joints are very close in minor folds [131].

There are number of shear zones aligned mostly along the foliation direction. The thickness of shear zones vary from 1 to 50 cm. Cross shear zones, longitudinal shear zones and transverse shear zones are also present. The foliation of the phyllites being inclined fairly steeply in a downstream direction, there is no likelihood of any appreciable amount of water loss by way of seepage except through shear zones. The dam lies in a seismically active area in zone IV of the seismic zoning map of India [8a] and Hydro Project Institute (HPI) Moscow considered 15 shear zones which can generate strongest earthquake motion (magnitude 8.0 on MM scale) [8b, 106 and 131].

4.2.3. Spillway Arrangement

Tehri spillways have been designed for a probable maximum flood of 15540 cumecs (inflow). The routed flood discharge at maximum water level (MWL) of 835.0 m through the spillways would be of the order of 13,043 cumecs. It would involve a drop of 220.0 m, which would require suitable arrangement for energy dissipation. The following spillway arrangement is envisaged for discharging surplus water from the reservoir :

- Gated chute spillway on right bank (3 bays each of 14 m span with a jump basin at the end at El.596).

Tabel 4.1. Salient Features of Tehri Hydro Power Complex

Description	*Upstream Reservoir (Tehri Dam)*	*Downstream Reservoir (Koteshwar Dam)*
Reservoir:		
1. Catchment Area (Sq.Km.)	7511	-
2. Max. Flood Level (m)	835.0	615.0
3. Full Reservoir Level (m)	830.0	612.5
4. Min. Draw Down Level (m)	740.0	598.5
5. Gross Storage (million cu.m)	3540	86
6. Live Storage (million cu.m)	2615	36
Dam :		
1. Type	Earth and Rockfill	Concrete Gravity
2. Top Level (m)	839.5	618.5
3. Height (m)	260.5	97.5
4. Top Width (m)	20.0	15.0
5. Length at top (m)	575.0	253.0
Diversion Tunnels :		
1. Type	Horse Shoe	Horse Shoe
2. Size / Nos.	Dia. 11 m / 4	Dia. 11 m / 1
3. Discharge (cumecs)	8120	1180
Spillway		
1. Crest Level (m)	815	594.5
2. Max. Discharge (cumecs)	13043	13240
Water Conductor System:		
1. Head Race Tunnels (Size / Nos.)	Dia. 8.50 m / 4	-
2. Penstock (Size / Nos.)	Dia. 5.75 m / 4	Dia. 6.5 m / 4
3. Tail Race Tunnel (Size / Nos.)	Dia. 9.0 m / 4	-
Power House :		
1. Type	Underground	Surface
2. Installed Capacity (MW)	2000	400
- Conventional Units (Nos x MW)	4 x 250	4 x 400
- Reversible Units (Nos x MW)	4 x 250	-
3. Gross Head (m)	231.5	75
4. Minimum Head (m)	127.5	58
5. Rated Head (m)	188.0	69
6. Speed (rpm)	214.3	142.8
7. Rating	-	-
- Turbine Mode (MW)	255	102
- Pump Mode (MW)	300	-

- Two ungated shaft spillways connected to right bank diversion tunnels (T-3 & T-4).
- Two gated shaft spillways connected to two left bank diversion tunnels (T-1 & T-2).
- An intermediate level outlet at elevation 700.0 m connected to right bank shaft of tunnel T-3.It will fuction as emergency outlet also.

The Salient Features of Tehri Project are given in Table 4.1.

4.2.4. Underground Power House as part of Tehri Hydro Power Plant (HPP).

The underground Power House of 1000 MW capacity is located on the left bank. It consists of four units of 250 MW each and the appurtenant works. Two head race tunnels (HRTs) of 8.5 m diameter, and each about 1100 m length will carry water from reservoir to the power house. The invert level of intake is at El.720 m. It is under construction as Stage-I

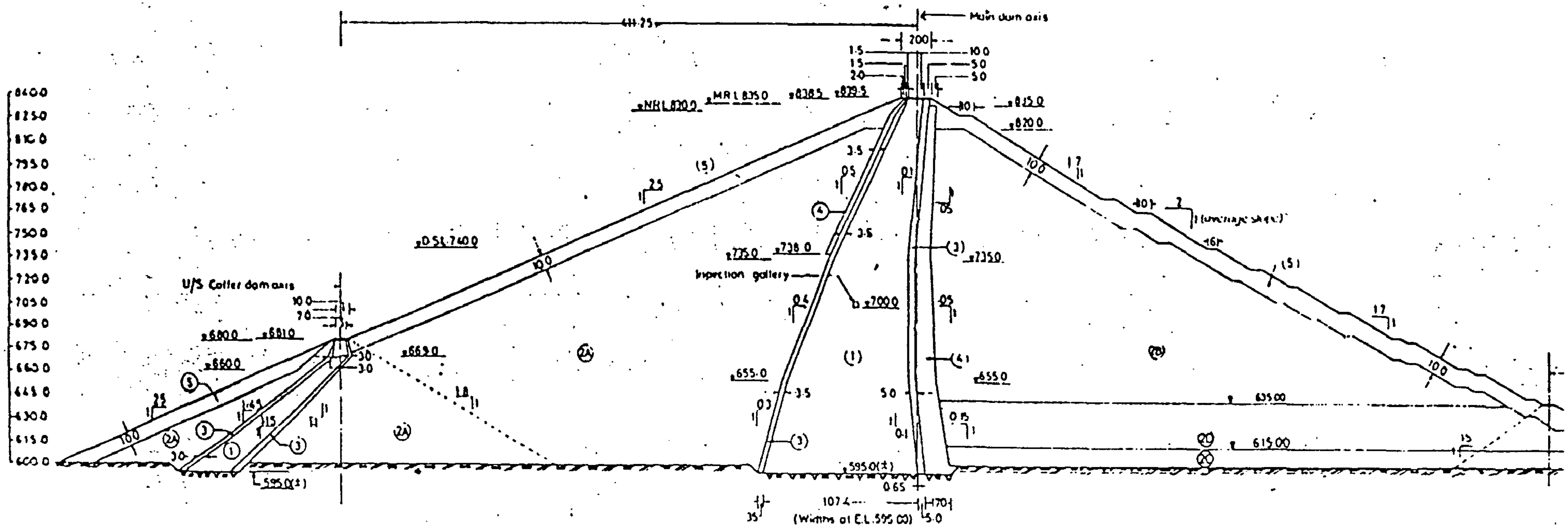
4.2.5. Koteshwar Dam

A 97.5 m high concrete dam is proposed across river Bhagirathi at Koteshwar about 22 Km downstream of Tehri Dam. The Koteshwar Dam Reservoir will have a live storage capacity of 36 million cu.m. This capacity is provided from the consideration that the reservoir may act as Balancing Reservoir for Tehri Power Station and to meet the requirements of the Pump Storage Plant at Tehri. This will augment the functioning of Tehri Power Stations as peaking stations.

A surface Power House of 400 MW (4 x 100 MW units) installed capacity shall be located at toe of the dam.

4.2.6. Tehri Pump Storage Plant (PSP)

An Underground Power House with installed capacity of 1000 MW (four reversible units each of 250 MW) is proposed to be constructed on the left bank. Two Head Race Tunnels of 8.5 m diameter, and each of about 1100m length will carry water from Reservoir to this Power House. There will be two Tail Race Tunnels each of 9.0 m diameter and 1000 m length up to Koteshwar Reservoir.



*(1)*Well Graded Impervious Blended Core Material, *(2A)*Well graded terrace gravelly material, maximum size 600 mm, fines ($\leq$ 4.75 mm) less than 35%, silt content ($\leq$ 0.075 mm) not more than 5%, provided further that 80% of the material should not have aleurite content exceeding 30%, *(2B)*Well graded terrace gravelly material, maximum size 600 mm, fines ($\leq$ 4.75 mm) less than 35%, silt content ($\leq$ 0.075 mm) not more than 5%, *(2C)*Well graded terrace gravelly material, maximum size 600 mm, fines ($\leq$ 4.75 mm) between 10-22%, *(2D)* Well graded terrace gravelly material, maximum size 600 mm, fines ($\leq$ 4.75 mm) between 10-18%, *(3)*Fine filter, silt content less than 3% size $\leq$ 20 mm, *(4)*Coarse filter, sand and gravel mixture , maximum size $\leq$ 60 mm, *(5)*Well graed hard blasted rock with maximum size up to 1200 mm.

Fig. 4.2. Typical Cross Section of Tehri Dam, India.

4.3. Design of Core

4.3.1. Core Slope

A moderately inclined core is adopted for Tehri Dam. It has varying slopes both on upstream and downstream faces. On upstream face the slopes are 0.5H:1V, 0.4H:1V, and 0.3H:1V from upper part to lower part respectively. On downstream face the slopes are -0.1H:1V, 0.05H:1V, and 0.1H:1V from upper part to lower part respectively (Fig.4.2). It is adopted on the considerations of safety and economy which are discussed below.

It was also found that, these slopes were within the range of slopes (upstream face from 0.38H: 1V to 1H:1V and downstream face –0.5H:1V to 0.1H:1V) which have been adopted in high dams reviewed in Chapter 3.

4.3.2. Core Thickness

At Tehri dam, the core material is scarce and therefore, a thiner core is the necessity. Considering all these aspects, the thickness of core for this dam is worked out as 107.4 m (0.4H) at the base. It gives maximum hydraulic gradient of about 2.43. The high dams (height above 200 m) reviewed in Chapter – 3 have base width ranging from 0.5H to 0.33H. *Sherard* [113] who has also stated that cores with a width of 30% to 50% of water head have proved satisfactory under diverse conditions and are adequate for any soil type and dam height. The core width of Tehri dam is in conformity with the *Sherard* criteria and the experience of other high dams [49a, 113 & 131].

4.3.3. Core with Blended Material

The expected characteristics of good core materials are, low permeability, low erodibility, non-dispersibility, flexibility, low plasticity, low compressibility, high shear strength.

The main source of core material for Tehri Dam is the Koti area. The test results have indicated that the loamy material in the upper 8 to 10 m depth is clay of medium plasticity and is non-swelling, non-dispersive, and relatively non erodible.

The tests which have been carried out for the borrow area are: Soil Classification Test, Minerological Composition Test, Dispersion Characteristics Test, Erodibility Test, Permeability Test, Compaction Tests, Consolidation Test, Triaxial Shear Test, Hydraulic Fracturing Tests, Test for Dynamic Shear Modulus, Test for Dynamic Shear Parameters.

The average values of the physical and geo-mechanical characteristics determined by the test are given as: *a)*. The gradation : Clay (15.4%), Silt (57.5%), and Sand (16.1%), Gravels (11.0%). *b)*. Atterberg Limits : Liquid Limit (32.6%), Plastic Limit (21.5%), Plasticity Index (11.1). *c)*. Minerological composition: Illite (64.34%), Kaolinite (19.33%), and Vermiculite (16.33%). *d)*. Permeability: 0.63 x 10^{-7} cm/sec. *e)*. Specific Gravity: 2.68. *f)*. Insitu Moisture content: 14.71%. *g)*. Standard Proctors Compaction Test: Optimum Moisture Content (14.15%), Maximum Dry Density (1.827 gms/cc). *h)*. Consolidation Tests at OMC: at 25 kg/cm2 av = 0.004 cm2/kg, mv= 0.003. *i)*. Consolidation Test at +2% OMC at 25 kg/cm2: av or coefficient compressibility = 0.004 cm2/kg, mv or coefficient of volume change = 0.0029. *j)*. Tiaxial Shear Consolidated Undrained / Consolidated Drained at Saturation : Effective Stress C'=0.57 kg/cm2, ϕ'= 20.9. *k)*. Triaxial Shear Unconsolidated Undrained at OMC: Total Stress (C= 1.74 kg/cm2, ϕ'= 14.9), Effective Stress (C= 1.59 kg/cm2, ϕ'= 16.5). 1). Triaxial Shear Unconsolidated Undrained at +2% OMC: Total Stress (C= 1.25 kg/cm2, ϕ'= 8), Effective Stress (C= 0.98 kg/cm2, ϕ'= 13.4). *m)*. Triaxial Shear Unconsolidated Undrained at +1% OMC: Total Stress (C= 1.59 kg/cm2, ϕ'= 3.7. *n)*. Hydraulic Fracturing Test (Ratio of Hydraulic Fracturing Pressure to Confining Pressure =1.2). *o)*. Dynamic Shear Modulus: at 0.01 Strain = 170 kg/cm2, at 0.0001 Strain = 230 kg/cm2, at 0.000001 Strain = 410 kg/cm2. *p)*. Dynamic Shear Parameter (insitu): ϕ'= 29.5.

These properties show that the material is suitable for core as it has sufficient shear strength and is not too compressible. But the *need for blending* was felt because available material has high percentage of silt content (more than 50%) which makes it susceptible to cracking. It is relatively more compressible also. The shells after compaction will have high density and hence will have very low compressibility. The difference in compressibility of the two materials may lead to stress transfer from core. Therefore, in order to reduce the compressibility of core, it is decided to blend it with pebbly material underlying the loamy material in the Koti Borrow Area. This will improve strength and reduce cracking.

In defining the gradations of blended core material, it was kept in view that besides low compressibility, the core should have enough plasticity and low permeability.

For the core, gradation of the blended material was selected as follows : (also shown in the Fig. 4.3) :

- Clay particles (< 0.002 mm) : not less than 5%.
- Silt : 22 to 50%
- Sand : 44 to 77%
- Pebbles (up to 200 mm) : balance

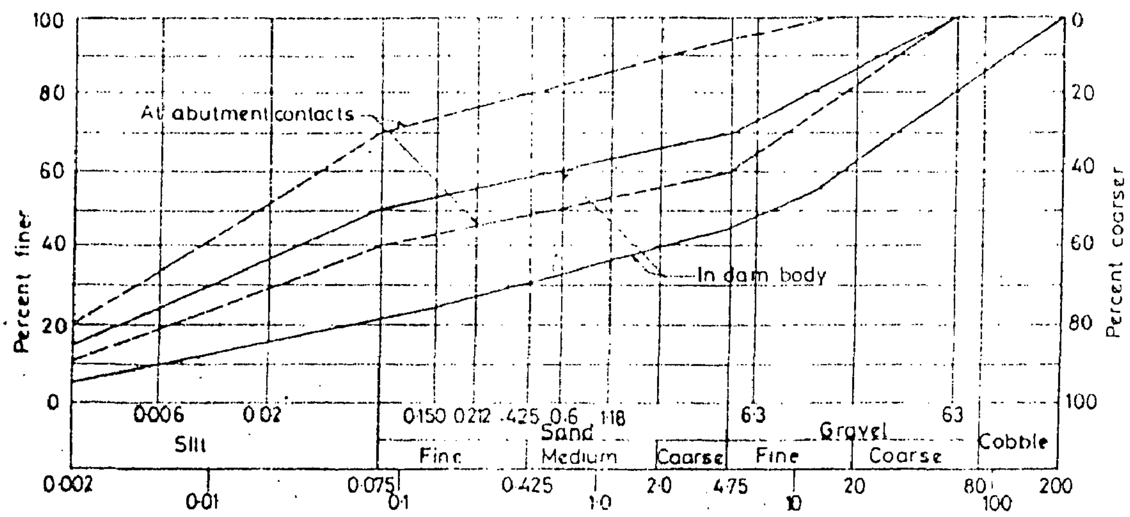


Fig. 4.3. Gradation Curves for Blended Core Material of Tehri Dam, India.

In order to reduce the effect of abutment core interaction leading to possibility of cracking, more plastic material in 2 to 8 m thickness normal to abutment has been proposed. Its gradation is proposed as follows:

- Clay content : 10%
- Silt Content : 40 to 70%

- Sand Content : 60 to 95%
- Gravel up to 63 mm size : balance

Other technical specifications for core material are as follow :

a. Moisture Content.

In the central portion of core it shall be $\pm$ 1% above OMC (optimum moisture content) with average moisture content being at optimum. Adjacent to foundation, abutment and concrete surface, moisture content shall be 2 – 3 % above OMC.

b. Layer Thickness

The layer thickness shall not exceed 40 cm (loose).

c. Compaction and Density

Each layer of core material shall be compacted by minimum 8 passes of 15 ton vibratory roller to achieve an average density of aleurite fraction (< 4.75 mm size) as 1.90 ton / cu.m.

4.3.4. Core Position (Geometry) & Seismic Consideration

Himalayas are a seismically active region. Doubts have been raised about the safety of dam. The doubts appear to be unfounded, as many high dams in the Himalayas such as Bhakra, Pong, Ramganga and Salal in India and Tarbela & Mangla in Pakistan have been constructed and are functioning satisfactorily. Moreover, most of the world's highest earth & rockfill dams are situated in seismically active region, such as Rogun & Nurek Dam in the then USSR with height 335 m & 300 m respectively; Boruca Dam in Costa Rica, Chicoasen Dam in Mexico, Keban Dam in Turkey, Mica Dam in Canada, are of height more than 200 m; and Infiernillo Dam in Mexico and Miboro Dam in Japan are of height, 148 m & 130 m respectively, etc. Infiernillo dam in Mexico and Miboro dam in Japan have experienced repeated earthquakes and an earthquake of magnitude 7.0 without any damage. These earth & rockfill dams are safe structures in high seismic zones due to their flexibility and capacity to absorb large seismic energy.

Tehri Hydro Development Corporation responsible for this project has used latest designed techniques in consultation with the experts of Hydro Project Institute

Moscow who have successfully executed Nurek Dam in a similar seismically active zone [8b, 131 and Chapter 3].

In view of high seismicity of area, detailed design studies have been carried out for ensuring the safety of the dam in the event of an earthquake. These studies relate to site specific assessment of seismicity, testing of fill materials for determining dynamic properties, and detailed dynamic analysis. The Department of Earthquake Engineering University of Roorkee has studied fill materials for the dam by carrying out field tests like Block Vibration Test, Wave Propagation Test and Vertical Dynamic Plate Load Tests. They also carried out liquefaction studies on the fill material using a shake table. The Soviet Consultant carried out dynamic tests on a dynamic triaxial testing machine and determined various parameters required in the analysis of dam using elasto-plastic model [8a & 131].

The core may be located either centrally or inclined upstream. Both vertical and inclined cores have their advantages and disadvantages. Besides, providing better contact with foundation the vertical core also provides more thickness of core for the same quantity of material. The settlement of core is independent of the settlement of downstream shell. In this case both core & shell are placed simultaneously. On the other hand inclined core provides the facility for placing bulk quantity of downstream shell before placing of the core. Foundation treatment can also be carried out independently without disturbing shell placement. During draw down condition, due to flow lines being nearly vertical, the pore pressures developed are considerably less than that in central (vertical) core. The inclined core is said (model studies at University of California 1958 and University of Roorkee 1962, [80, 102b & referred to Tehri Hydro Development Studies]) to be somewhat more earthquake resistant. However, if there is excessive post construction settlement in downstream fill, the sloping core may crack. Also, due to core material being structurally weakest of all fill materials, the upstream shell may become slightly flatter. The downstream slope is likely to be steeper but this effect is almost negligible.

Considering the properties of blended core material and seismicity of the region, a moderately inclined core was considered suitable for Tehri Dam.

The geometry of the moderately inclined core for Tehri Dam has been worked out by using Elasto-Plastic analysis. To optimize the shape of core, nine alternatives (Table 4.2) were analyzed. These studies were also taken up to finalize the need and extent of blasted rock replacing gravel fill in the upstream above the core [131].

Table 4.2 : List of Alternatives for Optimization of dam section , and core geometry of Tehri Dam (see also Fig.4.4)

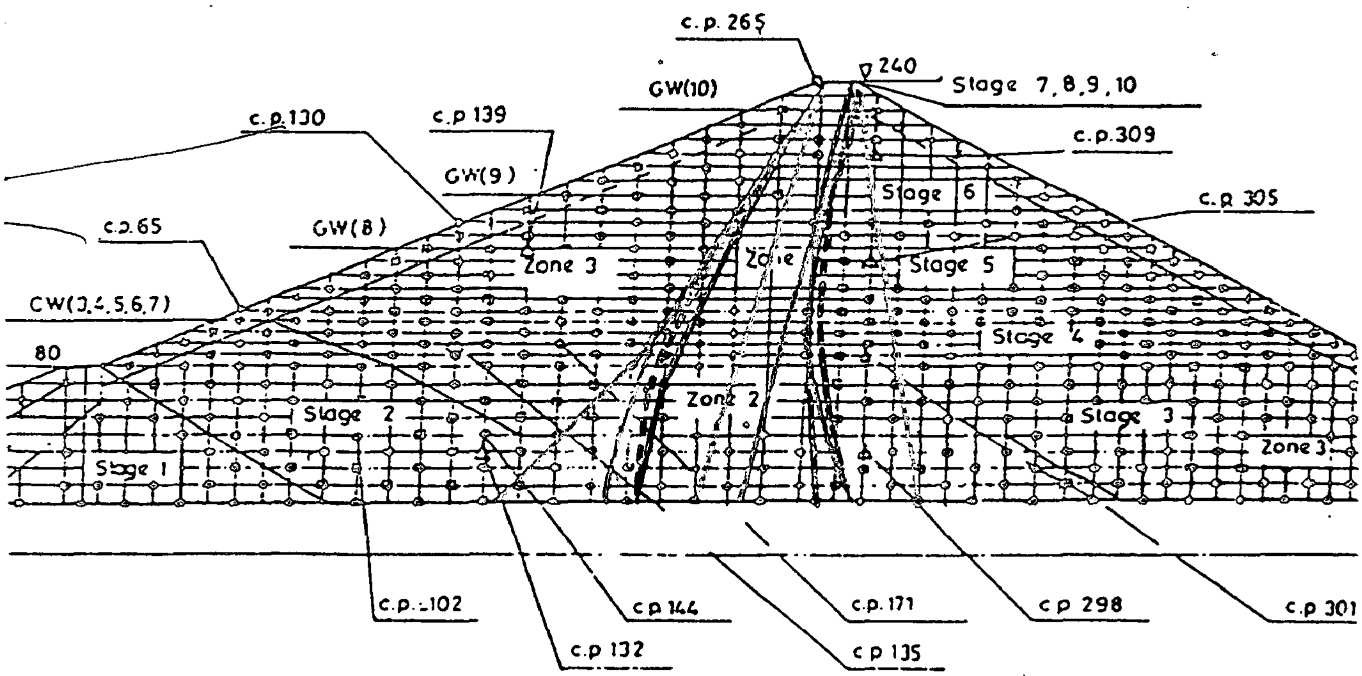
No.	*Alter-native.*	*Design Feature*
1.	1	Design section–*inclined core* with 80 m high blasted rock surcharge (*core shown in red*).
2.	1a.	Design section–*inclined core* with 20 m high blasted rock surcharge (*core shown in red*).
3.	1b.	Design section–*inclined core* with 20 m high blasted rock surcharge (*core shown in red*). Seismic loading is reduced by a factor of 1.5.
4.	2.	*Arch-shaped core* with 80 m high of blasted rock fill surcharge (*core shown in green*).
5.	2a.	*Arch-shaped core* with 80 m high of blasted rock surcharge (*core shown in broken black line*).
6.	2b.	*Arch-shaped core* with 20 m high of blasted rock surcharge (*core shown in broken black line*).
7.	3.	*Alternative core design* with 20 m high of blasted rock surcharge (*core shown in bold black line*).
8.	4.	*Alternative core design* with 20 m high of blasted rock surcharge (*core shown in bold black line*). Core composed of water saturated loam.
9.	5.	Alternative design with *central or vertical core* (*core shown in blue line*).

First of all, alternative 1 (upstream inclined core of 0.3 H thickness with its base flared to 0.5 H thickness with the help of upstream inclined slope of 1:1) and alternative 5 (central vertical core) were studied. It was found that under dynamic loading, plastic zones were formed at the upstream face of the dam with alternative 1 whereas alternative 5 was safe against the two accelerograms (response spectra) developed by Earthquake Engineering Department (University of Roorkee) and

Hydro Power Institute Moscow. The reason for this behaviour was analysed as under-consolidation of upstream shells due to presence of inclined core which resulted in compaction and hence development of excess pore pressures on application of dynamic loading. On the other hand in static condition, stress transfer from core to shell was highly pronounced in case of central core as compared to inclined core. Therefore, optimization studies were carried out to determine the most optimal geometry of the core which would lead neither to formation of plastic zones under dynamic load nor to excessive transfer of stress from core to shell leading to possibility of hydraulic fracture.

Three additional positions of core as represented by alternative 2, 2a, and 3 were studied. It was seen that non-damping plastic zone were not formed in any of these alternatives and stress transfer was also reduced as compared to central core. Stress condition in these 3 alternatives were similar and *alternative 3 (No.7) has accordingly been adopted* keeping in view the site topography and ease of construction. Comparative study carried out in alternative 2a and 2b with and without zone of blasted rock has established that there is no need to replace the gravel shell with the blasted rock.

After that, studies for seismic stability of dam has been carried out by two independent organizations viz Department of Earthquake Engineering, University of Roorkee and Hydroproject Institute, Moscow, who have been supported in their studies by inputs from the Geological Survey of India (GSI), Wadia Institute of Himalayan Geology, India Meteorological Department, National Geophysical Research Institute, Department of Sciences University of Roorkee, Roorkee, and Central Water Commission. In the process, stability of dam has been checked for three accelerograms and by various methods. Finite Element Analysis carried out by DEE, University of Roorkee has established that tensions are not created in the core and that tensions created in the shells are of small magnitude and transitory nature.



• 102, 144 & 171 : Number of check points for *Stress* measurement at earthquake loading.
• 65, 130, & 265 : Number of check points for *Displacement* measurement at earthquake loading
Δ 132, 135, 139, 301, 305 & 309 : Number of check points for *Accelerating* measurement at earthquake loading.

Fig. 4.4. Alternatives for Optimization of Dam Section, and Core Geometry of Tehri Dam

4.3.5. Core Contact Treatment

The design criteria of surface treatment for the core foundation design of Tehri Dam are as follow :

a) The foundation below core has been excavated to fresh, firm, and unweathered rock reasonably free from fissures and cracks. In the transverse direction, the excavated surface is given a slope not more than 2H:1V dipping in upstream direction and no slope dipping in downstream direction. In the longitudinal direction, the excavated surface is given slopes generally less than 1H:1V but not steeper than 0.75H:1V in any case and the sudden change in slope at any point did not exceed more than 20°.

b) All the cracks and joints and shear seams, clay bands or other incompetent materials which were exposed after stripping and cleaning up operations were scooped out to greatest depth practicable (not less than thrice their width at the surface) with the aid of trowel bars and cleaned with air water jets. Small size shears with depression up to 100 mm after cleaning were filled by guniting. Where the width of shear zone was more, it was back filled with M-20 concrete properly compacted and vibrated.

After carrying out the surface treatment of the foundation, the whole surface was covered by a 50 mm thick layer of gunite / shotcrete. Besides this, sub surface treatment of the foundation was also done by grouting the foundation in the core area. Under this treatment a 10 m deep shallow consolidation grouting in entire core rock contact area is being provided through foundation gallery [131].

4.3.6. Filter / Transition Design

Design of filter for Tehri Dam has been checked from various standard criteria like Terzaghi, USBR, and also by Soviet criteria.

Many Researcher / organization have proposed modified criteria for filter design such as:

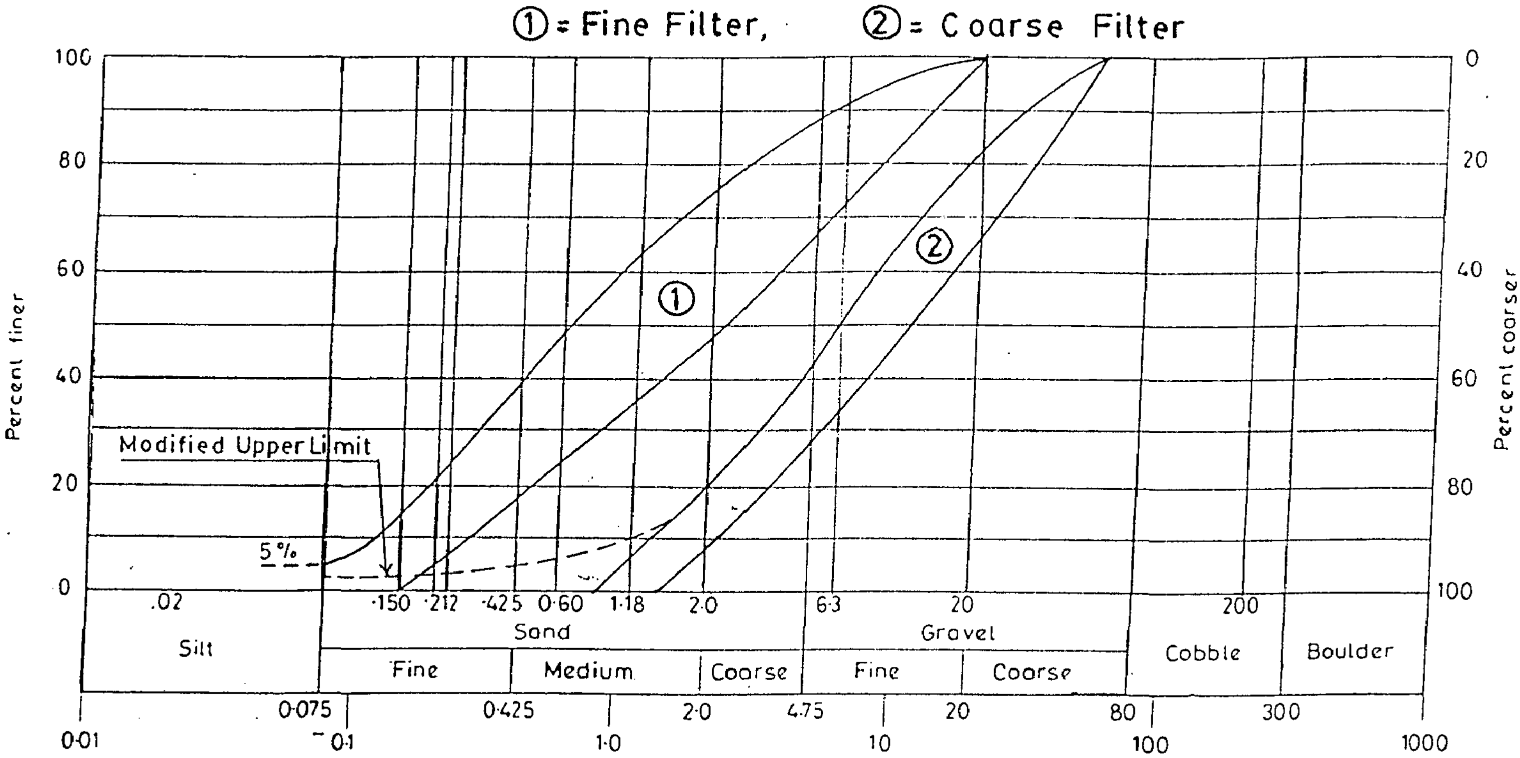
a) Sherard [115] has proposed modified criteria for filter design for silts and clays.

b) Vaughan [143] has suggested concept of a perfect filter, which is designed to retain the smallest particles which can arise during erosion of core material even if they arrive at filter interface.

c) Soviets (now Russian) [51] have very elaborate criteria for design of filters which ensures core material to be piping resistant against transverse as well as longitudinal seepage flow, prevention of disintegration and contact erosion of core material, prevention of irregular spreading of soil particles and its segregation, etc.

The filter zone was designed to be located immediately downstream of the core in order to prevent the failure of dam in the event of cracking of core either because of earthquake or otherwise. Also, the filter is designed to prevent migration of finest particles of core material, thereby not permitting the erosion of core when water flows through the crack [8a].

For Tehri dam, filters are provided on both upstream and downstream faces of core. These filter gradation curve are shown in Fig. 4.5.



(1)For fine filter in Zone 3, and (2) For coarse filter in Zone 4

Fig. 4.5. Gradation Curves for Filter Material of Tehri Dam, India.

4.4. Summary of Study

The above details reveal that for Tehri dam the core is designed by blending locally available materials and is in conformity with the inferences drawn in Chapter – 3 on the basis of the study of existing 25 high embankment dams above 130 m high, particularly above 200 m high.

The bottom thickness of core for Tehri Dam is about 0.4H. This is also in conformity with the inferences in Chapter –3 where for dams above 200 m height, thickness of 0.3 to 0.5 times the water head was found satisfactory.

A moderately inclined core is preferred on the basis of the analysis of several alternatives using FEM under seismic loading condition. Most of high dams in seismic region (chapter 3) are also provided with a moderately inclined core.

The slopes of moderately inclined core of Tehri Dam (upstream face 0.5H:1V, 0.4H:1V & 0.3H:1V from upper to lower part respectively, and downstream face –0.1H:1V, 0.05H:1V & 0.1H:1V from upper to lower part respectively) are in conformity with slopes provided in ten high dams (Chapter-3).

Chapter 5

Conclusions and Suggestions for Further Study

5.1. Conclusions

5.1.1. General

The design of core has been reviewed in 25 high embankment dams of the world and the details are given in Chapter-3. These have been regrouped according to the position and shape of core in Tables 5.1 to 5.3. Other special features are also given in these tables. The following observations are made from the Study of these details.

1. Core is a critical component of a Zoned Embankment.
2. Important parameters of core design are:
   (i). Selection of suitable material.
   (ii). Core thickness.
   (iii). Position of Core.
3. The design is site specific and depends on the design requirements and locally available material and its properties. The design of section in important dams can be analyzed by FEM for stresses & deformations and section core be modified accordingly. It is essential in high dams located in seismic area.
4. High Zoned Embankments have been found satisfactory for wide range of valley shapes (L/H = 1.1 to 10.0)
5. Core geometry and its position has not been found dependent on seismic condition and L/H ratio.
6. Most of dams above a height of 150 m are provided with blended core in order to reduce compressibility and the possibility of cracking and to increase erosion resistance.
7. All the dams above a height of 200 m have a base width in the range of 0.33 to 0.5 times the water head.

8. In highly seismic zones high dams have been constructed with vertical as well as moderately inclined cores. In some of these cases the core is inclined. Both Infiernillo Dam (148 m high, with vertical core) in Mexico and Miboro Dam (130 m high, with inclined core) in Japan have performed satisfactorily during repeated strong earthquakes. Moderately inclined core of Oroville Dam (235 m high) in Northern California, USA, has experienced a major earthquake (M5.8) with epicentre only 9.3 km from dam site in 1975, without any damage.
9. Most of the dams have been compacted with an adjustment of moisture content a little more than its optimum moisture content.
10. In some cases, a concrete block is provided on which the core rests
11. In all the dams the foundation below the core base has been consolidated by grouting.

5.1.2. For Tehri Dam

The Study of design of Core for Tehri Dam has shown that its core is in conformity with the inferences derived from the study of existing high dams.

5.2. Suggestions for Further Study

Further data shall be collected and study shall be carried out to verify the hypothesis that inclined core is better in highly Seismic zones.

Tabel 5.1. Vertical Core Embankment Dams (13 nos).

1	2	3	4	5	6	7	8	9
No.	Name of Dam	Country	Height of Dam (m)	L/H	Approx. Maximum Hyd. Gradient	B/H	Blending / Mixing / Processing Condition	Remarks on Seismic Condition
1	Nurek	USSR	300	3.0	2	0.5	yes	Seismic shocks might be of magnitude as high as 8 - 9
2	Chicoasen	Mexico	261	1.1	2.38	0.4	yes	Highly Seismic
3	Darmouth	Australia	180	4	1.4	0.7	yes	Within Seismic Area
4	Trinity	USA	164	4.56	0.55	1.8	yes	The dam is in Zone 3 (1973 Seismic Zone Map)
5	Palo Que-Mado	South-Africa	160	2.31	1.52	0.7	no	High Seismic Condition
6	Swift	USA	156	5	2.11	0.5	yes	Maximum Earthquake of magnitude 5.5 , happened
7	Geosche-neralp	Switzerland	155	2.32	3.08	0.3	yes	-
8	Gepatsch	Austria	153	3.95	3.7	0.3	yes	-

L = Length of dam along the axis at the top; B= Base width of core; H= Height of dam.

Tabel 5.1. Vertical Core Embankment Dams (13 nos), Continued

1	2	3	4	5	6	7	8	9
No.	Name of Dam	Country	Height of Dam (m)	L/H	Approx. Maxi-Mum Hyd. Gradient	B/H	Blending / Mixing / Processing Condition	Remarks on Seismic Condition
9	Infiernillo	Mexico	148	2.32	4.93	0.2	no	Frequent and intensive Earthquakes, and High Seismic Zone Condition
10	Netzahua-locoyotl	Mexico	137.5	2	2	0.5	no	Within Seismic Zone
11	Beas	India	132.6	14.5	0.7	1.4	yes	Highly Seismic Zone Region
12	Ambuklao	Philipi-nes	131	2.85	2	0.5	no	Safety Factor due to Earthquake has been considered
13	Mornos	Greece	130	4	1.37	0.7	no	The tectonic structure has an intensive plate characteristic with stronglyfolded cyncline & anticlyne.

L = Length of dam along the axis at the top; B= Base width of core; H= Height of dam.

Tabel 5.2. Moderately Inclined Core Embankment Dams (10 nos).

1	2	3	4	5	6	7	8	9
No.	Name of Dam	Country	Height of Dam (m)	L/H	Approx. Maxi-mum Hyd. Gradient	B/H	Blending / Mixing / Processing Condition	Remarks on Seismic Condition
1	Rogun	USSR	335	1.53	2	0.5	yes	High Seismic & Tectonic Tectonic activity, Intensity Grade IX.
2	Borucca	Costa-rica	302	2.99	2	0.5	yes	Velocities vary ; 1800-2800 m/sec.
3	Guavio	Colom-bia	245	1.6	2.94	0.3	yes	-
4	Mica	Canada	244	4	2.3	0.5	yes	Core slightly-inclined to counter Earthquake
5	Chivor	Colombia	237	1.3	3	0.3	yes	Highly Seismicity

L = Length of dam along the axis at the top; B= Base width of core; H= Height of dam.

Tabel 5.2. Moderately Inclined Core Embankment Dams (10 nos) , *Continued*

1	2	3	4	5	6	7	8	9
No.	Name of Dam	Country	Height of Dam (m)	L/H	Approx. Maximum Hyd. Gradient	B/H	Blending / Mixing / Processing Condition	Remarks on Seismic Condition
6	Oroville	USA	235	8.99	2.94	0.3	yes	A 0.1g horizontal seismic was included in analysis. Earthquake Model under direction Prof.H.Bolton Seed
7	W.A.C-Bennet	Canada	183	11.2	1.1	0.9	yes	-
8	Cougar	USA	158	3.09	3.7	0.3	yes	Seismic Stability of the embankment was evaluated
9	Round-Butte	USA	134	3.3	2	0.5	yes	-
10	Publeviejo	South Africa	130	1.9	2.6	0.4	no	Very High Seismicity

L = Length of dam along the axis at the top; B= Base width of core; H= Height of dam.

Tabel 5.3. Inclined Core Embankment Dams (2 nos).

1	2	3	4	5	6	7	8	9
No.	Name of Dam	Country	Height of Dam	L/H	Approx. Maxi-mum Hyd. Gradient	B/H	Blending / Mixing / Processing Condition	Remarks on Seismic Condition
1	Tarbela	Pakis-tan	145	10	4	0.3	yes	High seismicity & active seismic area
2	Miboro	Japan	130	3.3	1.33	0.8	yes	High seismicity

L = Length of dam along the axis at the top; B= Base width of core; H= Height of dam.

REFERENCES

1. Airapetyan,R.A, "Modern Rockfill Dams," *Textbook*, Energiya, Moskva (Russia), 1968.
2. Alberro,J. and, Moreno,E, "Interaction Phenomena in The Chicoasen Dam :Construction of First Filling," *14th ICOLD Congress, Rio De Janeiro (Brasil),* 1982, Q.52, R.10, pp.183-201.
3. Amaya,F.,et al., "Pore Pressure Measurements in The Core of Chivor Dam," 10th *Int.Conference On Soil Mechanic Foundation Engineering, Stockholm*, 1981, pp. 19-29.
4. Anagnosti,P.,Prof.Dr., "Evaluation of Shear Strength for Coarse-Grained Granular Materials," *14th ICOLD Congress, Rio De Janeiro (Brasil),* 1982, Vol. IV, pp.753-766.
5. Andrade,V.D., "Construction of Rockfill Dikes Transition Zones and Clay Core Placed in Deep Water for Itaipu Cofferdams," *14th ICOLD Congress, Rio De Janeiro (Brasil),* 1982, Vol. IV, pp.47-68.
6. Asawa,G.L.,Dr., Mittal,M.K.,Prof., et.al., "Design of Earth and Rock-Fill Dams," *Short Term Course*, Dept. Of Water Resouces Govt.of Orissa Bhubaneshwar & Centre for Continuing Education Univ.of Roorkee, May 2000, India.
7. Baldovin,G., Allaria,N., and Baldovin,E. "The Core of Castagnara Dam on Metramo River," *17th ICOLD Congress,* Vienna (Austria), Vol. 4, 1991, pp.179-196.
8. Balissat,M, and Bassoney,C., "Dynamic Behaviour of Three Rockfill Dams," *13th ICOLD Congress,* New Delhi (India), 1979, Q.51, R.58, pp.1115-1133.

8a. Balissat, M., & Wilhelm, R., " Design and contruction of the PuebloViejo Dam," Water Power and Dam construction, july 1986, pp. 65-70.

8b. Bansal, L.K., and Aggarwal, M., "Safety Evaluation of Tehri Dam in Seismic Environment," *Technical Proceedings Seminar on Hydropower Development,* Uttrakhand and Adjoining Himalayan Region, Feb.1997, pp.8292.

9. Bayer,F.L, "Grouting of an Aged Dam Core to Re-establish its Tightness, *Proceeding of 12th International Conference On SMFE,* Rio De Janeiro (Brasil), 1989, pp.1385-1388.
10. Beier, H., et al., "Subsequentt Sealing in The Impervious Core of the Sylvenstein Dam," *13th ICOLD Congress,* New Delhi (India), 1979, Q.49, R.8, pp.103-115.
11. Bennett,P.T., "Notes On Embankment Design," 4th *ICOLD Congress,* New Delhi, 1951, Report 28, Question 13, pp.3-13.
12. Bird,J.M., "Uncertainties in Earth Dam Design," *Journal of the SMFE,* Proc. of ASCE, SM.3, June1961, pp.33-67.
13. Blight,G.E., "Stresses in Narrow Cores and Core Trenches of Dams," *11th ICOLD Congress,* Madrid (Spain), 1973, Vol. III, Q.42, R.5, pp.63-79.
14. Bony, Marjono, "Erosive Resistance of Core Materials," *M.E. Disst.*, WRDTC, University of Roorkee, Roorkee, India, 1974.
15. Borovoi,A.A.; Mikhailov,L.P.; Moiseev,I.S.; and Radchenko,V.G., "Soils for and Methods of Embankment Dam Construction," *14th ICOLD Congress, Rio De Janeiro (Brasil),* 1982, Vol. IV, pp.503-515.
16. Bravo, G., "Effect of Step Slopes on the Canale's Dam Core," *14th ICOLD Congress, Rio De Janeiro (Brasil),* 1982, Vol. II, pp.989-1000.
17. BRE Building Research Series, "Dams and Embankments", *Text Book,* The Construction Press, England, 1978.
18. Cavounidis, Sypros; and Höeg, Kaare, "Consolidation During Construction of Earth Dams," Journal of *Geotechnical Engineering Division,* GT10, Oct. 1977, pp.1055.
19. Cadman,J.D., et al., "The Complex Methamorphic Rock Foundation Conditions of the Tucurui Hydroelectrical Project," *14th ICOLD Congress, Rio De Janeiro (Brasil),* 1982, Vol. II, pp.811-834.
20. Carlyle,W.J., "The Design and Performance of The Core of Brianne Dam," *11th ICOLD Congress,* Madrid (Spain), 1973, Q.42, R.26, pp.431-455.
21. Chadha, B.R., "Clay Chute at the Salal Dam," Water Power & Dam Construction, June 1982, pp.30-31.

22. Charles,J.A., and Watts,K.S., "TheMeasurement and Significance of Horizontal Earth Pressures in Puddle Clay Cores of Old Earth Dams, *Proceedings of Institution of Civil Engineer*, Vol.82, December 1987, Part.1, pp.1241-1268.
23. Committee On Earth and Rockfill Dams, Progress Report, "Problems in Design and Construction of Earth and Rockfill Dams," *Journal of the Soil Mechanics and Foundations Division*, Proc. ASCE, May 1967, SM3. pp.129-156.
24. Creager, W.P; Justin,J.D; and Hinds,J., "Engineering for Dams," *Text Book*, 1st Indian Reprint , 1995 Published by Nem Chand, Civil Lines, Roorkee,India.
25. Davidson,L.W., "Upgrading The Impervious Core Quality at Sugar Pine Dam," Q55, R25, ," *14th ICOLD* 1982, Vol. IV, pp.589-607.
26. Davis,F.J., and Wagner,A.A., "Control of Earth and Rockfill for Oroville Dam, Journ.SMFED, SM1, Jan.1967, pp.151-155.
27. Dolezalova M, " Effect of Steepness of Rocky Canyons Slopes On Cracking of Clay Cores of Rock-and-Earthfill Dams," *10th ICOLD Congress,* Montreal (Canada), 1970, Q.36, R.13, pp.215-223.
28. Duncan, James Michael, "State of The Art: Limit Equilibrium and Finite-Element Analysis of Slopes," *Journal of Geotechnical Engineering Division*, Proc. ASCE, July 1996, pp.577-593, (Example Analysis of New Mellones Dam).
29. Eggenberger,W.' "The Goeschenenalp Rock-Fill Dam Project, Swtitzerland," Proceeding of the 3th Int.Conf. On SMFE, Switzerland, Aug, 1953, pp.296-301.
30. Eisenstein,Z. & Law,S.T.C., "Analysis of Consolidation Behaviour of Mica Dam," *Journal of Geotechnical Engineering Division*, Proc. ASCE, August 1977, pp.879-892.
31. Emmelin,L., "Discussion Q42," *11th ICOLD Congress,* Madrid (Spain), 1973, Vol. IV, pp.430.
32. Engineering Staff, "Large Dams in Philipines," *Paper*, National Power Corporation, Manila
33. Eric.B.K.G.& Chad Wick, W.L., "Development of Dam Engineering in USA," Printed 1988 for 16th ICOLD Congress.
34. Fucik,E.M., and Edbrooke,R.F., "Ambuklao Rock-Fill Dam, Design and Construction," Trans.ASCE, 1960, Paper, No.3057, pp.1207-1227.
35. Gamboa,J., and Benassin,A., "Behaviour of Netzahualcoyotl Dam During Construction," Journ. SMFED, Proc.ASCE, SM4, July,1967, pp.211-229.
36. Gilg,B.(Electro-Watt-Zuric), "Theimpervious System of The Mattmark Dam, "*11th ICOLD Congress,* Madrid (Spain), 1973, Vol. III, pp.525-538.
36a. Goldin,A.L. & Rasskazov,L.N., "Design of Earth Dams," Textbook, Translated & Edited by Zeidler,R.B., Moscow, 1987.
36b. Goel,M.C., "Construction Pore Pressure - Case Study of Ramganga Dam," Paper, Univ.ofRoorke,Roorkee, India.
37. Golze,A.R., "Oroville Dam," *Journal VBB Consulting Engineers and Architects (Sweden), 1965*, (The State of California Department of Water Resources, Division of Design and Construction, Oroville Office, Sacramento), pp.242-254.
38. Gordon,J.L., and Duguid,D.R., "Experiences with Cracking at Duncan Dam," *10th ICOLD Congress,* Montreal (Canada), 1970, Q.36, R.29, pp.469-485.
39. Goyal,K.C, J.N. Srivastava, and V.K. Joshi, "Impervious Materials and Slope Protection at Ramganga Dams," *11th ICOLD Congress,* Madrid (Spain), 1973, Vol. III, pp.719-730.
40. Grandi,A.L, et.al, "Earth Dam in Argentina," *Proc. 5th Int.Conf. on SMFE*, July 1961, Vol.II, pp.613-618.
41. Growdon,J.P, "Rockfill Dams: Dams with Sloping Earth Cores," *Transaction ASCE*, 1960, Paper No.3069, pp.207-225.
42. Growdon,J.P., "Rockfill Dams: Performance of Sloping Core Dams," *Transaction ASCE*, 1960, Paper No.3070, pp.237-254.
43. Guillen, G.B., et al., "In Situ Tests for Control of Cracking of the Clay Core of Canales Dam (Spain)," *14th ICOLD Congress, Rio De Janeiro (Brasil)*, 1982, Vol. IV, pp.989-1005.
44. Gupta, D.K, "Influence of Core Geometry on Dynamic Response of Earth and Rockfill Dam," *M.E. Dissertation,* Dept.of Earthquake Eng., Univ.of Roorkee, Roorkee, India, July 1988.

45. Hasssouna,Moh.E., "Filling and Compaction of Sand By Deep Vibrators and Vibrorollers in The High Aswan Dam," *10th ICOLD Congress,* Montreal (Canada), 1970, C.1, pp.411-431.
46. Hjeldness,E.I., and. Lavania.B.V.K, "Cracking, Leakage, and Erosion of Earth Dam Materials," *Journal Geotechnical Engineering*, Proc. ASCE, GT.2, February 1980, pp.117-135.
47. Hollingworth, F., "Filter Cloth Partially Replaces and Supplements Filter Materials for Protection of Poor Quality Core Material in Rockfill Dam," *15th ICOLD Congress, Rio De Janeiro (Brasil),* 1982, Vol. IV, pp.709-724.
48. ICOLD, "Discussion," *6th ICOLD Congress,* New York, 1958, Report 8, Question 22, pp.124-127.
49. Idel, K.H., et al., "Influence of Geomechanical Investigation on Comparison of a Rockfill Dam with Upstream Membrane and an Earthfill Dam with Sealing Core," *14th ICOLD Congress, Rio De Janeiro (Brasil),* 1982, Vol. II, pp.309-323.
49.a. Irrigation Department,U.P., "Design & Construction of Tehri Dam Project," *Report of the Composite Team on Foreign Study Tour*, Vol. I & II, July 1985.
50. Jasson,S.and Nilson,A, "Experience of Halloysitic Clay in Dam fills and Foundations at Mrica Dam Indonesia," *17th ICOLD Congress,* Vienna (Austria), 1991, pp.1-11.
51. Jatana,B.L., "Embankment Dams," *Theme Paper*, U.P. Irrigation Department, Lucknow.
51a. Jernelius,I., "Indonesia's Mrica Hydro Project," Water Power & Dam Construction, July 1986, pp.15-18.
52. Kassiff,G., et al., "Analysis of Filter Requirements for Compacted Clays," *Proc. 6th Int. Conf. On SMFE,* Montreal (Canada), 1965, Vol.II, pp.495-499.
53. Kawakami,F., "Compation methods and field Moisture Content of The earth Core for Earth Dams Using The moist Materials," *6th ICOLD Congress,* New York, 1958, Report 28, Question 22, pp.165-183.
54. Kawashima, T., and Kanazawa, K. "Design of Rockfill Dams on Weathered Foundation with Large Scale Faults," *14th ICOLD Congress, Rio De Janeiro (Brasil),* 1982, Vol. II, pp.75-88.
55. Kennard,M.F., Penman, A.D.M., and Vaughan, P.R., "Stress and Strain Measurements in The Clay Core at Balderhead Dam," *9th ICOLD Congress,* Istambul, 1967, Q.31, R.9, pp.129-149.
55a. Khan, S.N. & Naqvi,A., "Foundation Treatment for Under Seepage Control at Tarbela Dam Project," *10th ICOLD Congress,* Montreal (Canada), 1970, Q.37 R.60, pp.1167-1193.
56. Kim,Y.I., "Dam Behaviour Measured by Embedded Instruments," *13th ICOLD Congress,* New Delhi (India), 1979, Q.49, R.28, pp.419-438.
57. Knight,D.J., et al., "Materials and Construction Methods for a Very Wet Clay Core Rockfill Dam at Monasavu Falls, Fiji," *14th ICOLD Congress, Rio De Janeiro (Brasil),* 1982, Vol. IV, pp.293-303.
58. Kobayashi, S., and Ichimasu,Y., "Grouting of a Fault to Stabilize The Core Zone of Nanakura Dam," *14th ICOLD Congress, Rio De Janeiro (Brasil),* 1982, Vol. II, pp.119-142.
59. Lane, K.S., "Flexibility of Clay and Cracking of Earth Dam," *Journal of the SMFE*, Proc. of ASCE, SM.6, Nov., 1963, pp.109-122.
60. Leliavsky,Serge, "Design of Dams for Percolation and Erosion," *Text Book*, 1st Published, Great Britain, London, 1965.
61. Leonard,G.A., and Narain,J., "Flexibility of Clay and Cracking of Earth Dams," *Journal of the SMFE*, Proc. of ASCE, SM.2, March, 1963, pp.47-98.
62. Leonard,G.K., and Raine,O.H., "Rockfill Dams: TVA Central Core Dams," *Transaction ASCE*, 1960, Paper No.3068, pp.190-205.
63. Li,C.Y., "Construction Pore Pressures in Three Earth Dams," *Journal of the SMFE*, Proc. of ASCE, SM.2, March, 1967, pp.1-43.
64. Lloyd,H.E., et al., "Rockfill Dams: Cherry Valley Central Core Dam," *Transaction ASCE*, 1960, Paper No.3075, pp.336-359.

65. Lofquist, B., "Earth Pressure in A thin Impervious Core," 4th *ICOLD Congress,* New Delhi, 1951, Report 13, Question 13, pp.1-11.
66. Logani,K.L., et al., "Unusual Foundation of Cerron Grande Dam," *14th ICOLD Congress, Rio De Janeiro (Brasil),* 1982, Vol. II, pp.661-679.
67. Lowe,J. (III), "Recent Development in The Design and Construction of Earth and Rockfill Dams," *10th ICOLD Congress,* Montreal (Canada), 1970, Q.36, GR.30, pp.1-28.
68. MacDonald,D.H, et al., "The Geotechnical Properties of Fill Materials in Some Canadian Dams," Proc. 5th Int.Conf. on SMFE, July 1961, Vol.II, pp.657-662.
69. Maksimovic, M., "Optimum Position of The Central Clay Core of a Rockfill Dam in Respect to Arching and Hydraulic Fracture," *11th ICOLD Congress,* Madrid (Spain), 1973, Q.42, R.44, pp.789-800.
70. Mandzhavidze,N.F, and Mamradze,G.P., "The High Dams of the World (Systematic Tables of Data and Bibliography on Dams over 75 m High)," *Text Book*, Academy of Sciences of the Georgia SSR. Institute of Power Engineering, Tbilisi, 1963.
71. Marsal,R.J., and Arellano,L.R., "Performance of El Infiernillo Dam, 1963-1966," Journ. SMFED, Proc.ASCE, SM4, July 1967, pp.265-298.
72. Marulanda,A., and Amaya,F. "The Design and Construction of Colombia's Guavio Dam," *Journal Water Power & Dam Construction*, December 1989, pp.41-50.
73. Mason,P.J., "The Evolving Dam," *The International Hydropower and Dams*, Vol.4, 1977, Issue 5, pp.69-70.
74. McConnell,A.D., "Materials and Construction Methods for the Dam and Dyke Embankments of the LG-4 Projects, (Canada)," *14th ICOLD Congress, Rio De Janeiro (Brasil),* 1982, Vol. IV, pp.123-200.
74a.Mermel,T.W., " Major Dams of the world,"Water Power & Dam Construction, May 1982, pp.93-103 .
75. Meyer-Peter, Prof.Dr., "Soil Mechanics and Foundation Problems of the Marmorera Earth Dam (Switzerland)," *Proc. 5th Int.Conf. on SMFE*, July 1961, Vol.II, pp.302-311.
76. Moammar, G., et al., "Evolution of the Salado Dam Design in Accordance with Geological-Geotechnical Investigation," *14th ICOLD Congress, Rio De Janeiro (Brasil),* 1982, Vol. II, pp.503-516.
77. Moiseev,I., "The World's Highest Dam Conquers Harsh Environment," *Journal Water Power & Dam Construction*, March 1986, pp.17-19.
78. Moreno,E., "Long Term Interaction Cases History of the Chicoasen Dam, Mexico," *17th ICOLD Congress,* Vienna (Austria), 1991, Q.67, R.18., pp.595-603.
79. Moreno,E, and Alberro,J., "Behaviour of The Chicoasen Dam:Construction of First Filling," *14th ICOLD Congress, Rio De Janeiro (Brasil),* 1982, Q.52, R.9, pp.155-165.
80. Mundal,T., "Mundal on Sloping Earth Core (Discussion)," *Transaction ASCE,* 1960, pp.226-235.
81. Mundal,T, "Rockfill Dams: Brownlee Sloping Core Dam," *Transaction ASCE,* 1960, Paper No.3082, pp.520-544.
82. Murley,K.A., and Cummins,P.J., "Design Considerations of Materials During Construction of Darmouth Dam, Australia," *15th ICOLD Congress, Rio De Janeiro (Brasil),* 1982, Vol. IV, pp.627-640.
83. Murthy,Y.K., et al., "Influence of Core Position on Stability of rockfill Dam Founded on Sand, Gravel and Boulder Deposits in Seismic Zone," *11th ICOLD Congress,* Madrid (Spain), 1973, Vol. III, Q.42, R.40, pp.707-717.
84. Nayak, J.N., "A study of Alternative Methods of Earth Handling On Tehri Dam," *M.E. Dissertation,* WRDTC, Univ.of Roorke, Roorkee, India, 1981.
85. Nonveiller, E.E., "Compation methods and moisture content for material used in the construction of earth core and supporting fill for earth and rockfill dams," 6th *ICOLD Congress,* New York, 1958, General Report, Question 22, pp.73-99.
86. Nonveiller, E.E., "Streeses and Deformation in Cores of Rockfill Dams," *Proc. 5th Int.Conf. on SMFE,* July 1961, Vol.II, pp.673-680.
87. Palta,B.R., et al., "Impervious Core and Slope Protection of Beas Dam, "*11th ICOLD Congress,* Madrid (Spain), 1973, Vol. III, pp.743-763.

88. Patel,V.J,Dr., et.al., "Pores Pressure in Earth Dams," Proc. 5^{th} *Int.Conf. on SMFE*, July 1961, Vol.II, pp.687-690.
89. Patel,V.B. (Chairman), "*Indian Standard*," Guidelines for Design of Large Earth and Rockfill Dams, (IS:8826-1978), pp.1-26.
90. Patric,J.G., "Post-Construction Behaviour of Round Butte Dam," *Journ. SMFED*, Proc.ASCE, SM4, July,1967, pp.251-259.
91. Patterson,F.W., and McDonald,D.H., "Rockfill Dams: Bersimis Sloping Core Dams," *Transaction ASCE*, 1960, Paper No.3081, pp.488-519.
92. Pare, J.J, et.al., "Design of Earth-and Rockfill Dams for La Grande," *Water Power and Dam Construction*, January 1978, pp.38-45.
93. Pena, G.D., et al., "Influence of Geology and Geotechnics on Design of Pueblo Viejo Dam," *14^{th} ICOLD Congress, Rio De Janeiro (Brasil)*, 1982, Vol. II, pp.849-857.
94. Penman,A.D.M., et al., "Effect of the position of the core on the behaviour of two rockfill dams," *11^{th} ICOLD Congress*, Madrid (Spain), 1973, Vol. III, Q.42, R.18, pp.315-338.
95. Pircher,W., and Schwab,H., "Austrian's Finstertal Rockfill Dam," *Journal Water Power & Dam Construction*, June 1980, pp.43-46.
96. Pope,R.J., "Evaluation of Cougar Dam Embankment Performance," *Journ. SMFED*, Proc.ASCE, SM4, July,1967, pp.231-250.
97. Priscu, R.; Propovici, A.; Stematiu, D; and Stere,C., "Earthquake Engineering for Large Dams," *text book*, 2^{nd} ed., 1985, published by John Wiley & Sons, pp.283-372.
98. Progress Report Committee on Earth and Rockfill Dams,"Problems in Design and Construction of Earth and Rockfill Dams," *Journ. SMFED*, Proc.ASCE, SM3, May,1967, pp.129-136.
99. Rao,K.L, Dr, "Stability of Slopes in Earth Dams and Foundation Excavations," Proc. 5^{th} Int.Conf. on SMFE, July 1961, Vol.II, pp.691-695.
100.Reinus,E. and Fagerstrom,H., "Impervious Earth Fill For Holjes Dam," *Paper*, The Institution of Civil Engineers, March,1966, London.
101.Robert ,E. & Luccia, "Earth Dam Construction in a Wet Climate," 6^{th} *ICOLD Congress*, New York, 1958, Report 128, Question 22, pp.385-407.
102.Rocha,M., "The Application of Cement Stabilized-Soil in The Construction of Earth Dams," Proc. 5^{th} Int.Conf. on SMFE, July 1961, Vol.II, pp.701-708.
102.a. Salembier, "Leakage control works at the Rouchain Dam," *Water Power and Dam Construction*, December 1984, pp.15-20.
102b. Saini, S.S., and Chandrasekaran, A.R., "Discussion on: Effect of Core on Earthquake Response of Earthdams", Proceeding of the Symposium on Earth and Rockfill Dams, Nov. 1968, Vol.II, pp.41-45 .
103.Schewe,L.D., "Earth Dam to Augment Athens Water Supply," *Water Power & Dam Construction*, Feb.1977, pp.27-33.
103.a. Schnitter,G.,Prof., "Geotechnical Investigations of Mixtures of Bitumen, Clay or Bentonite with Sandy Gravel," *7^{th} ICOLD Congress*, Rome, 1961, Report 38, Question 27, pp.197-239.
104.Schober,W., et.al., "Load Transfer in Embankment Dams – Model Testing," *Int.Conf. SMFE*, Austria, pp. 973-976.
105.Schober,W., "The Interior Stress Distribution of The Gepatsch Rockfill Dam," *10^{th} ICOLD Congress*, Montreal (Canada), 1970, Q.36, R.10, pp.169-187.
106.Sharma,H.D., "Nonlinear Analyses of a High Rockfill Dam with Earth Core," *A Thesis*, Doctor of Philosophy, Dept Civil Eng., Univ.of Roorke, Roorkee, India, 1976.
107.Sharma,H.D.,et al., "Nonlinear Analyses of a High Rockfill Dam with Earth Core," *13^{th} ICOLD Congress*, New Delhi (India), 1979, Q.48, R.49, pp.879-894.
108.Sherard, et al, "Hydraulic Fracturing in Low Dams of Dipersive Clay," *Proc. Of the Specialty Conference on Performance of Earth and Earth-Supported Structures*, June 1972, Purdue University, Indiana, pp.653-689.
109.Sharma,H.D., "Embankment Dams," Textbook Published by ; Oxford & IBH Publishing Co. PVT.LTD., 1991 (India).
110.Shenouda, W.K., "Quality Control and Testing Methods as Exercised in The Aswan High Dam," *14^{th} ICOLD Congress, Rio De Janeiro (Brasil)*, 1982, Vol. IV, pp.1047-1064.

111.Sherard, J.L.et.al.1984,"Basic Properties of Sand and Gravel Filters," Journ. Geotech.Eng.Proc.ASCE.
112.Sherard, J.L.,"Earthquake Considerations in Earth Dam Design," *Journ. Soil Mechanics and Foundations Division*," Proc.ASCE, July 1967, SM4 (5322), pp.377-401.
113.Sherard, J.L, et.al, "Earth-Rock Dams, Engineering problems of Design and Construction, *text book*, 1963.
114.Sherard, J.L., "Embankment Dam Cracking," *Embankment Dam Engineering*, Casagrande Volume, Joh Wiley and Sons, New York, pp.272-353.
115.Sherard, J.L.et.al.1984,"Filters for Silts and Clays," Journ. Geotech.Eng.Proc.ASCE.
116.Sherard, J.L, "Hydraulic Fracturing in Embankment Dams," *ASCE Journal of Geotechnical Engineering Division*, Oct.1986, Vol.112, GT-10, pp.905-927.
117.Sherard, J.L.,1963,"Sink Holes in Dams of Coarse Broadly Graded Soils," Q.49, R.2, 13th ICOLD Vol.II, New Delhi, pp.25-36.
118.Sherard, J.L, "Trends and Debatable Aspects in Embankment Dam Engineering," *Water Power and Dam Construction*, December 1984.
118a. Shrum,G.M., "The Peace River Project: Portage Mountain Dam," *Journal VBB Consulting Engineers and Architects (Sweden), 1965*, (B.C. Hydro & Power Authority, Vancouver, Canada), pp.216-219.
119.Singh, Alam, "Modern Geotechnical Engineering" *Textbook*, Published by: CBS, 3th Edition, 1989, India.
120.Singh Bharat, and Sharma,H.D, "Earth and Rockfill Dams," *Textbook*, Published by ; Sarita Prakashan, Meerut, Ist Ed., 1976, (India).
121.Singh,Bharat, and Varsney, R.S, "Engineering for Embankment Dams," *Textbook* ,Published by ; Oxford & IBH Publishing Co. PVT.LTD., 1995 (India).
122.Singh,J. (Consultan , Chandigarh), "Salient Construction and Design Fseature of Ranjit Sagar (Thein) Dam Project," *CE & CE March 1994*, India, pp.12-18.
123.Singh,R.P., "Three Dimensional Analysis of Rockfill Dams Under Gravity Loading," *A Thesis of Doctor of Philosophy*, WRDTC, University of Roorkee, Roorkee, India, 1991.
123a. Sinha, " A review of construction methods of High Embankment Dams," *M.E. Dissertation*, WRDTC, Univ.of Roorke, Roorkee, India, 1973.
124.SMFD ASCE, "Stability and Performance of Slopes Embankment," *a Text Book*, Berkeley, California, Aug.,1966.
125.Soil Mechanics Division, "Description and Identification of Soils," Paper, Dept. of Civil Eng. Univ.of Roorkee,Roorkee, 1966.
126.Sowers,G.F, et. Al., "Large Scale Preconstruction Tests of Embankment Materials for Earth-Rockfill Dam," *Proc. 5th Int.Conf. on SMFE*, July 1961, Vol.II, pp.717-720.
127.Squir, L.R., Load Transfer in Earth and Rockfill Dams," *Journal of the SMFE*, Proc. of ASCE, SM.1, Jan.1970, pp.213-233.
128.Sultan,H.A., and Seed,H.B., "Stability of Sloping Core Earth Dams," *Journal of the SMFE*, Proc. of ASCE, SM.4, July, 1967, pp.45-67.
129.Taylor,D.W.,Prof.(U.S.A),et.al.,"Stability and Deformations of Slopes and Earth Dams, Research on Pore-Pressure Measurement, Groundwater Problems," *Proceeding of the 3th Int.Conf On. Soil Mechanic and Foundation Engineering*, Switzerland , Aug. 1953, pp. 209-224.
130.Taylor,H., and Morgan,G.C., "Measures Taken to Limit The Possible Development of Cracks in a High Earthfill Dam," *10th ICOLD Congress*, Montreal (Canada), 1970, Q.36, R.40, pp.679-700.
131.Tehri Hydro Development Corporation Ltd, "*The Project (Brosur & Project Visit 2001)*," printed:Allled Traders, New Delhi (India).
132.Thomas Henry,H., "The Engineering of Large Dam," a Textbook Part II Published by John Wiley & Sons, 1976, pp. 377-455.
133.Tripathy,R.C., "Design of Dams of Clayey Soils Over Clayey Foundation," *M.E. Dissertation*, WRDTC, Univ.of Roorke, Roorkee, India, 1970.
134.Tyagi,M., "Stability Analysis of Earth Core Rockfill Dam," *M.E. Dissertation*, Dept.of Earthquake Eng., Univ.of Roorkee, Roorkee, India, July 1989.

135.Ul-Haq,I., "Foundation Problems of Tarbela Dam Project, "*17th ICOLD Congress,* Vienna (Austria), 1991, Q.66, R.72., pp.1353-1368.
136.University of Roorkee, "Proceeding of Workshop On Tehri Dam Project," *Paper,* Volume: I,II,III &IV, UOR-U.P.Irrig.Dept-U.P.State Elect.Board, 1979.
137.Ural, O.M., "Design and Construction of Large Dams on Difficult Foundation in Turkey," Paper, Director of Civil Engineering Design Section, Dam Division, D.S.I., Ankara.
138.U.S. Dept. of Interior, "Design of Small Dams," *A Text Book,* Bureau of Reclamation, 2nd Ed., 1974.
139.Vallespir,S.J., "Distribution and Treatment of The Core Materials of an Embankment Dam," *14th ICOLD Congress, Rio De Janeiro (Brasil),* 1982, Vol. IV, pp.495-501.
140.Valore,C., "A Tectonized Variegated Clay as Core Material," *17th ICOLD Congress,* Vienna (Austria), 1991, Q.67, R.18, pp.299-317.
141.Varadarajan, A., Prof.(Head, Dept. of Civil Engineerign I.I.T Delhi, New Delhi), "The use of Finite Element Method In Earth and Rock Fill Dam Design," *CE & CE March 1994,* India, pp.20-25.
142.Vaughan,P.R., et.al., "Cracking and Erosion of The Rolled Clay Core of Balderhead Dam and The Remedial Works Adopted for its Repair," *10th ICOLD Congress,* Montreal (Canada), 1970, Q.36, R.5, pp.73-93.
143.Vaughan,P.R., and Soares, H.F. "Design of Filters for Clay Cores of Dams," *Journal Geotechnical Engineering,* Proc. ASCE, GT.1., January 1982, pp.17-30.
144.Vaughan,P.R., et.al., "The Design Construction and Performance of Cow Green Embankment Dam," *U.K. Geotechnque 25,* No.3, December 1975, pp.555-580.
145.Vestad, H., "Viddalsvatn Dam, a History of Leakges and Investigation," *12th ICOLD Congress,* Mexico, 1976, Q.45, R.22, pp.369-389.
146.Vrymoed,J., "Dynamic FEM Model of Oroville Dam," Journ. Geotechnical Engineering, Proc.ASCE, GT8, Aug.1981, pp.1057-1077.
147.Vutsel,I., et al., "Measures Providing Impermeability of Nurek Dam," *11th ICOLD Congress,* Madrid (Spain), 1973, Vol. III, Q.42, R.32, pp.551-562.
147a. T.A. Wafa, "The Aswan High Dam, "*Journal Dams-Planning & Construction,*" 1971, pp.182-186.
148.Walker,F.C., "Design of the Trinity Dam, an Earthfill Structure 537 Feet High," Proc. 5th Int.Conf. on SMFE, July 1961, Vol.II, pp.743-748.
149.Webster, J.L., "Mica Dam Design with Special Attention to Control of Cracking," *10th ICOLD Congress,* Montreal (Canada), 1970, Q.36, R.30, pp.487-509.
150.Wilson,S.D & Marsal, R.J., "Current Trends in Design and Construction of Embankment Dams", *a Text Book,* Prepared for Committee on Int. Relations carlos S.Ospina, Chairman ICOLD and Geotech.Div., ASCE W.F. Swiger, Chairman, published by ASCE, 1979.
151.Wood,D.M, et al., "Thought Concerning The Unusual Behaviour of Hyttejuvet Dam," *12th ICOLD Congress,* Mexico, 1976, Q.45, R.23, pp.391-414.
152.Xinh D.T., , "A Critical Study of The Design and Construction Features of Some Recent High Earth-Rockfill Dams," *M.E. Disst.,* WRDTC, University of Roorkee, Roorkee, India, 1969.
153.Yim Ko. H, and Scot,R.F., "Deformation of Sand in Hydrostatic Compression," *Journal of the SMFE,* Proc. of ASCE, SM.3, May 1967, pp.137-156.
154.Zahaf,H., et.al., "Preparation and Compaction of Impervious Fill On The Bourguiba Dam Project at Sidi Saad," ," *14th ICOLD Congress, Rio De Janeiro (Brasil),* 1982, Vol. IV, pp.433-451.