EARTH AND ROCKFILL DAMS

Principles of design and construction

CHRISTIAN KUTZNER
Consulting Geotechnical Engineer
Honorary Professor, Darmstadt University of Technology, Hofheim, Germany

A.A. BALKEMA / ROTTERDAM / BROOKFIELD /1997

Prof. Dr.-Ing. Christian Kutzner
Adolph-Kolping-Straße 6
D-65719 Hofheim, Germany

Published by
A.A. Balkema, P.O. Box 1675, 3000 BR Rotterdam, Netherlands
Fax: +31.10.4135947; E-mail: balkema@balkema.nl; Internet site: http://www.balkema.nl

A.A. Balkema Publishers, Old Post Road, Brookfield, VT 05036-9704, USA
Fax: 802.276.3837; E-mail: info@ashgate.com

ISBN 90 5410 682 4

Printed in the Netherlands

Contents

Notes:
Slopes are figured out '1V : mH' or '1 : m'. '1' always indicates the vertical component.

If there is no other indication, dam sections show the upstream side on the left.

Foreword

In the course of the past hundred years, modern structures of embankment dams for water reservoirs and hydropower plants emerged from a continually evolving fundamental basis consisting of extensive knowledge about soil and rock mechanics, constructional engineering and calculating procedures. This fundamental basis is treated in the literature by means of specialist articles which reach an immense number of publications.

This book was written to consider and treat all specialist fields and peripheral know-how, and its most important goal has been reached, as this knowledge is an indispensable requirement for each engineer specializing in dam design and construction. It offers the necessary technical basis to both the young student engineer and the experienced expert to pursue further undertakings. It may be emphasized that the topics are described in an easy, comprehensible, and extensive way.

In many regions world-wide, the tremendous increase in population requires further construction of dams in order to safeguard basic living conditions with regard to energy and nutrition. Due to restricted habitat space and scarce resources, nowadays it has become imperative that essential provisions, such as sufficient water supply, shall be guaranteed for the following generations. This leads to the conclusion that dam construction has become indispensable. Existing dams need to be kept safe from the operational point of view.

The German Committee on Large Dams (DTK, 'Deutsches Talsperren-Komitee') being a member of the most significant international dam construction forum, the International Commission on Large Dams (ICOLD), has taken up the task of transmitting to experts from all over the world any discovery, experience and know-how gained by professional German dam engineers. They shall even introduce norms, guidelines and recommendations.

This book achieves this aim in a convincing way. Because of the standards requiring the highest technical levels, it is the conviction of DTK that this book makes a German contribution to international evolution in the dam construction field, since the reader is led through all continents of the world by well portrayed examples of dam projects.

The book reflects the many years of experience, diligence and high professionalism of the author. It remains to say that this English edition may meet with as much response as the German first edition.

Professor Dipl.-Ing. Wolfgang Haug
President of the German Committee on Large Dams
January 1997

Preface

In this book an attempt is made to describe methodically the principles of design and construction of earth and rockfill dams. The problems to be solved and the questions to be answered by those involved are presented in the same order as they will come up in the different phases of work. Starting with preliminary works, proceeding first to the design, then to the construction work and finally to the monitoring of the structure. This method of following up the problems was selected to demonstrate the requirement of the permanent presence and cooperation of the geotechnicians involved – from the preliminary works up to the commissioning date and, if appropriate, later as a consultant for the owner. In no way is it advisable that isolated groups, without close contact with each other, should work on the details of design and construction.

The problems discussed herein reflect the experiences which I was able to collect during 20 years of professional activity in the field of earth and rockfill dams. Notably, the basic knowledge of dam engineering and experiences of other experts are presented as I have deemed necessary and helpful to cover the wide range of geotechnical problems in this field – as I hope, for the benefit of the readers. Thankfully, I acknowledge the fruitful advice given to me by well known experts, and the many professional discussions in the course of national and international symposia and congresses.

Whoever has undertaken the task of compiling a book will know that expert knowledge is not all that is required to complete it. In this aspect I recall with great thanks the help and support of my wife who has, over the years, borne with me the burden of excess work and restricted family life. She is also due credit for having the book completed. My thanks to other colleagues, assistants, institutions and companies contributing to the work are expressed in the acknowledgements.

This book is the revised English version of my book *Erd- und Steinschütt-dämme für Stauanlagen*, which was published in German by Enke Verlag Stuttgart in 1996. The user will not expect to find comprehensive answers to all questions in this wide field. I have tried to address the main problems and the different approaches to their solution which have developed in different countries. I am confident the large number of references will help to show the way through details which may need further clarification.

It is, finally, my wish that my colleagues will accept the book as an aid in their work. Beginners may draw from it basic knowledge of the geotechnical problems of dam engineering; experienced colleagues may more easily find a suitable approach to questions whenever such an approach is not readily present.

Christian Kutzner
November 1996

Acknowledgements

The author acknowledges with thanks the support and cooperation of experts in the profession, of colleagues, institutions and companies who gave most valuable help in contributing to the text of the book and in giving additional information for the readers.

These are the following personalities:

Prof. Dipl.-Ing. Wolfgang Haug, President of the German Committee on Large Dams, with the Foreword.

Mrs Anka Both from Lahmeyer International, who drafted a part of the diagrams and drawings.

Dr.-Ing. Wolf-Dieter Geiseler, who reviewed the sections on asphaltic concrete and who gave valuable information by verbal and written communication.

Dr.-Ing. Klaus Hönisch with his review of Chapter 8 and related suggestions.

Prof. Dr.-Ing. Theodor Strobl for written communication.

The following institutions and companies submitted photographs, in this way helping to realize the pictorial part of the book:

Bauer Spezialtiefbau GmbH, Schrobenhausen, Germany.

Dynapac Maskin AB, Solna, Sweden.

Glötzl Baumesstechnik, Forchheim-Bahnhof, Germany.

IBF, Institut für Bodenmechanik und Felsmechanik, Universität Karlsruhe, Germany.

Interfels GmbH, Bad Bentheim, Germany.

LI, Lahmeyer International GmbH, Consulting Engineers, Frankfurt am Main, Germany.

Maihak AG, Hamburg, Germany.

Strabag Bau-AG, Köln, Germany.

Trischler & Partner GmbH, Darmstadt, Germany.

Troxler Electronics GmbH, Alling, Germany.

Lahmeyer International and their clients have enabled the author to improve his knowledge of dam engineering, to apply it in practice, and finally to transfer the know-how to others. Their noble assistance in making documents and literature available has been invaluable help.

CHAPTER 1

Introduction

The design and construction of earth and rockfill dams always result in a unique structure. Necessarily, dam engineering – like any other engineering discipline – has to follow basic rules which can be noticed in all structures as design criteria or as constructional peculiarities. Such criteria and peculiarities are discussed in this book. We will notice that the variety of natural construction materials and of potential dam locations demands that designers and builders show extreme scientific and practical effort. It is a challenge for flexibility and innovation.

Such effort can only be successful on the base of proven engineering procedures. Starting design work without respect for known geotechnical rules would lead to extended working time and would involve risks for the safety of the structure.

Nowadays no engineer would like to follow exclusively the methods of 'trial and error' and 'learning by doing'. On the contrary, there is a tendency to put too much confidence in the result of computations, thereby underestimating the imponderabilities of nature. Such imponderabilities exist not only due to the limitation and incompleteness of all investigations, but also due to the facts that the original conditions at the dam location are strongly affected by the weight and the size of the structure and that the hydraulic conditions around the dam are changed to a great extent by impounding the reservoir.

The best way of developing a structure which is safe and functional over its life, will be found between the two extremes 'trial and error' and 'computation only'. This way has its individual character for each project, depending on the complexity of the construction materials and on the special conditions of the location.

Responsible engineers have to follow this way up to the start of operating the project. Design and construction cover the first part of this way. At the end of the design phase the responsibility cannot be passed over to those who have to care for the realization. Other than with structural engineering the construction phase is not mainly a matter of logistics. Instead, we must always ensure that substantial aspects, such as foundation conditions and material properties, still comply with the presuppositions of the design.

As a consequence for advanced dam engineering the following is stated (Fetzer 1988b): 'Uncertainties in the foundation and abutment conditions and in borrow materials have led to the conclusion that the design must continue through construction. Many engineering organizations schedule field trips for their design geologists and engineers at critical stages of construction to determine if field conditions are the same as those assumed during design...'. The author would like to stress that each engineering organization must schedule such field trips for the team of designers not only at critical stages but also periodically to make sure that the field conditions at any stage comply with the design assumptions.

This means in practice that the designer has to supervise the construction. Designer and supervisor in cooperation are called 'the engineer'. The continuous responsibility of this engineer should again and again be stressed, as it is – fortunately – in recent literature (e.g. Leps 1988a). Incomplete continuity in this aspect was discovered during the investigations on the failure of the Teton dam, and it was found to be one of the reasons for failure.

The second part of the way from design to routine operation covers the phases of the first impounding and of the first period of operation. Essential supervision of these phases by the engineer cannot be expected. Responsibility is taken over by the owner who has to be provided by the engineer with a complete documentation of the results of investigations, of the design criteria and of experiences collected during construction.

The first impounding must prove the correctness of earlier made predictions concerning not only the safe function of the structure, but also the effect of the structure on the foundation and the effect of the new hydraulic conditions on the environment. There are two contrasting opinions on the mode of first impounding.

On an international level it is recommended 'the designer should fully consider that the reservoir may fill very quickly, regardless of the generally assumed merit of controlling the filling rate' (Leps 1988a). This recommendation is made 'because of inevitable hydraulic uncertainties, and difficulties in forecasting precisely when critical water control outlets may be completed and serviceable'.

In Germany, the standards request strict control of impounding. According to DIN 19 700 the serviceability of the water control outlets is a precondition for the start of impounding. The rise of the water level is controlled so as to follow stepwise a previously established filling programme. The next step of filling is permitted only when the previous step did not show any critical condition. Such procedure is reasonable under the hydrologic conditions prevailing in Central Europe. It is justified with respect to the safety requirements of a densely populated and highly industrialized country.

Readers of this book or of the table of contents only will find the book compiled according to the common sequence of work: project development under consideration of neighbouring disciplines, investigations of the subsoil and of the construction materials, design and – finally – construction and its supervision. The dam instrumentation with a measuring programme for safety control and the above mentioned complete documentation are the link between the engineer and the owner.

Once the reservoir operation has started and instrument readings are made according to the measuring programme, and once the documentation is completed, the work of the geotechnicians involved is finished. Fortunately, their stock of knowledge will in most cases be greater than before the project had commenced.

CHAPTER 2

History

Building dams is one of the oldest technical activities applied for the benefit of large human groups. Dams made of soil and rock have been known since the 3rd millennium before Christ. Schnitter (1987) nominates not less than 34 dam structures existing at the beginning of our chronology in all civilizations of ancient times. The largest of these was 30 m in height. Even in our day this would be defined as a 'large dam'. At the end of the 16th century the list includes about 300 structures, serving almost exclusively for water storage. Of course, land irrigation, water supply and flood control have been the prevailing functions.

Sadd-el-Kafara near Cairo in Egypt, a 14 m high rockfill dam with earth core, was one of the world's oldest dams, being constructed around 2600 years BC (Fig. 2.1). The dam was designed for flood control. Studies on the history of this structure have been summarized by Garbrecht (1987) as follows (extract):

'...The dam was stable in the sense of recent considerations; it would have been able to resist expected overtoppings. During construction an extreme flood overtopped and destroyed the existing parts of the dam. ...It was a tragedy for the Egyptian engineers to see this unexpected flood leading to a disaster. We owe respect to them for taking on the challenge of starting dam construction given the limited technical potential of their time. They earn our compassion for failing due to an unforeseeable event'.

Another important structure was the earthfill dam Marib, 7 m in height, in North Yemen close to the city of Marib. The dam served for land irrigation. It was constructed in 750 BC. It had been in use over more than 1000 years when it was destroyed in the 2nd half of the 6th century. Its overtopping is mentioned in the 34th Sura of the Koran as a punishment of God to the people who had abjured their faith.

This dam was part of an irrigation system for an area of about 1600 hectares of agricultural land to provide food for about 300,000 people, at this time and in this region of the Queen of Saba (Jenner 1983). A new dam was equip all dam structures with suitable instruments and to evaluate the easured values continuously.

The present state of the art allows us to construct large embankment dams vithin a reasonable time and at justifiable costs. This is due to the high caacity of all earth moving equipment and of compacting machines giving a naximum of density and hence shear strength to all relevant construction naterials. The typical view of a modern dam site is marked by a fleet of self-moving machines being operated by a minimum of people (Fig. 2.3).

Necessarily, all periods of dam engineering had to cope with adverse events. Examples are Sadd-el-Kafara in ancient times and the Oker-dam in Germany in 1714. The dam failed during construction due to an inadequately steep slope of the upstream portion of masonry. At this time Coulomb's theory on earth pressure on retaining walls (1786) had not come to the knowledge of the engineers. As well, our time is not proof against failure. In 1976 the 92 m high Teton earth embankment dam in Idaho, USA failed at the first reservoir impounding. Eleven people died; the damage to property amounted to almost $500 million. According to expert reports (Penman 1977, Londe 1983) failure was caused partly by unreasonable design and construction details ignoring the relevant state of the art. Also human insufficiency came into play.

Apart from such negative experience it may be noted that expert knowledge enables us now to design, to construct and to operate large embank-

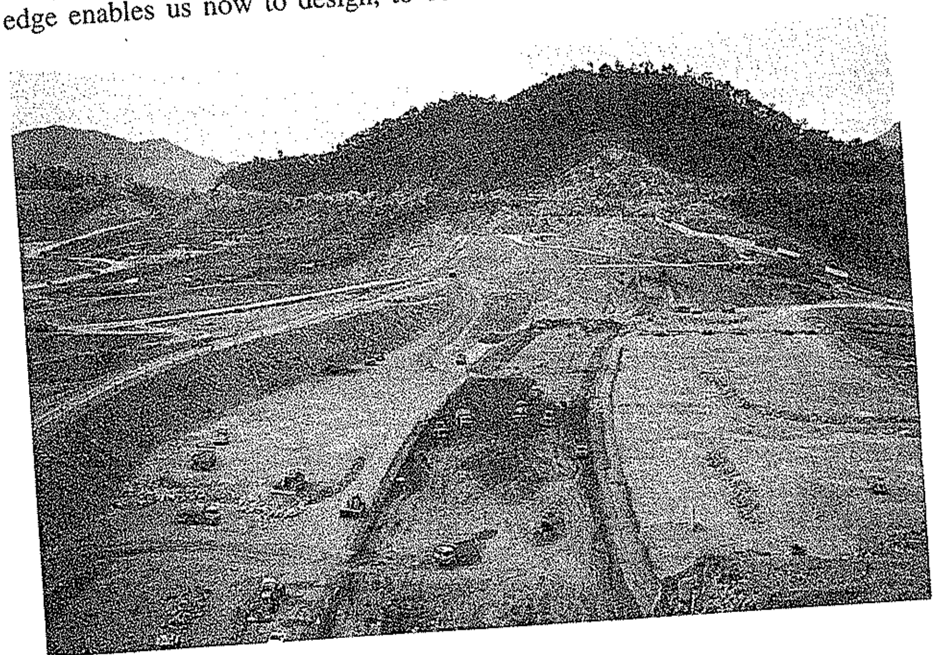


Figure 2.3. Kinda dam, Burma 1983 (courtesy of LI).

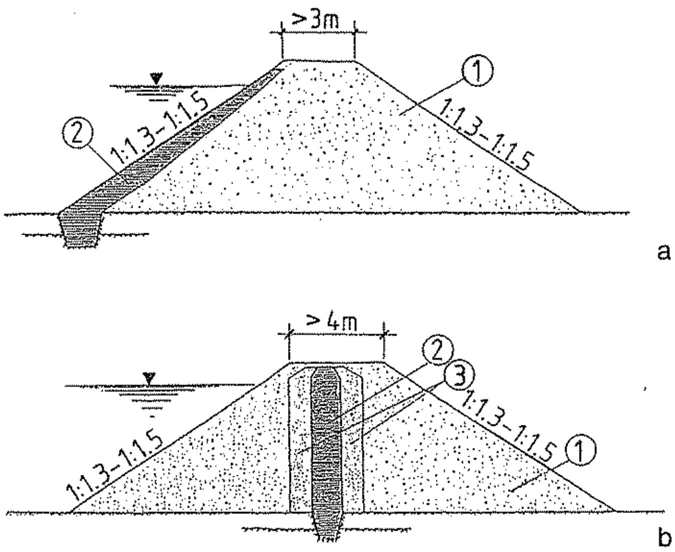


Figure 2.2. Dams of the mining industries, Harz mountains, Germany, 18th century AD (adapted from Schmidt 1989).

a Old method of construction
b New method of construction
1 Embankment material
2 Sealing material, sods
3 Transition zone of fines

– non-linear deformation behaviour of soils in relation to triaxial state of stress and strain.

Scientific research in this respect resulted in increasing confidence of the profession and of the public in dam engineering and its evolution after the 2nd World War. Considerations of geotechnicians are now mainly focused on erosion stability, on dam deformation during construction and reservoir operation and on safety against cracking, in addition to static and dynamic stability.

Stress and strain considerations have been improved essentially by the development of static and dynamic computations based on finite elements, with respect to the non-linear behaviour of earth and rock materials. Static methods cover all conventional load cases, while dynamic methods simulate the effect of earthquake loads. Such computations have been developed mainly in the USA. Comprehensive comments on them are given by Duncan et al. (1980) and Seed (1979). Unfortunately, it is quite expensive to make full use of such methods. In parallel, measuring instruments have been developed to control stresses and strains in existing dams. Today it is common to equip all dam structures with suitable instruments and to evaluate the measured values continuously.

The present state of the art allows us to construct large embankment dams within a reasonable time and at justifiable costs. This is due to the high capacity of all earth moving equipment and of compacting machines giving a maximum of density and hence shear strength to all relevant construction materials. The typical view of a modern dam site is marked by a fleet of self-moving machines being operated by a minimum of people (Fig. 2.3).

Necessarily, all periods of dam engineering had to cope with adverse events. Examples are Sadd-el-Kafara in ancient times and the Oker-dam in Germany in 1714. The dam failed during construction due to an inadequately steep slope of the upstream portion of masonry. At this time Coulomb's theory on earth pressure on retaining walls (1786) had not come to the knowledge of the engineers. As well, our time is not proof against failure. In 1976 the 92 m high Teton earth embankment dam in Idaho, USA failed at the first reservoir impounding. Eleven people died; the damage to property amounted to almost $500 million. According to expert reports (Penman 1977, Londe 1983) failure was caused partly by unreasonable design and construction details ignoring the relevant state of the art. Also human insufficiency came into play.

Apart from such negative experience it may be noted that expert knowledge enables us now to design, to construct and to operate large embank-

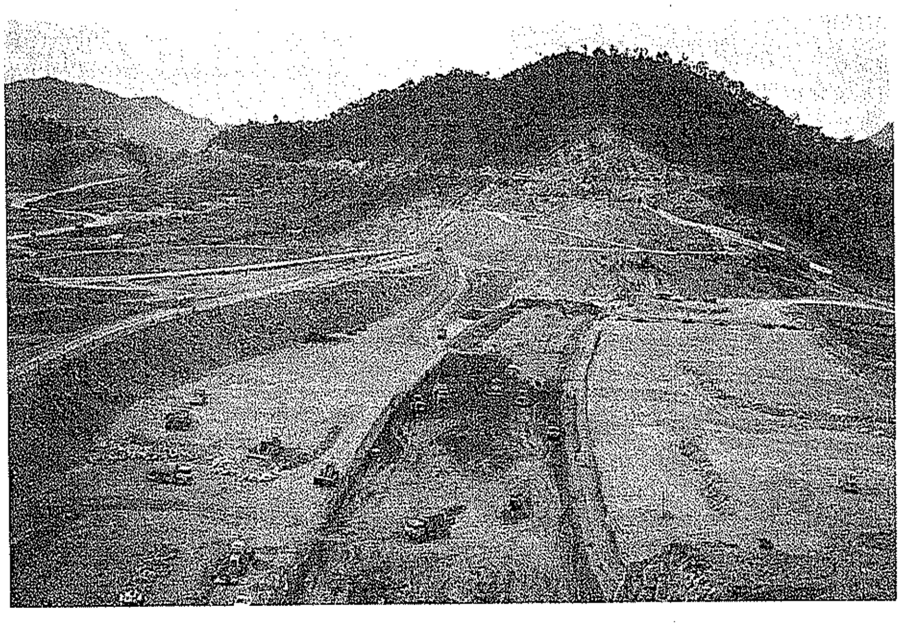


Figure 2.3. Kinda dam, Burma 1983 (courtesy of LI).

ment dams and their reservoirs at a high level of safety. This applies to geotechnicians who responsibly apply relevant knowledge and who are cautious where no experience exists. It is prudent to be cautious about designing even higher dams than before, in constructing even more rapidly than before and in realizing the projects at lower costs than before. Progress in these directions is justified, but only small steps should be made.

One institution must be mentioned specifically in view of the high standard of dam engineering. It is the International Committee on Large Dams (ICOLD) with its 78 member countries. The committee calls experts from all over the world to make their relevant experience public in an international congress, held every three years. Each congress is devoted to four selected questions dealing with all types of dams and appurtenant structures. Fifteen questions related to embankment dams have been dealt with in the last six congresses in the period from 1979 to 1994. All contributions and a General Report on them are published in four volumes of proceedings, followed by a fifth volume with the discussions held during the congress.

By this activity the relevant state of the art is constituted permanently on an international base – to the advantage of those geotechnicians involved in dam engineering and to the public in general, which benefits from the operation of dams and reservoirs.

CHAPTER 3

Project development

3.1 OVERVIEW

All projects have to cope with regional and local conditions. For dam construction mainly the following conditions have to be respected:

- Geology,
- morphology and topography,
- hydrology.

At the beginning comprehensive studies and investigations are needed under these headings, which commonly last for several years.

As is known, dam incidents cause human disasters and economic setbacks to a great extent. Statistical investigations of dam failures have been made evaluating the origin of such incidents. A remarkable number of failures was caused by insufficient investigations and by misinterpretation of geological and hydrological conditions. This is confirmed by the last statistics of ICOLD (1974) shown in Table 3.1. Probably, a considerable number of incidents summarized under 'design' is caused also by misinterpretation of geological or soil mechanical investigations. This applies also to projects in karstic areas which failed to function for long time or even permanently.

3.2 GEOLOGY

The geological conditions at a particular site make a project feasible or not: the foundation conditions must be adequate and the major part of the required construction materials should be available within a short distance. Without these preconditions the site may be unfeasible.

All seismic aspects are usually attributed to geology. The seismic conditions have to be studied by region and area (not just locally) to enable the definition of the operational basis earthquake (OBE) and the maximum credible earthquake (MCE) which are appropriate for the site. Respective

Table 3.1. Number of dam incidents (adapted from ICOLD 1974).

Fundamental cause	Earth dams	Rockfill dams	Concrete dams	Miscella-neous	Total
Exploration	49	2	20	1	72
Material	8	–	3	–	11
Layout	17	3	5	–	25
Design	48	3	23	2	76
Construction	32	5	4	–	41
Operation	5	1	–	–	6
Supervision	3	–	2	–	5
Total	162	14	57	3	236
Percentage:					
– Exploration and material	35		40		35
– Design and construction	49		47		50

studies should cover past times, as far as possible, since earthquake events of the past will give valuable information. The seismic conditions may also render a project unfeasible, for instance, if a geological fault of unknown seismic activity constitutes a safety risk for the dam or one of the appurtenant structures. An example is discussed in Section 3.5.

Rock and soils are the main sources of construction materials. Soils are defined according to known classification systems. For rock no unified classification system is available. Many experts have worked on such systems which are used mainly in underground and slope engineering. For dam engineering such a system is dispensable. In practice it proved to be appropriate using weathering grades as a classification, according to Table 3.2 (or equivalent), in combination with drill protocols and the results of field and laboratory tests. Another classification is related to the strength, the pre-failure and failure characteristics, the gross homogeneity and the continuity in formation (Thomas 1976).

3.3 MORPHOLOGY AND TOPOGRAPHY

The morphology, i.e. the form and structure of the surface of the earth, is another criterion to select a dam site. For embankment dams the criterion is restricted to finding the most appropriate location among several choices. This is because embankment dams do not demand special conditions of abutment stability or of valley size as, in contrast, arch dams do.

The topography of the dam site and of the reservoir area is a matter of geodetic survey. The survey is made in parallel to the other geotechnical in-

Table 3.2. Rock classification system due to weathering (Geological Society Engineering Group Working Party 1972). Reproduced by permission of the Geological Society, London.

Term	Grade symbol	Diagnostic features
Fresh	W I	Parent rock showing no discolouration, loss of strength or any other weathering effects
Slightly weathered	W II	Rock may be slightly discoloured, particularly adjacent to discontinuities, which may be open and will have slightly discoloured surfaces; the intact rock is not noticeably weaker than the fresh rock.
Moderately weathered	W III	Rock is discoloured; discontinuities may be open and will have discoloured surfaces with alteration starting to penetrate inwards; intact rock is noticeably weaker, as is determined in the field, than the fresh rock. (*The ratio of original rock to weathered rock should be estimated where possible.*)
Highly weathered	W IV	Rock is discoloured; discontinuities may be open and will have discoloured surfaces, and the original fabric of the rock near to the discontinuities may be altered; alteration penetrates deeply inwards, but corestones are still present. (*The ratio of original rock to weathered rock should be estimated where possible.*)
Completely weathered	W V	Rock is discoloured and changed to a soil but original fabric is mainly preserved. There may be occasional small corestones. The properties of the soil depend in part on the nature of the parent rock.
Residual soil	W VI	Rock is discoloured and completely changed to a soil in which original rock fabric is completely destroyed. There is a large change in volume. (*Genesis should be determined where possible.*)

vestigations described in Section 4. The topography of the area in question should be surveyed at the latest when the most appropriate site is to be selected. The topography is the base of the reservoir filling curve.

In developing countries the survey is commonly made by aerial photogrammetry, one to fifty thousand or similar in scale. Under conditions of dense vegetation a tolerance up to 10 m in height has to be taken into account. A more accurate surface survey is often made only at the time when the dam location has been selected. After removal of all vegetation a small change of the previously selected dam location may then be advisable.

Once the reservoir is being impounded the filling curve may prove to be inaccurate, according to the tolerances in height. Then, certain storage levels will be reached later than expected. One has to live with such uncertainties.

Reservoir filling is related also to the saturation of the surrounding rock. This contributes to the inaccuracy of the reservoir filling curve.

3.4 HYDROLOGY

Hydrological aspects are dealt with here only as far as required for better understanding. Hydrology is related to dam construction and reservoir operation by the need to divert rivers during dam construction and by flood control. The forecasting of hydrological events is necessarily based on the available data from previous times. Correct prediction benefits from the length of the period which is covered by accurate and reliable data. In remote areas such periods may be short. This, commonly, results in generous dimensions of the structures for flood control including emergency spillways to exclude dam overtopping.

Common criteria for spillway design are the probable maximum flood (PMF) and the reservoir retention capacity. In some projects PMF is replaced by the 10,000-year flood. For PMF an extreme range of floods to be controlled is existing, e.g. 100 m^3/s at the reservoir Prims and almost 20,000 m^3/s at Atatürk. This demonstrates the importance of correct data and data processing since the size and the construction and operation costs of the spillway depend directly on the hydrological data.

The structures built in diverting rivers are designed according to hydrological data, but not exclusively. Additional criteria are the length of the construction period and the acceptance of a remaining risk of inundating the construction site. Again, there is a wide range of construction flood events to be controlled, e.g. about 20 m^3/s for Prims and 8000 m^3/s for Atatürk. The design will be different for diversion structures than for structures for permanent use.

3.5 SELECTION OF THE DAM LOCATION

The dam location is selected as soon as morphological and topographical studies permit and as soon as geological investigations with respect to bearing capacity, permeability and seismic activity have confirmed the suitability of the prospective site. That means, selection is made in the course of the field investigations described in Section 4.2. The geological, morphological and topographical conditions must be considered. Favourable conditions are:

Geology

- Adequate bearing capacity of the foundation,
- low permeability of the foundation,
- no existing geological faults,

– no risk of seismic activity.

Morphology

– Smooth and symmetrical valley with gentle slopes,

– exposed flanks forming an arch of dam and abutments.

Topography

– High abutments, well above normal pool level,

– high flanks around the reservoir with long seepage path to neighbouring valleys,

– no depressions requiring lateral dams.

Examples of favourable conditions are shown in Figures 3.1 and 3.2. The more the conditions differ from such an 'ideal' state the more costly is project realization.

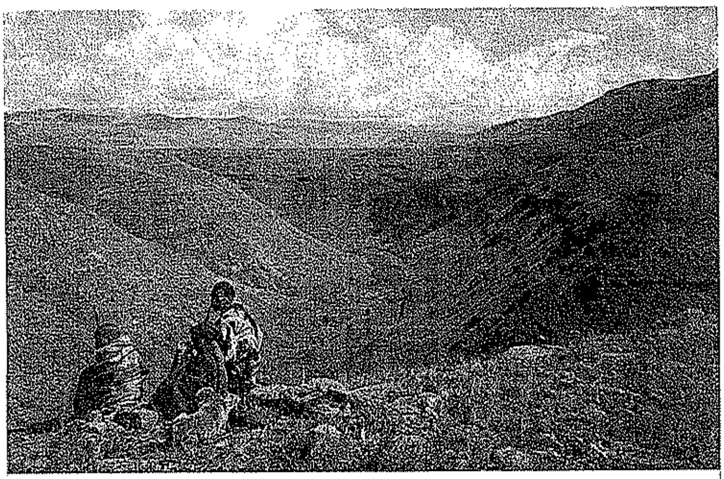


Figure 3.1. Well shaped valley, almost symmetrical, Lesotho. Topographically ideal conditions for a 200 m high embankment dam.

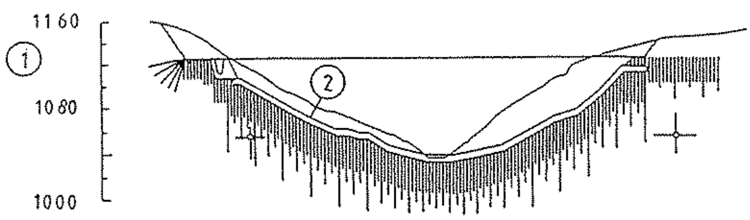


Figure 3.2. Inamura dam, Japan 1982. Well shaped core foundation after deep excavation of soil and weathered rock (adapted from Nakayama et al. 1982).

1 Elevation (m a.s.l.)

2 Inspection gallery, constructed in open trench

As is known, engineers are challenged nowadays to cope with unfavourable conditions since the most favourable sites are occupied by existing dams. Such unfavourable conditions, but still under control, are shown in Figures 3.3 and 3.4:

Geology

– Thick layers of unconsolidated and permeable soil above the bedrock, in many cases requiring deep excavation,

– extremely permeable rock, e.g. karst,

– geological fault in the foundation area (Fig. 3.3). (Faults of seismic activity are excluded.)

Morphology

– Deeply eroded valley with deep excavation (Fig. 3.3) or comprehensive sealing measures (Fig. 3.4),

– irregular contour of the rock surface and extremely steep slopes causing large differential settlements in the dam body (Figs 3.3 and 3.4).

Topography

– Abutment as in Figure 3.3 (right) with lateral extension of the dam (or reduced dam height),

– depressions along the reservoir rim requiring additional dams.

Necessarily, construction costs increase with the length of the structure. In principle, however, there is no limiting length. Large dams have been constructed which are some kilometers in length.

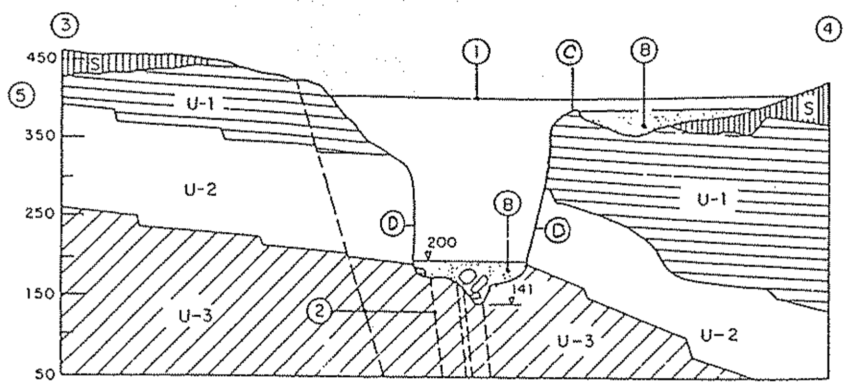


Figure 3.3. Chicoasen dam, valley section (adapted from Moreno & Alberro 1982).

1 Dam crest 402 m a.s.l.
2 Geological faults, parallel to the valley
3 Left abutment
4 Right abutment
5 Elevation (m a.s.l.)
S Clay shales

U-1 Limestone interbedded with clay shales
U-2 Massive limestone
U-3 Limestone interbedded with clay shales
B River deposits with boulders
C Irregular rock surface
D Extremely steep slopes

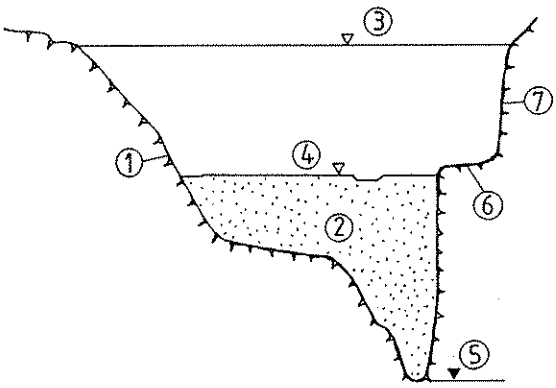


Figure 3.4. Sylvenstein dam, valley section (adapted from Lorenz 1966).

1 Rock: Dolomite
2 Alluvions: Sandy gravel, erratic blocks, mud layers
3 Dam crest 766 m a.s.l., length 175 m
4 Valley bottom 725 m a.s.l.
5 Erosion channel 626.7 m a.s.l.
6 Irregular rock surface
7 Extremely steep slope

Kinda dam is an example of how to select the final dam location. This example demonstrates the required evaluation and quantification of advantages and disadvantages given by the geological, morphological and topographical conditions (Fig. 3.5).

An optimum storage level at 200 m a.s.l. had been identified according to hydrological and topographical conditions. An early study of the project had dam location A in mind, leading to a length of the structure of only about 270 m. At this site the left abutment consists of compact limestone. The right abutment is made of porous Travertin limestone of high and very irregular permeability. A favourable place for the spillway was seen in the depression north-west of the dam. This depression is cut by a geological discontinuity of unidentified seismic activity, named the Kinda-fault.

The area west of the Kinda-fault presents better geological conditions due to the quartzite there. However, a dam at location B would require a length of 630 m to bridge the valley. In the course of studies a connection of the travertine zone with a remote karstic area was discovered by tracer tests resulting in increased risk for impermeabilization of the right abutment at location A. Therefore, it was decided to place the dam at location B with better geological but less favourable topographical conditions. Any effect of the Kinda-fault was deemed to be under control, now running parallel to the center line of the dam at a distance of about 470 m.

Later on the risk of seismic activity of this fault was identified in more detail: the fault was cut 7 m deep by the excavation for the eastern part of the diversion system (no. 6 in Fig. 4.1). The alluvions there did not show any

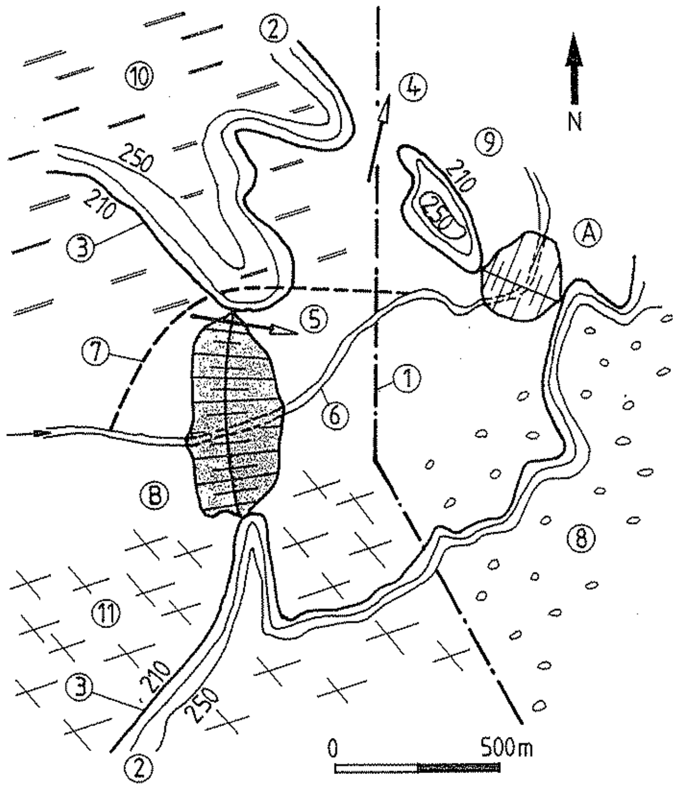


Figure 3.5. Kinda project 1977, selection of dam location.

A Dam location A (previous study)
B Dam location B (final study)
1 'Kinda' geological fault
2 Contour lines (m a.s.l.)
3 Full supply level 210 m a.s.l.
4 Spillway location A
5 Spillway location B
6 River channel
7 River diversion
8 Karstic limestone (travertine)
9 Compact limestone
10 Quartzite, in part slaty
11 Quartzite, slightly jointed

sign of shear movement. This is considered as an indication that there was no seismic activity in the last 10,000 years.

The main structures are concentrated at the left abutment. An economic design was found using the diversion tunnel later on to conduct water to the turbines for power generation and to the downstream regulating reservoir for irrigation. The two entrances for diversion and power/irrigation can be seen in Figure 4.3 (bottom).

CHAPTER 4

Investigations of the substrata and the natural construction materials

4.1 GENERAL CONSIDERATIONS

After description of the studies needed for the project's development, now those investigations are discussed which are required for the elaboration of a tender design. The first studies cover a large area, while the investigations in question are more concentrated on the area where the structures are located and on the close vicinity where the construction materials should be found. The schedules for studies and investigations may overlap each other.

It is presumed here that the relevant hydrological data have previously been evaluated with respect to river diversion, flood control and dam height. Usually, the selection of the dam location is made in an early phase of the investigations. This was discussed at the end of Chapter 3. What remains now is the task of selecting the type of dam. The selection is made as soon as sufficient information on foundation and material conditions is available. This will be discussed at the end of this chapter. DIN 4020 may serve as a guide for the investigations.

Investigating programmes, prior to the works, are planned and scheduled tentatively. The results of the investigations accumulate with the progress of the works. Progressively emerging results may affect the remaining works. It is, therefore, necessary to keep the programmes flexible and to have enough money budgeted from the beginning. Usually, the costs of substrata and material investigations amount to 2 to 3% of the construction costs.

The main investigatory works are listed in Table 4.1. It is up to the designer to compile the work schedule so as to have the results available in due time. For this goal, the duration of each part of the works must be estimated. In industrialized countries it is common practice to contract tight work schedules. Drilling companies will be flexible enough to adjust their staff and equipment to the high production rate. As an example: the monthly drilling rate of one core drilling rig will be in the order of 300 to 500 drill meters. Such rates can, as a rule, not be expected in remote areas of develop-

Table 4.1. Investigations of the substrata and the natural construction materials.

Type of investigations	Result	Sampling	Field tests
Geological mapping	General overview, identification of material deposits	–	–
Core drilling	Stratification of soils and rock	Rock and soil samples for lab tests	Water pressure tests and test grouting
Penetration tests	Stratification of soils, identification of material deposits	–	–
Test pits and test trenches	Stratification of soils, identification of material deposits	Undisturbed and disturbed soil samples for lab tests	Moisture content, moist unit weight, gradation
Adits and shafts	Rock conditions	Rock samples for lab tests	Rock mechanical tests
Geophysical tests (calibration by core drillings required)	Stratification, thickness of overburden	–	–
Large scale tests, desirable prior to the elaboration of tender documents			Blasting test, compaction tests, grout test

ing countries. The monthly drilling rate of a core drilling rig might not exceed 30 to 50 drill meters, because of idle time waiting for spare parts, drill bits and the like. Also, bureaucratic objections like customs clearance, etc. have to be considered.

Usually, investigating takes 2 to 4 years. Table 4.2 gives an example of an investigating programme for the tender design of a hydropower project in a developing country, consisting of a 100 m high rockfill dam and appurtenant structures. This programme is typical for such a project, with favourable conditions of foundations and construction materials provided. A similar programme was performed for the Kinda hydropower project in Burma (Fig. 4.1). Drillings in the quarry area for rockfill material are not shown.

Note: the curvature of the center line of the dam to be seen in Figure 2.3 was established in a later phase of the design work.

The following should be kept in mind: investigating the substrata and construction materials will not be finished when construction starts. The investigations required for the project development, its design and its con-

Table 4.2. Example of field investigations for a rockfill dam with earth core at favourable geological conditions.

Item	Work to be done	Responsible reporter	Period of performance (months) Beginning	End
1	Site mobilization with two rigs, equipment for borehole tests, workshop, site office, housing, all accessories	Geologist, engineer	Beginning of 1	End of 3
2	Preparation of access to 30 to 35 drill hole locations and 20 to 30 test trench locations	Geologist	Beginning of 4	End of 5
3	Geological mapping	Geologist	Beginning of 5	End of 7
4	Identification of material deposits in close vicinity	Geologist, engineer	Middle of 7	Middle of 9
5	200 m of core drilling in mapped area of quarries, no tests	Geologist, (engineer)	Beginning of 6 Drill rig A	End of 7 100 m per month
6	1400 m of core drilling in 20 to 25 boreholes with complete water pressure testing. The location of boreholes covers the area across the valley and about 300 m d/s and 300 m u/s of the dam	Geologist	Beginning of 6 Drill rig B Beginning of 16 Drill rig A	End of 19 80 m per month End of 19 80 m per month
7	400 m of core drilling in 6 selected boreholes with complete water pressure testing and test grouting	Geologist	Middle of 8 Drill rig A	Middle of 15 60 m per month
8	Excavation of 20 to 30 trenches in mapped borrow areas for core material, filter and concrete aggregates incl. necessary auger drilling and penetration testing	Geologist, engineer	Middle of 8	Middle of 12
9	Soil sampling from all material deposits	Geologist, engineer	Middle of 10	Middle of 13
10	Drafting of complete reports on items 3 through 9	Geologist, engineer	Beginning of 19	End of 20
11	Geodetical survey of all borehole and trench locations	Surveyor	Middle of 19	End of 20
12	Wrapping and shipping of rock and soil samples:	Engineer		
	– to a local laboratory,		Middle of 13	Middle of 14
	– to a laboratory abroad for special testing		Middle of 13	Middle of 16
13	Laboratory testing and reporting:	Engineer		
	– local laboratory,		Beginning of 15	End of 20
	– laboratory abroad		Beginning of 17	End of 20
14	Period to cover delays and unforeseen works		Beginning of 21	End of 24

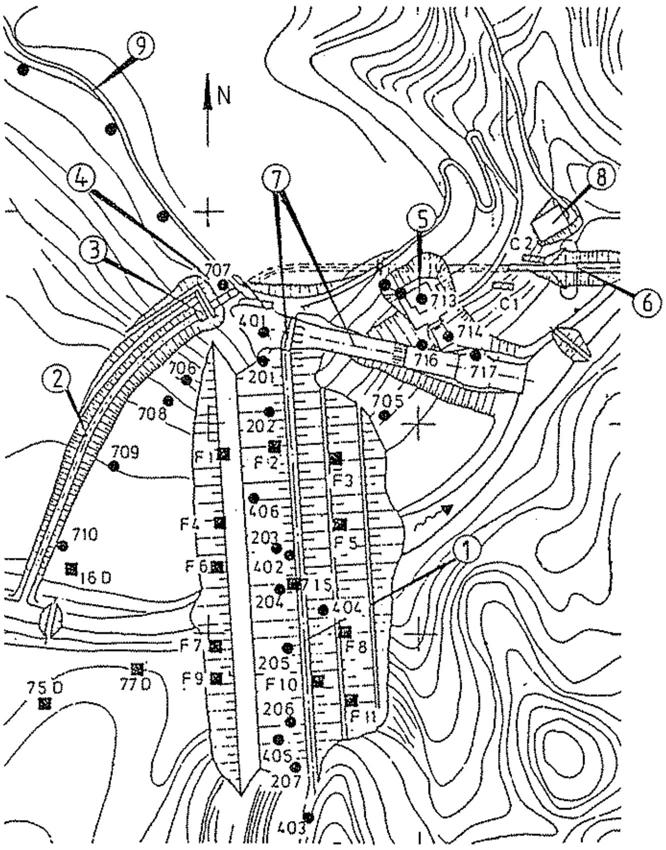


Figure 4.1. Kinda dam. Investigations of the substrata 1977 to 1980.

1 Main dam, early design phase
2 Diversion channel
3 Intake of diversion tunnel
4 Intake of power tunnel
5 Power house
6 End of diversion tunnel and low level outlet, beginning of irrigation channel
7 Spillway
8 Switch yard
9 Access road
Dots: Core drillings
Squares: Test pits and trenches
C1, C2: Adits

struction will in most cases overlap each other. Experienced designers know the unpleasant situation that parameters required for the design work are not available in time. Then, work has to be continued using estimated data. The main reasons for such a situation are:

– Delay in field and laboratory work,

– misleading interpretation or missing special investigations, with the subsequent need to fill the gap of information,

– unexpected results requiring additional investigations.

All the parties involved should realize that investigating programmes do not run automatically according to a given scheme. In contrast, they have to be adjusted to the actual conditions found in the course of the work. This claims for good cooperation of the parties from the beginning, namely employer, geologist, materials engineer, laboratory, design office and contractor.

4.2 FIELD INVESTIGATIONS

4.2.1 *Geological and geotechnical investigations*

4.2.1.1 *Regional and local geology*

The report on the regional geology must describe the most important geological structures of the region and their effect on the project. These are e.g. the location, strike and dip of faults which border different geological structures and which are elements of seismic activity. The report must consider landslides and land subsidence of the geological past and the risk of such events in future, under the conditions of reservoir operation.

The regional report covers the whole reservoir area. It must consider reservoir tightness and slope stability before and after impounding. Both items may require special investigations, such as tracer tests for tightness and drill holes for slope stability. In this respect regional and local geology overlap each other.

Investigations over a wide region are particularly required in karstic areas. Impervious layers may cover karstic formations, giving the impression that the reservoir is tight. Such layers may be destroyed and pressed into karstic caves by the hydrostatic pressure of the water in the reservoir, thus diminishing the tightness of the reservoir. Undetected connections may exist between the reservoir and karstic formations outside. There are examples of both problems, leading to excessive costs beyond the budget or to an entirely abandoned project. Karst investigations must include the ground water regime.

The report on the local geology is the continuation and detailing of the regional report. It is usually restricted to the area where the structures are located and to the material borrow areas. It describes and evaluates mainly the nature of the existing rock and soil and their effect on the design, construction and operation of the project. Thus, the work on the local geology demands more geotechnical experience on the part of the reporter than the work on the regional geology does.

The local geological report must include:
– Description of rock types and soils,
– geotechnical surface mapping,
– mapping of joints and faults,
– graphical presentation of the strike and dip of all discontinuities,
– evaluation of the risk of landslides,
– borehole and test pit profiles,
– a map showing the location of all boreholes, pits and other geotechnical investigations.

The strike and dip of discontinuities are shown in the polar stereographical net and the frequency diagram (Fig. 4.2). This graphical presentation should be known by the reader. Details may be taken e.g. from Müller (1963), Hoek & Brown (1980), Kutzner (1996). The diagrams inform us on the stability of existing geological structures. Furthermore, they point to the risks emerging from excavation and slope modelling, and point to the required support measures. The examples of Figure 4.3 demonstrate that the designer must know whether and where he has to respect sliding wedges or other sliding masses. The examples demonstrate also the need for continual geological supervision during the construction period.

The geological survey is, simultaneously, a precondition for elaborating investigatory programmes. As an example: the depth and direction of boreholes do not depend only on the structure to be designed, but also on the

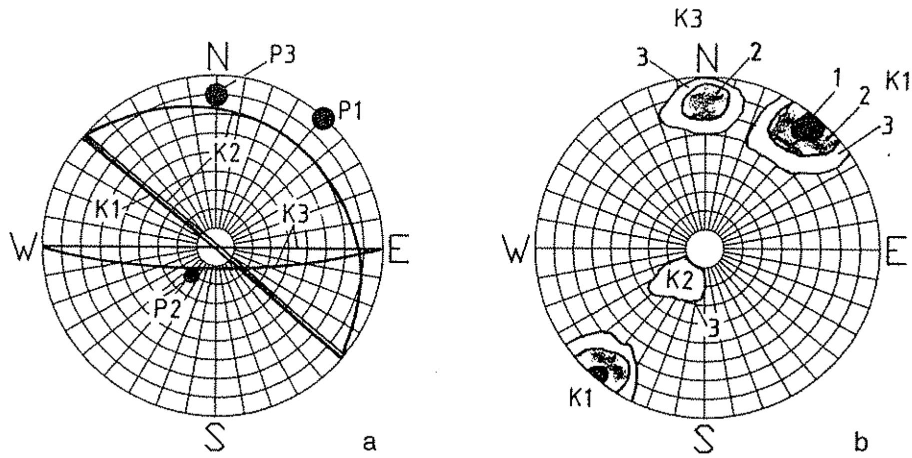
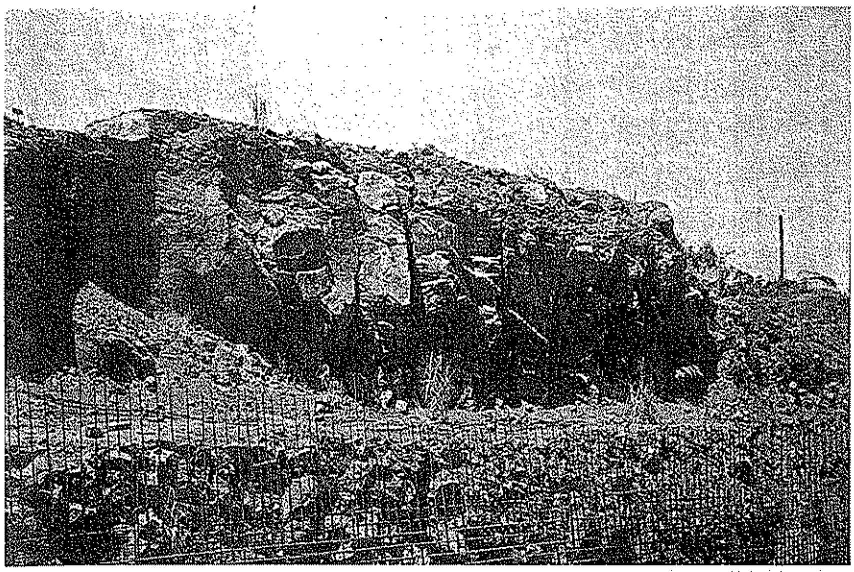
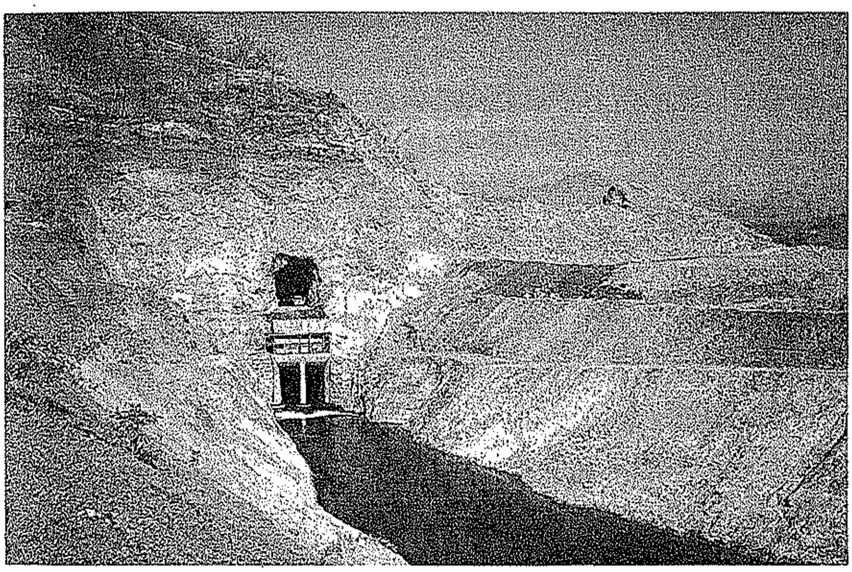


Figure 4.2. Graphical presentation of discontinuities.

a Strike and dip in the polar stereographical net 1 20 to 15%
b Frequency diagram 2 15 to 10%
K1, K2, K3 Sets of joints 3 <10%
P1, P2, P3 Poles



a



b

Figure 4.3. Rock excavations with the requirement of geological evaluation in the design phase and continually during construction (courtesy of LI).

a Project Agus IV. Column basalt at the spillway
b Project Kinda. From bottom to top, excavations for diversion channel, diversion tunnel, power tunnel and valve chamber of the power intake

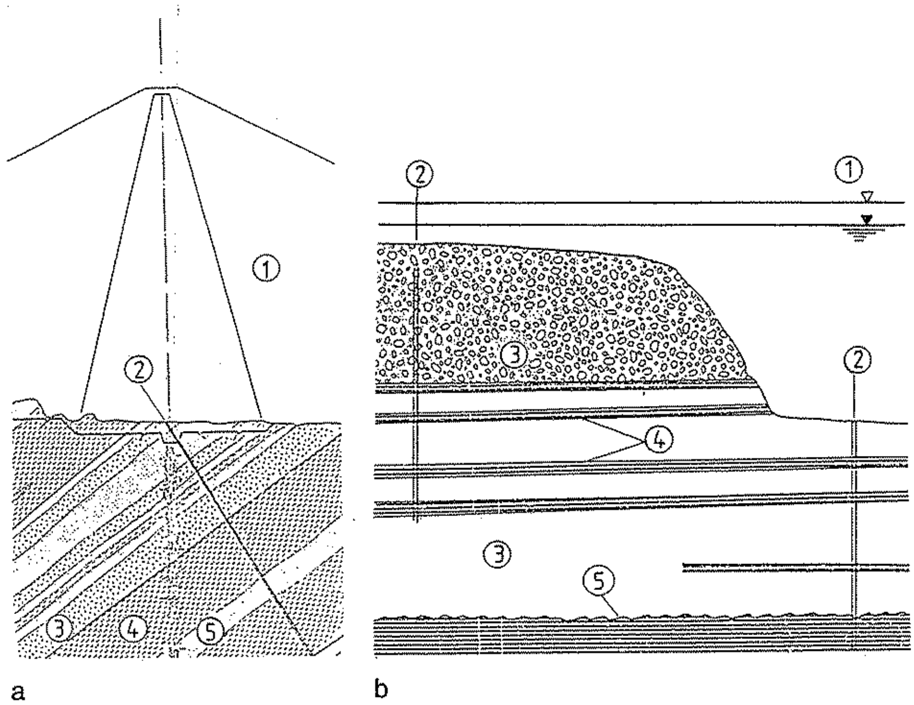


Figure 4.4. Examples of different substrata conditions.

a 1 Contours of a rockfill dam with earth core
  2 Drilling
  3 Conglomerate, very strong
  4 Claystone
  5 Sandstone
b 1 Crest and reservoir level of a gravel dam with earth core
  2 Drillings
  3 Conglomerate, weak and moderately strong
  4 Mudstone layers
  5 Sandstone

geological conditions (Fig. 4.4). The conditions of Figure 4.4a need mainly inclined boreholes, and those of Figure 4.4b need mainly vertical ones. Such conditions should be discovered by geological mapping and then be confirmed and detailed by the drillings. The complete picture of the geological conditions will develop progressively with mapping, drilling and other geotechnical investigations.

Work on the local geology includes the evaluation of all investigations described in Sections 4.2.2 to 4.2.6. Close cooperation of the geologist and of the responsible engineer or geotechnician is required.

4.2.1.2 *Geophysics*

There is a number of physical methods of determining geological conditions and rock and soil mechanical data. All such methods suffer from difficulties in interpreting the results because of the heterogeneity of rock and soil masses. Accordingly, the methods are distrusted by the profession and, accordingly, their application is of minor importance compared to other field investigations.

The most common method is a seismic refraction survey to identify boundaries between features of different physical properties. It is applied to find the bedrock below the overburden of soil, thus assessing the necessary depth of excavation. Because of easy handling and low costs, a seismic refraction survey is often applied with remote projects where difficulties arise in operating heavy machinery like drill rigs.

Seismic waves are triggered at the surface by light blasting, drop weights or hammer blows. The waves are reflected by a solid bed and monitored by geophones at the surface. The geophones are arranged along a measuring profile at different distances from the source of the waves. The depth of the solid bed can be derived from the time of wave transfer between the source and the geophone, provided the velocity of wave propagation in different layers is known, at least approximately, and provided the layers are sufficiently isotropic. Fell et al. (1992) report: 'It is the author's experience that in a residual weathered profile the base of the lowest velocity layer is usually a reasonable approximation of the probable general foundation stripping level and the base of the second layer a reasonable approximation of cutoff excavation level'.

Examples of wave velocities are given in Table 4.3. In general, the wave velocity increases with increasing strength and density of the penetrated medium.

These provisions reflect the limitations of the method: the reliability of the results increases with the differences of wave propagation within the penetrated layers and with the homogeneity of the layers. That means in practice: the interpretation is more difficult when differently weathered layers of rock exist between the surface and the solid bedrock. It is necessary to calibrate the seismic measurements with the help of core drillings which permit us to correlate the seismic readings with the profile of boreholes. The spacing of the geophones may be 20 to 50 m, and that of drill holes 150 to 200 m. The spacing depends on the number of different layers, on the consistency of layer thickness and on the homogeneity of each layer.

Another geophysical tool is the measurement of electrical resistance. The method requires near horizontal stratification. It can be used to determine the extent of a given layer. As an example: the method was applied to investigate whether the top mudstone layer shown in Figure 4.4b extended over a large area or whether it was a large lens only. Details of the method are de-

Table 4.3. Velocities of propagation of longitudinal elastic waves in m/s.

Reference	Medium	Velocity of propagation
1	Sandstone, strongly jointed	340 to 440
	Sandstone, jointed	700 to 1100
	Claystone, dolomite	2000 to 2050
2	Air	330
	Water	1400 to 1500
	Sand	300 to 1500
	Residual soil	300 to 1500
	Sandstone	1500 to 4300
	Limestone, dolomite	4000 to 4500
	Granite	5800 to 6300
	Gabbro	6400 to 7600
	Peridotite	7800 to 8400
3	Sand, moist	300 to 1800
	Clay	770 to 1900
	Loess	770 to 2100
	Sandstone	970 to 5300
	Limestone	1600 to 6300
	Dolomite	3200 to 7000
	Granite	4250 to 6200
	Diabase	6250 to 6850
	Basalt	5000 to 6400
	Gabbro	5100 to 6800
	Gneiss	3700 to 6000
	Phyllite	1700 to 5000
4	Gravelly, sandy soils	
	– above ground water table	1000
	– below ground water table	> 1400

References:
1 Militzer et al. (1978)
2 Kretzke (1969)
3 Militzer & Weber (1987), mainly laboratory results
4 Heitfeld (1991)

scribed by Whiteley (1983). The reliability of the method depends again on the homogeneity and isotropy of the substrata – and, necessarily, on the experience of the measuring and interpreting team.

Other geophysical methods are applied in boreholes to recognize different lithological formations and a limited number of rock properties. Most of them require uncased, i.e. stable, boreholes. Stable boreholes for investigatory purposes are usually cored at full length. The rock properties can be derived from tests on the drill cores. So, borehole geophysical logging is not commonly applied in dam engineering.

4.2.2 *Boreholes*

4.2.2.1 *Core drilling*

The core drilling methods serve to collect undisturbed or largely undisturbed samples of rock and cohesive soil for visual inspection and laboratory testing. Therefore, the core drilling methods are excellent tools for investigating the substrata. The core drilling methods are developed to high standard. Apart from this, skill and experience on the part of the drilling crew is required. The details of drilling methods and equipment are presumed to be known to the reader. They may be taken e.g. from drilling manuals and from Kutzner (1996).

In solid rock the core drillings are put down with the aid of diamond bits and water flushing. In weak rock and in soil carbide bits are in use. For investigatory purposes multiple core barrels only should be applied, the sets consisting of 2 or 3 coaxial tubes. Such an arrangement minimizes the disturbance of the core due to the mechanical attack of the drilling tools. In soil, dry drilling is preferable to using water or mud flushing. Only then are the drill cores largely undisturbed. In cohesive soil, flushing might effect erosion of the sample and the bottom of the borehole. In non-cohesive soil, fines might be washed out resulting in a change of the original soil particle composition. All the samples (drill cores) are stored in core boxes in the correct order, for inspection and classification.

Another method of subsoil investigation is auger drilling. The auger is a tool like a corkscrew being rotated by hand or mechanically. Auger drilling is a dry drilling method, restricted to soil down to a depth of about 30 m. It works in moderately cohesive soil, non-cohesive sand and sandy gravel. Disturbed samples of soil are conveyed to the surface, permitting us to identify the stratification of the subsoil and the suitability of the material for dam construction. The method is, therefore, used to investigate construction material deposits.

The auger may be attached to a drill pipe of large diameter serving as a core barrel. Such a device was employed to catch samples, 70 mm in outer diameter, from 40 m depth applying a wire line core barrel (Formazin 1987). The outer diameter of the drill pipe was 180 mm. A similar tool, but without the wire line technique, is described by USBR (1974).

The quality and hence the informative potential of the drill cores depends mainly on the strength properties of the substrata. Other effective parameters are the diameters of the borehole and the core, the quality of the drilling equipment and the skill of the drilling crew. Table 4.4 gives an assessment of suitable diameters.

The drilling progress must be carefully registered. Loss of flushing and drop of drill rods are indications of the rock conditions. The nature and the conditions of the drill cores are evaluated by the geologist. He produces also

Table 4.4. Recommended minimum diameters of drill cores.

Substratum	Uniaxial compressive strength (MPa)	Minimum drill core diameter (mm)	Borehole diameter (mm)[1]	Drilling procedure
Rock, very strong, slightly jointed	> 80	≥ 56	≥ 76	Rotary, diamond bits
Rock, moderately strong, moderately jointed	50 to 80	≥ 66	≥ 86	Rotary, diamond bits
Rock, strongly jointed and/or broken	20 to 50	≥ 80	≥ 101	Rotary, diamond or carbide bits
Rock, weak, friable	10 to 30	≥ 80	≥ 101	Rotary, carbide bits
Conglomerates, slightly cemented, without coarse gravel	10 to 15	≥ 90	≥ 116	Rotary, carbide bits
As above, with coarse gravel	5 to 15	≥ 120	≥ 150	Rotary, diamond bits
Cohesive material, very stiff (e.g. siltstone)	5 to 15	≥ 66	≥ 86	Rotary, carbide bits
Cohesive soil, plastic (silt and clay)	< 5	≥ 120[2]	≥ 150	Rotary, carbide bits or pipe driving without rotation

[1]Borehole diameter compatible with minimum core diameter at the use of double core barrel
[2]Undisturbed sampling for laboratory tests

a core box protocol. It is state of the art to take colour photographs of the core boxes, in the correct order of the borehole depth.

The location and the direction of the boreholes are selected so as to investigate the following:

– The foundation conditions of the dam and of all appurtenant structures with respect to the bearing capacity (items 6 and 7 of Table 4.2),

– the foundation conditions with respect to the permeability below the dam and related structures. These are the spillway and all tunnels penetrating the grout curtain (items 6 and 7 of Table 4.2),

– the subsoil conditions with respect to the stability of existing and new slopes (item 6 of Table 4.2),

– the extent of potential borrow zones for construction materials and their properties (item 5 of Table 4.2).

The investigations with respect to the bearing capacity are concentrated on the foundation area of the sealing element, since critical settlements will affect mainly the sealing. This means, for a dam with a central core, that a number of boreholes must be located along the center line of the dam (Fig. 4.1). For a dam with a surface sealing the boreholes must partially be located in the vicinity of the upstream toe of the dam, where the load impact on the sealing is maximum. The direction of the boreholes must be adjusted to the dip of the bedding and joint systems (Fig. 4.4).

In the following cases an appropriate number of boreholes must be located also below the dam shells:

– The dam stability is adversely affected by weathered rock or weak layers in the foundation. Critical slip circles cut through the foundation below the shells (Fig. 6.7c),

– excessive deformations because of collapsing karstic caves cannot be excluded.

The boreholes should show the elevations of highly and moderately weathered rock and of the bedrock, which are needed to quantify the necessary excavations. Discolouring indicates the degree of weathering, in most cases from grey/black to reddish/brown. The weathering is classified according to Table 3.2 (or other equivalent systems). The test pits of series F in Figure 4.1 are mentioned in this context, the pits enabling the identification of a layer of liquefiable soil which had to be removed from the foundation.

The location, depth and direction of boreholes below the appurtenant structures are selected according to common rules of foundation engineering. Figure 4.1 shows boreholes for the structures of power intake, spillway sill, spillway chute, stilling basin and power house.

The boreholes investigating the bearing capacity serve also to investigate the permeability, where required. The depth must be selected accordingly since the permeability conditions and the system of seepage paths must be known down to greater depths. In general, the boreholes reach an impermeable stratum or such a depth where seepage does not create critical effects on the dam. The boreholes and the tests made in them serve to define the depth of the required sealing measures (Sections 4.2.2.2 and 4.2.2.3). Karstic and artesian conditions may lead to borehole depths of several hundreds of meters.

Boreholes investigating permeability are needed in both abutments, in extension of the center line of the dam. As a rule, the grout curtain is extended to where the storage level coincides with the natural ground water level. This defines the area of investigations on both sides of the dam. Additional boreholes are required along the reservoir rim where significant seepage to neighbouring valleys might occur. At Kinda dam, for example, investigations and grouting measures had to be extended along the access road 1700 m beyond the left dam shoulder. Respective investigatory boreholes can be seen in Figure 4.1.

Particular attention has to be paid to the thalweg, i.e. the line connecting points of greatest depth along the river course. Inclined boreholes are commonly located at the riverbanks to encounter the bedrock contour. Often, there exist deeply eroded gullies, filled with river alluvions, necessitating special excavation and sealing measures. An instructive example is the Aigueblanche dam, France (Fig. 4.5). In addition to initial investigations a group of boreholes had to be drilled from the diversion tunnel to identify the actual rock contour.

The report on local geology must consider the slope stability in the reservoir. Impounding may cause instability of such slopes with subsequent

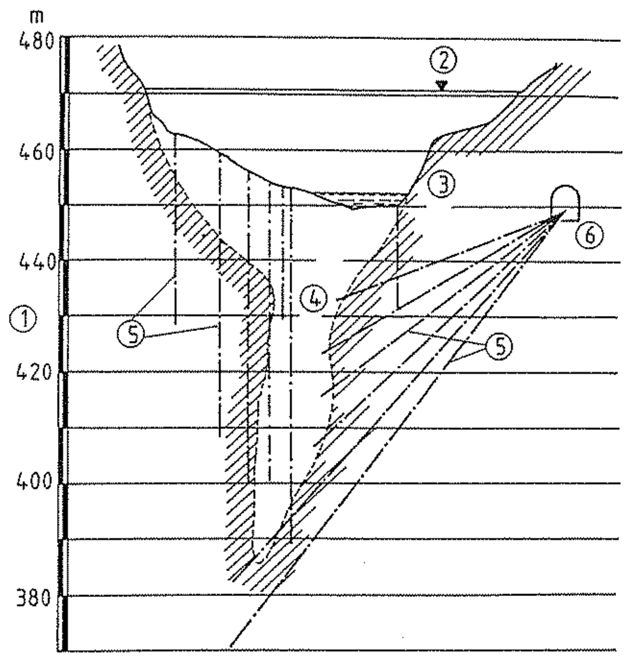


Figure 4.5. Barrage Aigueblanche, France 1950. Drillings investigating the rock contour.

1 Elevation (m a.s.l.)
2 Full supply level
3 Rock
4 Alluvions
5 Drillings
6 Investigatory and diversion tunnel

sliding of large rock and soil masses. Boreholes may be required for sampling and defining material parameters for stability computations.

The reservoir slopes close upstream on both sides of the Mornos dam are an example (Fig. 6.8): the mylonite layers, as shown, made a detailed survey advisable. Core drillings were put down to encounter the stratification and to take samples from weak zones. Calculations revealed the risk of instability of the existing slopes under earthquake loading after impounding (Kutzner 1980). Slide failure of slope debris or colluvium might produce a wave overtopping the dam. This had to be excluded. The design of rehabilitation measures resulted in extensive re-modelling of the slopes, with related earth moving works, to the amount of 7.3×10^6 m^3. (The dam volume is about 12×10^6 m^3).

Finally, core drillings are needed to investigate the borrow pits for construction materials (item 5 of Table 4.2). The identification of the pits is part of the geological survey. The responsible geologist and the materials engineer should, in close cooperation, locate the boreholes and evaluate the drill cores. The drilling results must permit us to distinguish rock to be blasted from rock to be ripped. They should inform the geotechnicians on the joint systems and hence on the reasonable blasting pattern to exploit quarries. We must also know the amount of overburden, as an input for tender documents.

4.2.2.2. *Water pressure tests*

Water pressure tests (WPTs) are made in the boreholes investigating the permeability. The test is also called the LUGEON-test, after its initiator (Lugeon 1933). The test gives the water absorption potential of the rock in relation to the pressure applied at the particular testing place.

The water absorption potential is related to the permeability. However, any existing anisotropy with respect to permeability cannot be found out by the test since the flow direction of the water pressed into the ground remains unknown. Such anisotropy affects the seepage conditions below the dam. Irrespective of this, the result of WPTs is one of the main criteria for designing sealing measures. The test procedure is as follows (Fig. 4.6):

The lower and the upper section of the borehole are separated by a packer. Water is pressed into the lower section, commonly at ascending and descending pressures. After steady flow conditions have developed the amount of water absorbed per unit of time is registered and expressed in terms of Lugeon-values (LU). 1 LU corresponds to 1 l of water absorbed per 1 m of borehole and per one minute at a pressure of 1.0 MPa. Usually, investigatory boreholes are WP-tested at full length, the test sections being 5 m long. It is common practice, where borehole stability permits, to do the tests according to the drilling progress from the top to the bottom of the borehole using a single packer, as shown.

The maximum applied pressure should not exceed 1.0 MPa. With indi-

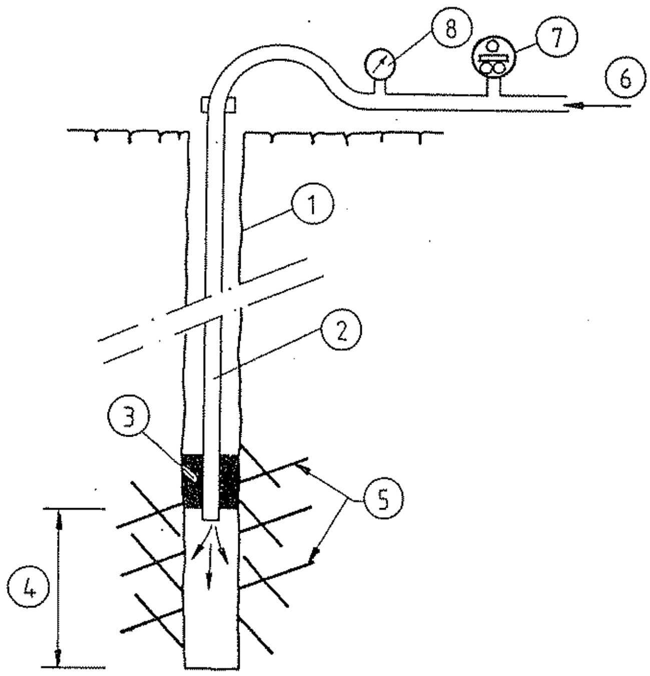


Figure 4.6. System of water pressure test after Lugeon (WPT).

1 Borehole	5 Joints
2 Packer rods	6 Pipe from the pump
3 Packer	7 Water meter
4 Test section	8 Pressure gauge

vidual tests the test pressure should be selected so as to define the cracking pressure, which helps to specify the permissible grouting pressures. Details of the tests, their limitations and sources of mistakes, and the interpretation of the results are described, e.g. by the author (1996).

In weathered rock, close to the surface, and in soil, the packer will not find a fixed and tight position. Only low pressures can be applied. Under such conditions percolation tests using a stand pipe are often made instead of water pressure tests. Usually, the results are expressed in terms of permeability after Equation (4.2), irrespective of the limitations in correlating the percolation and the permeability (Section 4.2.6.2a). Accordingly, the value of stand pipe tests is limited. The correlation renders a rough approach only to k-values in the sense of Darcy. In most cases the result is irrelevant for the design of sealing measures. Loose and permeable strata close below the sealing element of a dam must always be excavated or consolidated and sealed by grouting, or made impermeable by a diaphragm.

4.2.2.3 *Test grouting*

As is known, in most cases the water absorption of rock and soil cannot be correlated with the take of grout. This is, in part, due to the different rheological properties of water and grout material. It is, essentially, due to the penetration potential of suspensions which depends mainly on the particle size of solids in relation to the width of individual joints, instead of the water absorption potential of the joint system.

Groutability can be determined by test grouting, which should be done in some investigatory boreholes, in addition to water pressure testing. This test grouting in individual boreholes is different from a large scale grout test as described in Section 4.2.6.3. Such a grout test serves to determine the most reasonable grouting procedure. In contrast, test grouting in individual boreholes serves mainly to test the groutability of the rock. The result will give valuable help for the design and execution of grouting work.

Test grouting may be performed, alternating with water pressure tests, together with drilling from top to bottom, or, after reaching the bottom depth, from bottom to top. A comparison of the water and grout takes for each section is enabled by equal packer positions with water testing and test grouting.

Stable suspensions only at a water/cement-ratio of about 1.0 should be used if cement suspension is a reasonable grout material. Processing of the material including pre-activation of bentonite must comply with the usual procedures. The use of colloidal mixers is a precondition for success. According to experience the author likes to note the following: many 'practitioners' on site feel tempted to do test grouting with too large a variety of parameters. This, frequently, obscures the test leading to loss of information and misinterpretation. Instead, test grouting should be done in the respective boreholes under the same work conditions, irrespective of the type of rock at the particular borehole location. For all boreholes the grout pressures must be kept constant at selected depths, and the grout materials must be the same.

The following example demonstrates the value of test grouting in a single borehole. During the design of a multi-purpose project in Iran it was advisable to check the groutability of the substrata, before a large scale grout test was to be budgeted. With respect to the height and the length of the dam, 100 and 3000 m respectively, it was prudent to investigate the feasibility of conventional and hence economic grouting of ordinary cement.

The substrata, constant over the width of the valley and down to great depth, consist of moderately to slightly cemented conglomerate of sand and gravel with 20 to 40% sand < 2 mm. The coarsest particles are cobbles up to 250 mm edge length. With respect to the grain size composition the conglomerate is in a critical range of groutability, namely at the lower limit of soils groutable with ordinary cement.

Water pressure tests revealed low and high absorption potentials in the range of 5 to 70 LU. The high values are related to highly permeable gravel layers without a significant content of sand, so called open-work gravel. Such layers had been found in trenches close to the surface (Fig. 4.7). These conditions had not been clearly identified from the core boxes since no compact drill cores had been obtained, due to the low strength of the material.

Test grouting was performed in a 90 m deep cored borehole, progressively after water pressure tests from top to bottom. The procedure was adjusted to the prevailing conditions, as far as possible, using core barrels 101 mm in diameter and inflatable packers exclusively. Only stable or almost stable mixes at a W/C-ratio of 1.0 were grouted. In general, the take of grout was less than 30 kg of cement per meter of borehole. Three sections 5 m long had a take of 130 to 160 kg/m. These sections had penetrated layers of open-work gravel, 0.8 to 1.4 m in thickness. Such groutability and the high

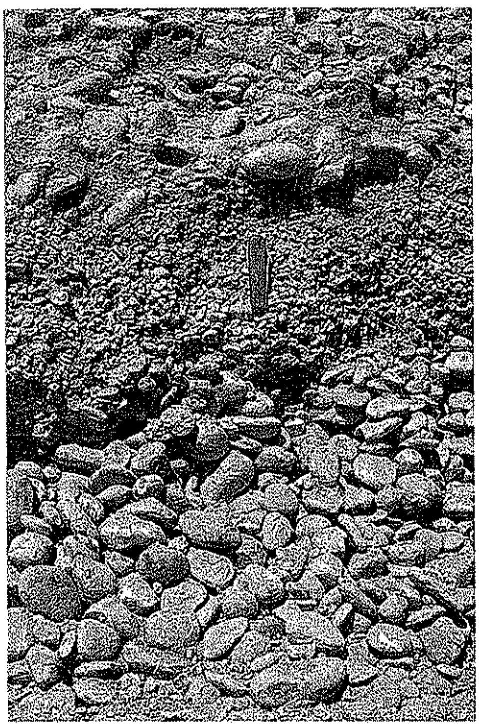


Figure 4.7. Stratification of slightly cemented conglomerate with layers of very different permeability.

Top: Clayey gravel with cobbles
Center: Sandy gravel with cobbles
Bottom: Poorly graded gravel (open-work gravel) and cobbles

LU-values gave evidence that the gravel layers, being the prominent paths of seepage, could be grouted successfully with ordinary cement.

The question was left open whether other sections of moderate water absorption but low take of grout had to be sealed, and which procedure would be reasonable. Ultra fine cement would probably be feasible. Remaining sections of low take, less than 30 kg/m, were seen as sufficiently tight without grouting.

4.2.2.4 *Rock mechanical tests in boreholes*

Rock mechanical tests in boreholes are useful mainly for the design of subsurface structures. Such tests are rarely needed for the design of embankment dams.

An example is the dilatometer test, serving to determine the stress and strain behaviour of rock at triaxial stress conditions. Such tests may be helpful if drill cores for laboratory tests cannot be obtained. Examples are coarse conglomerates of low strength (Fig. 4.7) and shale. The cementation of such conglomerate may not resist the drilling attack, and the shale may disintegrate to individual plates due to stress relief when the core is separated from its environment. Dilatometer tests may serve also to determine the strength anisotropy of a rock mass.

The dilatometer must fit the diameter of the borehole. Usual borehole diameters range from 75 to 100 mm. A description of different types of dilatometers and the evaluation of test results can be found e.g. from Wittke (1990). In evaluating the results it has to be recalled that the stress conditions of the tested rock around the borehole are completely different from those to which a tested drill core is exposed in the laboratory. This may lead to great differences in calculated moduli of deformation.

In general, a critical remark should be made about dilatometer and other borehole tests: the tests must be done by people experienced in rock mechanical testing. Otherwise the interpretation will not lead to results which are useful for the design work. For this reason test results have sometimes to be disregarded. Then the question arises whether it was worthwhile spending the money for testing.

4.2.2.5 *Observation of ground water*

The ground water level must be registered in all cored boreholes. Daily readings are made in the evening after the end and in the morning before the beginning of work. An example is given in Figure 4.8, providing the following information:

- The permanent ground water level is at 12 m depth,
- an artesian pressure of 4 m water head exists in the porous sandstone. The information helps in evaluating the risk of hydraulic ground failure in case of excavations and in controlling grouting work,

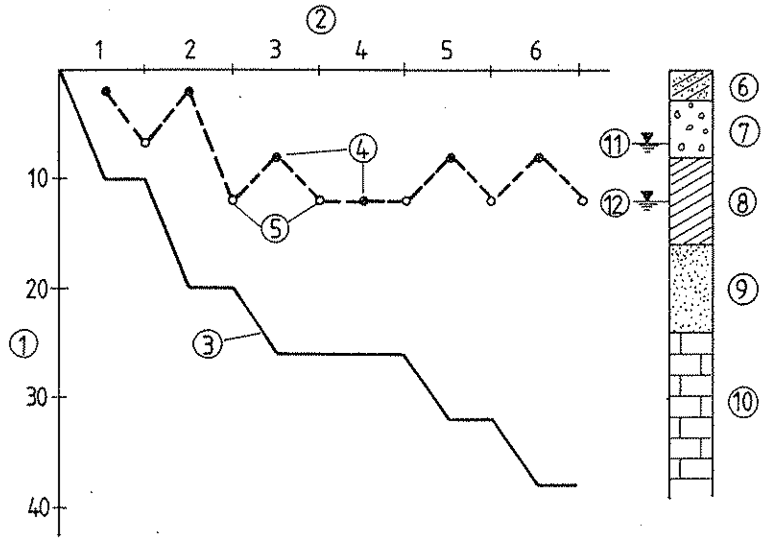


Figure 4.8. Ground water observation during drilling.

1 Drilling depth (m)	7 Gravel
2 Working days	8 Clay
3 Drilling progress	9 Porous sandstone
4 Water level in the borehole, evening (dots)	10 Impervious rock
5 Water level in the borehole, morning (circles)	11 Water lens
6 Sandy silt	12 Ground water level

– the temporary water level in the gravel represents a confined lens of water. This information helps in evaluating the dewatering necessary for excavations,

– the bedrock is tight. The permanent water level does not change after penetration of the rock.

In addition, ground water observation is required to design sealing works in the abutments (length of grout curtain), to specify grouting pressures and to calculate the effective pressure with water pressure tests. Observation of a group of boreholes reveals information on the ground water conditions in the vicinity of the dam and related rock permeability.

4.2.3 *Penetration tests*

In general, penetration tests serve to investigate the sequence of strata and the bearing capacity of the subsoil. The tests are applied mainly in foundation engineering. In dam engineering they are applied to find out the sequence of strata and the layer thickness, for instance to investigate potential deposits of construction materials.

Most common is the Standard Penetration Test (SPT) which is applied in boreholes. A split-tube sampler is driven from the bottom of the borehole into the ground. The sampler is about 60 to 100 cm long (24 to 40 inches). The outer and the inner diameters are 51 and 35 mm, respectively (2 and 1.4 inches). The driving length is usually 30 cm (1 foot) in three sections of 10 cm each. The driving force is supplied by a hammer assembly consisting of a drop weight of 63 kg (140 pounds) and a guide pipe long enough to allow a 76 cm free fall (30 inches).

The test enables us to determine the resistance of the soil to the penetration of the sampler and to obtain a representative disturbed sample. The penetration resistance is expressed as the number of blows required to drive the sampler 10 + 10 + 10 cm (1 foot) into the soil. The sampler is applied in non-cohesive and in cohesive soils. The upper limit of application is given by the maximum grain size of graded soils at about 60 mm. In coarse material a closed tip is attached to the sampler, avoiding damage of the sharp edge of the standard tip. The penetration resistance is not significantly different from that of the open sampler, but soil samples cannot be obtained.

The penetration resistance is related to the depth, gradation and density of non-cohesive soils and to the stiffness of cohesive soils. Examples: 20 blows per 30 cm penetration are required to penetrate very dense coarse sand at the surface, and 50 blows at about 7 m depth. Penetration of a stiff cohesive soil by 30 cm requires about 8 to 15 blows. (For figures of penetration and exact dimensions of the equipment refer e.g. to Schultze & Muhs 1967, USBR 1974). In combination with the sample the penetration record permits us to evaluate the type and thickness of layers and the density/stiffness conditions, and – in foundation engineering – the bearing capacity. The test is useful also to assess the liquefaction potential of saturated sands and silts (see the end of Section 7.3.4.2).

Similar penetration tests involve driving a cone of defined dimensions into the soil. Typical dimensions are: cone diameter 36 and 44 mm, tip angle 90°, drop weight 10 and 50 kg, free fall 50 cm. In principle, the findings are the same as with the standard penetration test.

An alternative is the static Cone Penetration Test (CPT) where a cone is pressed into the ground. The pressure required to force the cone into the ground is mechanical or hydraulical. Usually, the equipment is mounted on a carrier or truck which counterweights the penetration resistance of the soil. The carrier is anchored to the ground, if required.

The penetration resistance is expressed in terms of the tip pressure. It amounts to about 65 MPa in very dense sand at 10 m depth above the ground water. Below the ground water it is considerably reduced. Again, the penetration resistance is related to the depth, gradation and density of non-cohesive soils. An example of a record is shown in Figure 7.66. The test record enables an assessment of the sequence of layers, but soil samples and classification tests are required for a complete evaluation of cone penetration tests.

4.2.4 *Test pits*

4.2.4.1 *Geotechnical soil description and sampling*

Soil deposits without much overburden are investigated by test pits or – far better – by bulldozer trenches (Fig. 4.9). The quality and quantity of information increase with the size and depth of the exposed areas. In non-cohesive soils the excavating depth is commonly limited by the ground water. In cohesive soils it may be limited by the risk of collapse. The investigatory depth can be increased further by auger drilling or penetration testing from the bottom of the pit.

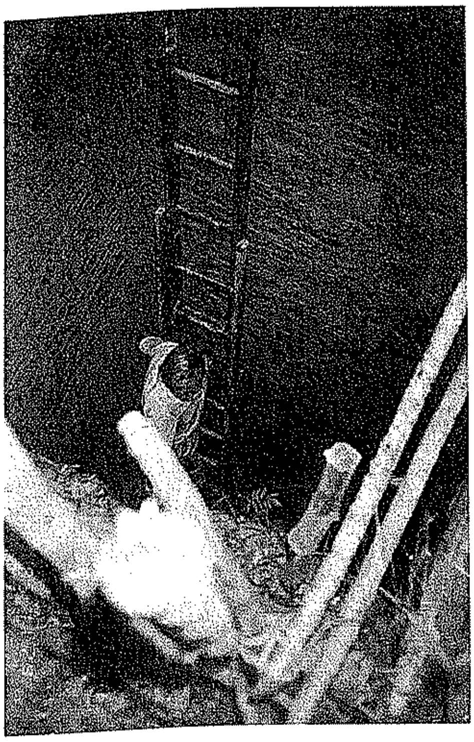
At least one wall of the pit must be mapped. Mapping should include soil classification, as far as possible by visual inspection. It should show the stratification, lenses, intrusions and the like. Remarks on soil properties must be made, such as 'with stones', 'very fine', 'plastic', 'hard, strong', 'extremely dry' or 'extremely wet'. The last remarks for a cohesive soil, for instance, point to the need of wetting or drying out before placing such soil as a dam sealing material. 'Friable contents' of an alluvial soil point to the suspect the material may not be suitable as filter material or concrete aggregate. Such remarks require close cooperation of the geologist and the material engineer, as indicated in Table 4.2. The remarks help to decide on the suitability of the material for the purpose in question.

Disturbed and undisturbed soil samples for laboratory testing are taken from the pits, according to common rules of sampling (USBR 1974). The responsibility is mainly with the engineer because he will know best which tests are required and how many samples should be taken with respect to the type and weight. The amount of samples is not only limited by the costs of testing but also by the costs of transportation and shipping to overseas laboratories. In some countries customs restrictions on importing soils have to be considered.

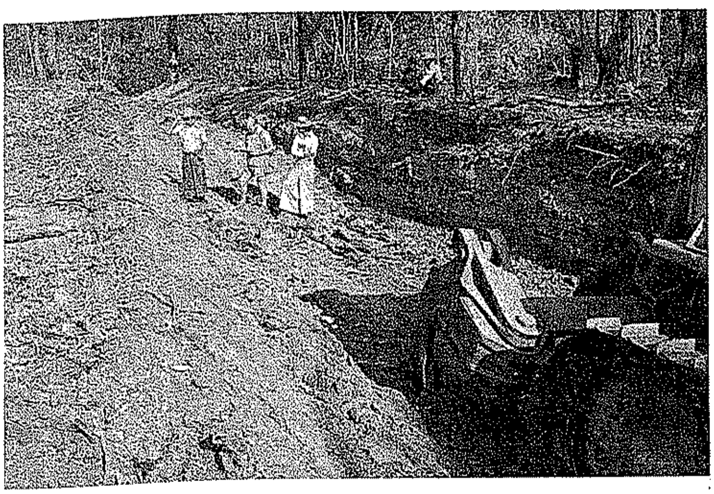
4.2.4.2 *Soil mechanical field investigations*

a) Cohesive soils

Some field testing is recommended in the course of test pitting. The natural water content of cohesive soils must be determined to know its difference from the water content which is optimum for compaction. If there is a risk of drying out during transportation the water content must be determined in the field. The risk of drying out cannot be excluded in all subtropical and tropi-



a



b

Figure 4.9. Soil investigations in a borrow area (courtesy of LI).

a Test pit
b Bulldozer trench

cal areas where a competent soil mechanical laboratory is not located close to the site.

The natural water content can be determined in the field using the carbide bomb moisture meter (CM-device, Fig. 4.10). A soil sample, about 20 g in weight, is placed in the pressure proof container (bomb) with carbide. A gas develops of water and carbide, and its pressure is related to the water content of the sample. The correlation of gas pressure, sample weight and water content can be read from a calibration chart.

A simplified method is to weigh the sample before and after artificial drying out. In tropical areas the process of drying out may be done in the open air with the sample spread on a pan (hotplate method). In other areas the pan and the sample must be heated on an oven. Drying out with the CM-device or hotplate provides the water content to an accuracy of about 2% which is sufficient for a quick test.

Other field tests on fine cohesive soils are:

- Determination of the moist unit weight,
- estimation of the shear strength by means of an inplace vane shear test,
- estimation of permeability by means of a percolation test.

The moist unit weight (in situ) helps to calculate 'loosening' and 'compacting' factors related to the actual fill unit weight of the compacted soil in the embankment. These factors, however, are of limited value since an amount of about 200% of the required material has to be identified in the field (Section 4.4) which by far exceeds the amount calculated by the use of 'loosening' or 'compacting' factors.

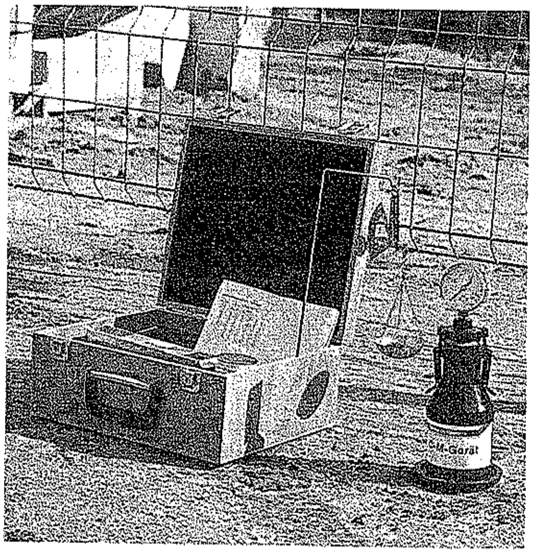


Figure 4.10. Carbide bomb moisture meter (courtesy of Trischler & Partner).

From left to right: Calibration chart in the box, balance, pressure proof bomb with pressure gauge

The shear strength can be estimated in the field by visual inspection of the test pits or of soil samples. Such an expert estimate will be sufficiently accurate for classification and suitability considerations. For design purposes more accurate values have to be determined by laboratory tests. The inplace vane shear test (USBR 1974) may be useful to estimate the bearing capacity in foundation engineering.

Permeability can also be estimated by visual inspection. About the value and the limitations of percolation tests see Section 4.2.6.2.

b) Non-cohesive soils

It is common practice to prepare grain size analyses of coarse materials in the field. A large amount of material can quickly be sieved with a small number of sieves, e.g. 200, 60, 20 and 6 mm, applying a set of vibrating sieves or hand-sieving. The fraction below 6 mm is sent to the laboratory for further analyses. Shipping of large samples is omitted.

c) Field laboratory

These field investigations should be seen as acceptable improvisations. It is preferable to have a field laboratory already operating in the design phase. The laboratory should be equipped for the following procedures:

- Determination of the water content,
- determination of the plasticity index,
- sieve analysis,
- hydrometer analysis (desirable),
- compaction test.

Such a laboratory was operated, for instance, for the large field compaction test on weathered mudstone which is described in Section 4.2.6.2. The output of the laboratory enabled evaluation of the material as a dam core material and selection of the type of dam, namely a rockfill dam with earth core.

4.2.5 *Adits*

Investigatory adits are rarely needed for embankment dam projects without subterranean structures. An exception are projects with very unfavourable or extremely complex geological conditions, such as karstic rock or abutments of doubtful stability. The project Atatürk is an instructive example of the need for adits in karstic rock (Fig. 4.11). Tunnels, several hundreds of meters in total length, had been driven. The effort was not sufficient to discover the complexity of the karst. The sealing measures, entirely necessary, exceeded by far the predicted amount.

The clearance of adits must be sufficient for geological mapping and for the performance of rock mechanical tests. Reasonable dimensions are 2.5 to

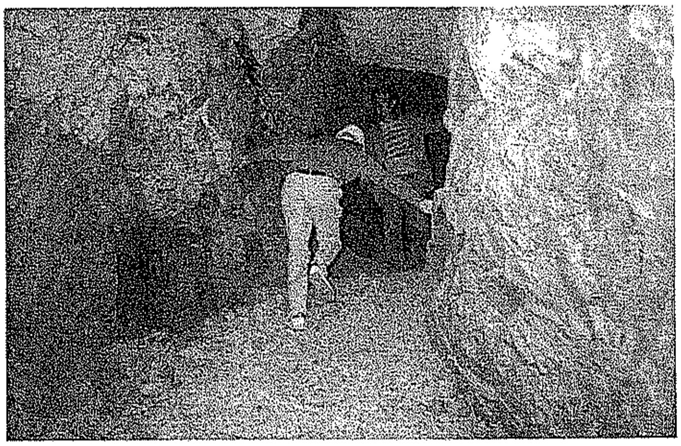


Figure 4.11. Large cavity in karstic limestone, about 20 m below river Euphrates, Turkey (courtesy of LI).

3.0 m in height and 2.0 to 2.5 m in width (5 to 8 m^2, Fig. 4.12). Excavation of adits is expensive. They require support and sometimes drainage and ventilation.

Complete mapping covers the roof, the walls and the bottom of the adit (4-face mapping). Such mapping is rarely possible, because of interference with work progress and the need to clean the bottom. Therefore, 3-face mapping of the roof and the walls is usually practiced. Further limitations should not be accepted. Only 3-face mapping can reveal correctly the orientation, strike and dip of bedding and joint systems. Joints, in particular stress relief joints, have an important effect on the bearing capacity of the rock mass. Mapping should be accomplished by an assessment of rock mechanical parameters and a description of seepage conditions.

Rock mechanical tests are frequently performed in lateral chambers. There are:

– Radial pressure tests,

– plate load and flat jack tests investigating the deformation moduli of the rock,

– shear tests,

– dilatometer and water pressure tests in boreholes which are individually directed according to the purpose.

An example is explained here in more detail: in the feasibility study for a hydropower project in Guatemala the right river bank was to be investigated as the potential right abutment of a 130 m high embankment dam. The rock consisted of strongly sheared serpentinite. Investigations were directed to the stability of the abutment under the load of the dam and to the effect of seepage from the reservoir. Adits had been driven for rock inspection and sampling and to perform shear and plate load tests at nine individual locations.

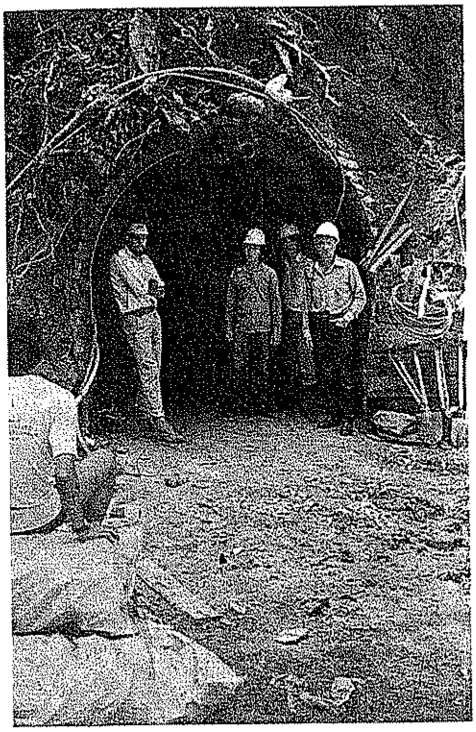


Figure 4.12. Investigatory adit, clearance about 8 m^2.

The angle of shear resistance of the rock was found to be about 20° at unsaturated and 10° at saturated conditions. Cohesion was zero. Deformation moduli were between 200 and 1000 MPa. (The project was not followed up for political reasons. So, no further aspects of the design were considered.)

It is mentioned that such low shear strength is typical for gauge material in joints. Similar parameters have also been found of the mylonites of the Mornos project (Fig. 6.8). For further design work the extension and orientation of the sheared zones have to be evaluated, as well as their effect on the stability of structures and slopes.

4.2.6 *Large scale tests*

4.2.6.1 *Blasting test*

A blasting test is desirable prior to the completion of tender documents in case a large amount of construction material has to be quarried and that the suitability of the quarries is not clearly indicated by other investigations. The test blasting must enable the designer to assess the quality of the rock and the homogeneity of the rock mass which is to be exploited. Quarried rock is used for rockfill material and, if there is not enough material available from alluvial deposits, for concrete aggregates.

A typical example of the value of a blasting test is the situation shown in Figure 4.13. The rock consists of sandstone and siltstone, with layer thickness of some centimeters to some decimeters. It was to be excavated in the spillway area of the Chico project in the northern part of the Philippines. The idea was to use the material for the shells of the 160 m high rockfill dam with earth core, related to the project. The drilling results left some doubts whether the material was suitable for use in the upstream saturated dam body, because the region is highly prone to earthquakes. Therefore, the blasting test was carried out.

The quarried material revealed rapid disintegration of the siltstone as soon as it was exposed to atmospheric conditions. The remaining mixture of solid sandstone and disintegrated siltstone proved – after a field compaction test – to be of sufficient strength and low deformability, but also of low permeability in the order of 10^{-7} m/s. Because of such permeability it was not suitable as a material for the upstream saturated dam body in respect of earthquakes.

A blasting test discovers the breakdown and crushing behaviour of the

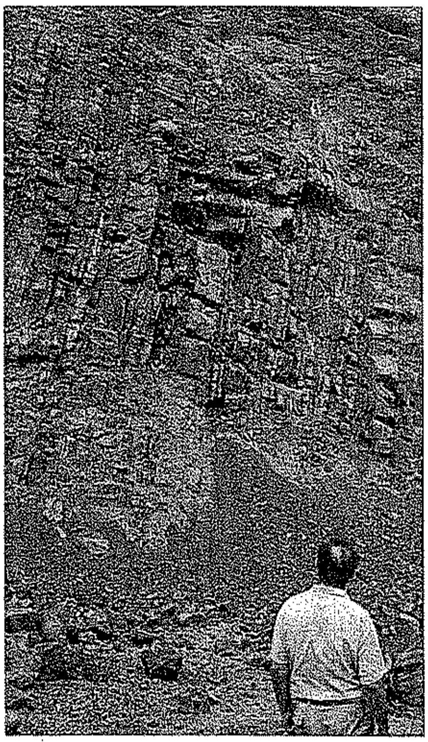


Figure 4.13. Sequence of steeply dipping layers of sandstone and siltstone.

material in the quarry. The effect of blasting depends on the strength of the solid rock and on the visible and latent jointing of the rock mass. The grid of drillholes in the quarry and the amount of explosives affects, to some extent, the grain size distribution of the blasted material. Usually, a small number of blasts is required to achieve a specified range of grain size distributions.

According to experience it can be left to the contractor's responsibility to find the most appropriate blasting parameters. It must only be ensured that the specified material, regarding quality and quantity, can in principle be obtained from the quarry and excavation areas which are indicated in the contract documents. Then, a blasting test prior to contractual work can be dispensed with.

The suitability of the blasted material as rockfill material or as concrete aggregate is investigated from samples by means of common laboratory tests (Sections 4.3.3 and 4.3.4). Point load tests in the field are a good indicator for the unconfined uniaxial compressive strength (Fig. 4.14, ISRM 1985), according to the following relation:

$$\sigma = 24 \times I_{S\,(50)} \tag{4.1}$$

where: σ = unconfined uniaxial compressive strength (UCS)
$I_{S(50)}$ = strength index of the point load test on a core sample 50 mm in diameter.

The constants for other core sizes are, approximately: 20 mm = 16, 30 mm = 19, 40 mm = 21, 60 mm = 25, 80 mm = 35.

Most reasonable samples are drill cores and fist-like pieces of rock. The size factor, length/diameter, should be 1.0 to 1.4. It is not necessary to cut the samples into geometrically defined pieces, which is one of the advantages in applying the point load test. Many tests made in the field, at low cost, permit us to assess the range of strength satisfactorily.

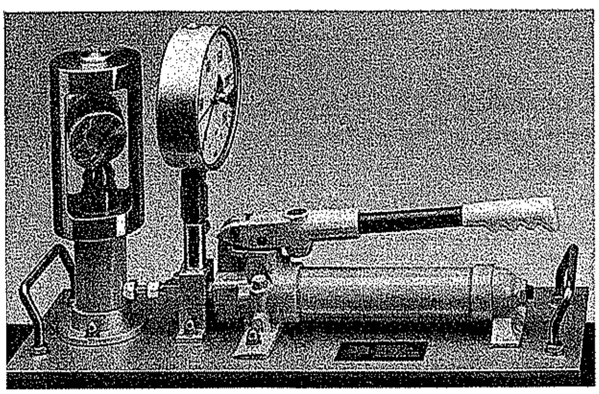


Figure 4.14. Point load tester (courtesy of Interfels).

From left to right: Hydraulic jack with testing frame and sample (drill core), pressure gauge, pump

4.2.6.2 *Field compaction test*

a) Cohesive soils
Usually, the potential construction materials are subjected to a field compaction test. Such a test serves to investigate the following:
- Method of excavation in the borrow area,
- method of drying out or moistening before compaction, if required,
- dumping and compacting method with respect to lift thickness, fill water content and tolerances, type of compacting machinery, number of passes and speed of compactor,
- achieved properties of the soil, particularly the density.

The compaction test may be done prior to completion of tender documents, or at the beginning of contractual work. The best time for testing depends on difficulties or irregularities which are expected to occur during compaction. In simple cases specifying the compaction details in the documents is justified according to experience, and doing the test after contracting. The test must confirm that the selected compaction method is appropriate. Necessary modifications must remain within the limits set by the documents.

An example is a test which was conducted for the Shen River dam (Kutzner 1982a). Recent experience was available from similar soils used for other projects in the area. The construction material in question was a laterite, a product of tropical weathering of granite. In terms of soil mechanics it is a residual clayey and sandy silt with a plasticity index of 10 to 20 and a liquid limit of 25 to 45%. The natural water content was 15 to 19% and hence close to the optimum. The soil was loaded by scrapers from a horizontal borrow area, several meters in thickness, and then carried to the embankment.

The compaction test covered three fields, 20 m × 30 m in plan, enabling the compaction of 20, 25 and 30 cm thick layers by 4, 5, 6 and 7 passes of the compactor (Fig. 4.15). The test was satisfactory in finding the most appropriate compaction method:

Compaction method
- Lift thickness 25 cm uncompacted,
- fill water content optimum to optimum plus 2%,
- compactor 300 kN static padfoot roller,
- number of passes 5 to 6,
- speed max. 8 km/h.

Material properties
- Average density 100% Standard Proctor,
- minimum density 98% Standard Proctor,
- friction angle 28° to 30° (CD-triaxial),
- cohesion 20 kPa,

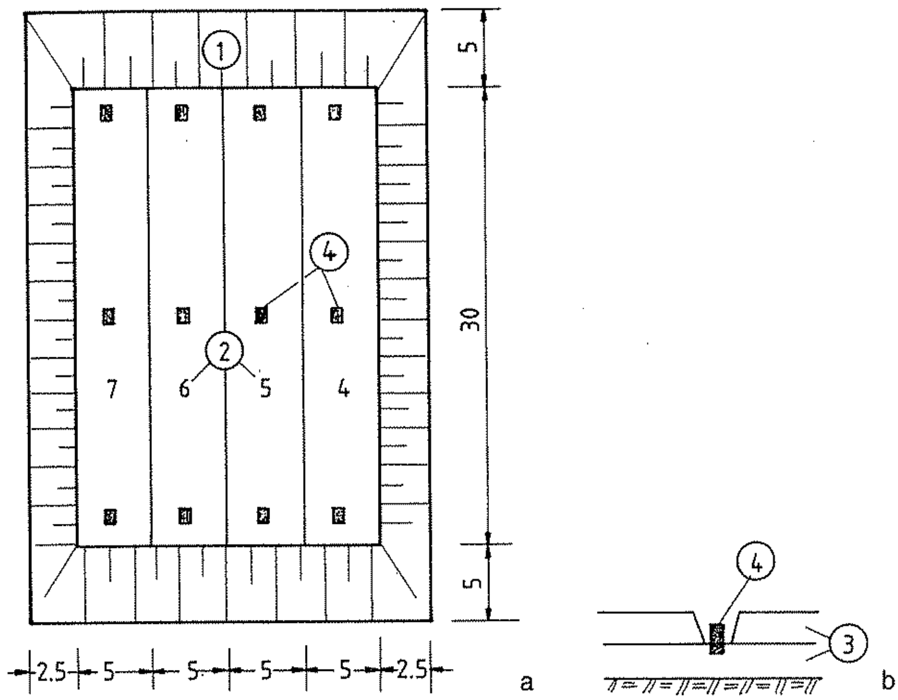


Figure 4.15. Layout of a field compaction test on cohesive material (dimensions in m).

a Plan	2 Number of passes
b Section	3 Compacted layers, thickness about 0.2 m each
1 Ramp	4 Undisturbed samples

– permeability max. 5×10^{-10} m/s.

A more complex example is Bakun which was – in a previous phase – designed as a 200 m high rockfill dam with earth core. In 1985, prior to completion of tender documents, a compaction test was performed in order to confirm the suitability of the selected core material. It is a residual material consisting of mudstone and soil thereof at different conditions of weathering (Fig. 4.16). The test served to clarify the following questions:

– Maximum depth of exploitation by excavator (no blasting),
– water content of the mix of different layers,
– lift thickness and number of passes,
– homogeneity of the compacted soil,
– fill water content and density compared to laboratory tests (Standard Proctor),
– soil mechanical properties of undisturbed samples taken from the test embankment.

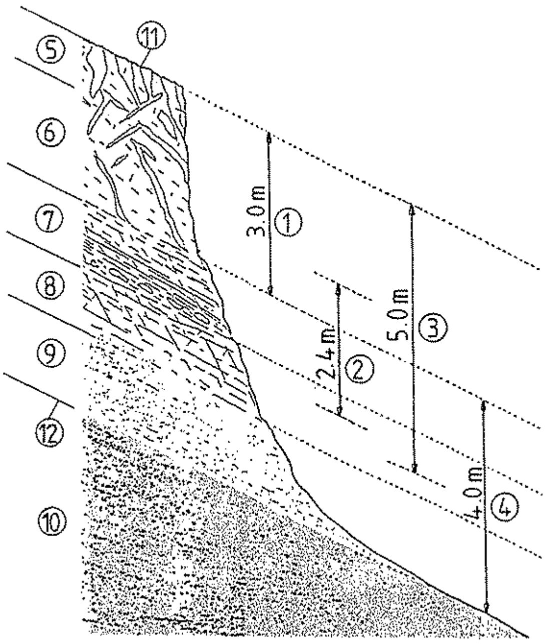


Figure 4.16. Compaction test Bakun 1985. Soil profile of potential core material.

1 to 4 Mixes of different layers, to be dumped and compacted
5 Residual soil
6 Mudstone, completely to highly weathered
7 Mudstone, highly to moderately weathered
8 Mudstone, moderately weathered
9 Mudstone, slightly weathered
10 Mudstone, fresh
11 Surface after stripping of about 30 cm rooted soil
12 Lower boundary of material exploitable by excavator or ripper

The answer to the first question was decisive since excavation down to a minimum of 4.5 to 5.0 m was required to cover the necessary amount of core material. The answer to the second question was also decisive, since only mixing different layers would result in an acceptable water content. Because of the climatic conditions of the region with a precipitation of up to 5000 mm per year, without significant differences of a wet and a dry season, the uppermost layers were too wet for compaction. Also because of these climatic conditions simple drying out of the material in the open air was problematic, in spite of high temperatures.

Excavation was managed so as to achieve 4 different types of materials as shown in Figure 4.16. The four materials were dumped and compacted separately applying lift thicknesses of 20 cm (nos 1 to 4) and 30 cm (nos 1 and

A 180 kN static padfoot roller was used for compaction by 4, 6 and 10 passes.

The total area of the test embankment was about 120 m × 26 m in plan. Four inspection trenches were excavated across, protected from rainfall and sunshine (Fig. 4.17). Such trenches proved to be the best measure to evaluate visually the homogeneity and related properties of the material. Visual inspection gives a much more instructive picture than any in situ test would do. In addition, undisturbed block samples of 30 cm edge length were sent to the laboratory for triaxial and other routine testing (Fig. 4.18).

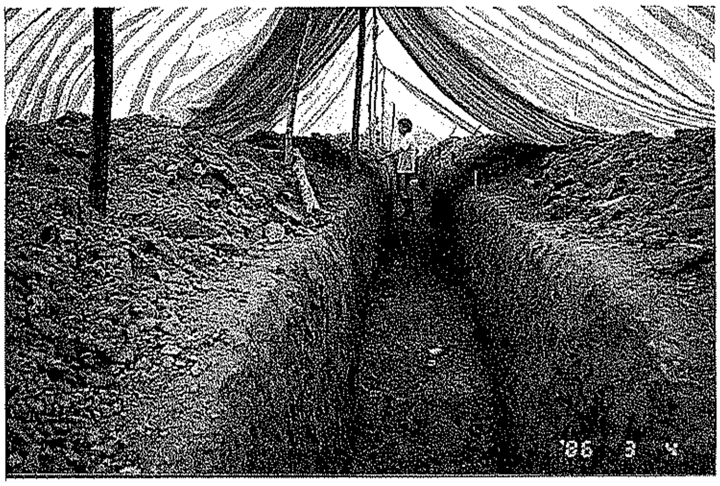


Figure 4.17. Inspection trench across the test field with shelter (courtesy of LI).

Figure 4.18. Undisturbed block sample (courtesy of LI).

A very careful evaluation of the field compaction test yielded the following results:

– Excellent mixing of the original layers was achieved by the selected method, of excavating from bottom to top,

– material no 3 as in Figure 4.16 proved to be the most suitable mixture,

– the content of pebbles and non-disintegrated particles in the compacted layers was small. Such particles were floating in the matrix of soil without contacting each other. In a practical sense the layers were homogeneous. Joints between lifts could not be found,

– the maximum compaction was achieved at 20 cm uncompacted lift thickness and 6 passes of the compactor.

The laboratory tests gave the following parameters: the saturated shear strength (CU-triaxial) corresponds to angles of friction of 30° and 25° at fill water contents of optimum plus 2% and optimum plus 4%, respectively. Cohesion is zero. The compressive strength at UU-triaxial conditions is 300 kPa.

It is frequently requested by the site supervision team to determine the permeability of the compacted soil in the field. A percolation test in a borehole with a stand pipe on top is done for this purpose. Testing can be done with falling or with constant water head in the stand pipe. The latter corresponds to the water pressure test after Lugeon at low pressure (Section 4.2.2.2). The permeability is calculated according to the following formula, which applies to constant water head and isotropic soil:

$$k = \frac{q}{2\pi \cdot L \cdot H} \cdot \ln \frac{L}{r} \tag{4.2}$$

where: q = percolation rate (cm^3/s)
L = length of test section in the borehole (cm)
H = water head (cm)
r = radius of borehole (cm)
$L/r > 10$.

The percolation test and Equation (4.2) are described in many textbooks, but their limitations should be considered: converting the percolation rate into a permeability factor according to Darcy is not 'clean' in the physical sense because Darcy's (1856) law applies to laminar and parallel flow at a hydraulic gradient $i = 1$. The borehole test deviates from such conditions. Usually, the material is not isotropic with respect to the permeability. The test does not enable separate measuring of horizontal and vertical permeabilities.

In addition, the test result depends very much on the homogeneity of the soil in close vicinity of the borehole. That means in practice, that one piece of gravel or similar in a matrix of clay will affect the result significantly. Furthermore, the borehole wall may be either loosened or compacted, due to the drill method and the type of soil. The test period must be very long to enable the water to penetrate a reasonable large body of soil. Given that the permeability is $k = 10^{-7}$ m/s, the travelling radius is 20 cm and the test section is 1 m in length, the test would have to last several days. The water level in the stand pipe would be lowered by about 10 cm per hour in a pipe 10 cm in diameter.

The k-value of 10^{-7} m/s represents the upper limit of an 'impervious' material. The calculation above helps to find an approach to the permeability of the tested soil: with properly compacted cohesive soil of low plasticity the drop in the water level in the stand pipe will be in the range of some centimeters per hour. With medium and highly plastic soils it must be in the range of some millimeters. Apart from this, a percolation test gives valuable information on irregularities due to improper bonds between layers or sandwich-like structure of the embankment with interchanging coarse and fine layers.

In general, the permeability conditions can best be evaluated by inspecting trenches. Percolation tests may help to find irregularities between the trenches.

b) Non-cohesive soils and rockfill material

Usually, a field compaction test prior to completion of tender documents is not required for non-cohesive soils and rockfill materials. There is sufficient experience to predict achievable parameters and so leave it to the contractor to select the compaction machinery and method. Bertram (1973) describes a large field test which was regarded as essential in the early years of rockfill material handling. The test served to investigate the effects of blasting and compacting methods. The test was very similar to tests on cohesive soil (see above) in studying the lift thickness, the achievable unit weight, the grain size composition before and after compaction, different types of compactors, the number of passes and the speed, including the excavation of inspection trenches.

It is common practice to check the suitability of the selected compaction method at the beginning of contractual work. The check is mainly focused on determining the wet and dry unit weights of the compacted material. For this purpose a pit is excavated, and the material is weighed. The volume is determined by the water or sand replacement method. The water replacement method requires sealing of the pit with plastic sheeting; the sand replacement method requires the use of standard sand (e.g. 0.6 to 1.2 mm, US-sieves no 30 and 16, respectively) with known relationship of density and weight. It is useful to fortify the upper edge of the pit by a steel frame.

The relation between the unit weight and the shear strength and modulus should be investigated by laboratory tests on the actual material. So, no other test than the determination of the unit weight is required in the field. The limits of this procedure when applied to rockfill are discussed in Section 9.5.3. Plate load or similar tests are rarely practiced, because of the costs and the interference with construction work. The range of typical material parameters is:

- Dry unit weight 20 to 24 kN/m^3,
- friction angle 38° to 42°, relating to great normal stresses,
- modulus of deformation 50 to 150 MPa.

The following is noted: The unit weight will depend essentially on the gradation and on the specific gravity. Leps (1988b) mentions values of 18.4 kN/m^3 as an average for rockfill with a specific gravity of 26.5 kN/m^3 and a maximum of 24 kN/m^3 for Oroville gravel (Fig. 4.32). The moduli of rockfill material depend largely on the breakdown behaviour. The tendency to break down under handling and compaction increases with decreasing strength of the rock fragments. It should be investigated on graded material by laboratory compaction tests using large containers, e.g. 1.0 m in diameter. The tendency towards particle breakage is particularly related to rock having different strength in saturated and unsaturated conditions. For these, field compaction tests with and without the addition of water are recommended. It is required to compare the grain size compositions before and after compaction and to compare the field results with laboratory strength tests. The need to add water during compaction should be known when prices are calculated.

The permeability of embankments of non-cohesive soils must be high. Rockfills must be essentially free-draining. For field determination of permeability, the limitations should be considered, as described for cohesive soils (Section 4.2.6.2a). In practice, percolation tests cannot be performed in rockfill material because of the drilling difficulties. In sandy gravel the problem is to keep the borehole stable along the test section. In addition, difficulties will arise in supplying the large amount of water required for steady percolation conditions. Free-draining properties of rockfills can be convincingly demonstrated as shown in Figure 4.19. In a free-draining rockfill no water table will develop in the excavated pit.

4.2.6.3 *Field grout test*

In general, designers like to conduct a field grout test on a large scale prior to completion of tender documents. The test is designed to provide information on the most suitable grouting method. Such a test is different from test grouting in single boreholes, described in Section 4.2.2.3, which serves mainly to give information on the groutability of the substrata and the value of grouting. The design and performance of large scale grout tests was discussed in detail by the author (1996).

Provided the test is performed prior to completion of tender documents significant grouting parameters are then available to elaborate more precise documents. Such parameters are the amount of drilling work and the type

Figure 4.19. Field demonstration of free-draining properties of a rockfill (courtesy of LI).

and amount of materials to be grouted. The permissible grouting pressure can be incorporated in the technical specifications.

Frequently, the value of such tests is overestimated. The variety of the test parameters is limited, simply because of the costs and related size of the test field. It cannot be excluded that the geological conditions of the test field do not apply to the whole project. Other conditions at other sections may demand new tests or a modified approach to the grouting work. Accordingly, grouting work may give the impression of being a test from the beginning to the end.

Due to such limitations employers are frequently reluctant to finance a costly field grout test prior to the commencement of contractual work. Such a tendency increases with remote construction sites and tight schedules. According to experience it is the author's opinion that large scale tests prior to the beginning of contractual works can be dispensed with, given the following precondition: the value of grouting in terms of seepage reduction was definitely confirmed by previous test grouting in individual boreholes, or such value can doubtless be derived from other similar projects.

4.3 LABORATORY INVESTIGATIONS

4.3.1 *Cohesive soils*

Cohesive soils are the construction material for homogeneous dams and for

Table 4.5. Laboratory tests to determine the properties of earth and rockfill materials.

No	Property/index	Cohesive soils Test/apparatus	Non-cohesive soils Test/apparatus	Rockfill materials Test/apparatus	Related properties
1	Mineralogical composition	X-ray and chemical analysis	X-ray and chemical analysis	Microscopic examination (TH)	
2	Soluble contents (gypsum etc.)	Divers (D)	Divers (D)	Divers (P)	
3	Organic matters	Ignition (D)	Ignition (D)	–	
4	Gradation	Sieve analysis, sedimentation (D)	Sieve analysis (D)	Sieve analysis (G)	Coefficient of uniformity, liquefaction potential[1], permeability[1]
5	Specific gravity	Flask, pycnometer method (D)	Pycnometer, siphon method (D)	Siphon, suspension method (D)	Pore volume, void ratio
6	Moist/dry unit weight (density)	Weighing and volume determination (U)	Weighing and volume determination (D)	Weighing and volume determination (P, G)	Pore volume, void ratio
7	Water content	Oven drying (U)	–	–	
8	Liquid limit	LL-device (D)	–	–	Plasticity index
9	Plastic limit	Hand-rolling (D)	–	–	Plasticity index
10	Shrinkage limit	SL-device (D)	–	–	
11	Permeability	Flow test (U, D)	Flow test (D)	–	*k*-value after Darcy
12	Water absorption	–	Saturation (D)	Saturation (P)	
13	Optimum moisture, max. dry unit weight	Compaction test (Proctor) (D)	–	–	Compactability
14	Relative density	–	Loosest and most compact state (D)	–	Compactability
15	Consolidation (versus pressure and time)	Oedometer (U)	–	–	Compressibility, stiffness coefficient
16	Compressive strength (uniaxial stress conditions)	Compression test (U, D)	–	Compression test (P)	Failure strength

Table 4.5. Continued.

No	Property/index	Cohesive soils Test/apparatus	Non-cohesive soils Test/apparatus	Rockfill materials Test/apparatus	Related properties
17	Shear strength (uniaxial stress conditions)	Direct shear test (U, D)	Direct shear test (D)	–	Angle of friction, cohesion
18	Deformation (triaxial static stress conditions)	Static triaxial test (U, D)	Static triaxial test (D)	Static triaxial test (G)	Angle of friction, cohesion, pore-water pressure, deformation modulus, Poisson's ratio, dimensionless parameters for FE-computations (Fig. 4.23)
19	Deformation (triaxial dynamic stress conditions)	–	Cyclic triaxial test (D)	–	Pore-water pressure, shear deformation (Figs 4.26 and 8.6)
20	Dynamic behaviour	–	Resonant column test (D)	–	Dynamic properties
21	Abrasion	–	Los Angeles test (D)	Los Angeles test (D)	Abrasion resistance
22	Breakage	–	Not defined (D)	Not defined (G)	
23	Weathering	–	–	Slake durability test (ISRM 1979) (G)	Slake durability index
24	Dispersivity	Emerson crumb-, pinhole-, double hydrometer test (D)	–	–	
25	Swelling	Swelling test (U, D)	–	–	Swelling pressure

(U) = Undisturbed sample
(D) = Disturbed sample
(P) = Piece of rock
(G) = Graded rockfill
(TH) =Thin section of rock
[1]From sieve analysis of non-cohesive soils

the sealing elements of other types of embankment dams. Tests for the classification and for the determination of soil mechanical properties are made on disturbed and undisturbed samples. Usually, the test samples are small in size, because of the fineness of the soils. This facilitates a number of tests – e.g. triaxial tests – and lowers the costs. Coarse particles which are floating in the matrix of the fines without contacting each other do not significantly affect the properties. For testing they are removed from the sample. The size of the coarse particles which are removed depends on the sample size and on the type of test. With a number of tests, e.g. the determination of the plasticity index, particles over 0.4 mm in diameter are removed (US-sieve no 40).

The main tests are listed in Table 4.5. These are tests for soil classification and for determining mechanical and hydraulic properties. For test details

Table 4.6. Example of testing programme on the core material of a 100 m high rockfill dam with earth core. Dam length about 1000 m, favourable conditions as in Table 4.2.

Test/property	Number of tests/determinations
Mineralogical composition	3
Determination of soluble contents	5
Determination of organic matters	5
Gradation	40
Specific gravity	5
Moist/dry unit weight	15
Natural water content	40
Liquid limit	40
Plastic limit	40
Shrinkage limit	5
Permeability:	
– 100% Proctor density, 2 gradients	5 × 2
– 95% Proctor density, 2 gradients	5 × 2
Water absorption potential	3[1]
Compaction test (Proctor)	20
CD triaxial test:	
– 3 confining pressures σ_3	3 × 3 individual tests
– max. confining pressure σ_3	2 × 1 individual test[2]
CU triaxial test:	
– 3 confining pressures σ_3	3 × 3 individual tests
– max. confining pressure σ_3	2 × 1 individual test[2]
UU triaxial test:	
– max. confining pressure σ_3	2 × 1 individual test
Dispersivity	4[1]
Swelling potential	2[1]

[1]Non-dispersive, non-swelling soils, otherwise more tests required
[2]Tests to round off scattering results

refer to respective textbooks (for instance Schultze & Muhs 1967, USBR 1974). Table 4.6 gives a tentative programme of tests to be made on the core material of a 100 m high rockfill dam with earth core, 1000 m in length. This programme corresponds to the example of Table 4.2 and the assumption that the total amount of material is of a common type and is available from only one borrow area. Tests on undisturbed samples taken from a test embankment, as in Figure 4.18, are not included.

4.3.1.1 *Mineralogical composition, soluble and organic contents*

The mineralogical composition reflects the nature of the material in general. The content of a number of distinct minerals points to particular properties which require other than the common tests. This applies, for instance, to montmorillonite, with respect to the dispersivity and the swelling potential.

The permissible content of soluble matters is small. Therefore, the content of soluble lime, gypsum and others must be determined, if appropriate. This also refers to organic matter which might disintegrate, hence leading to undesired post-construction deformations of the embankment.

4.3.1.2 *Grain size distribution and classification*

The grain size distribution, the liquid limit and the plasticity index allow us to classify cohesive soils according to the Unified Soil Classification System which has, in practice, proven to be an excellent tool. Extracts are shown in Figure 4.20 and Table 4.7. It was developed on the base of Casagrande's studies (1948) and adopted in 1952 by USBR and USCE. The system also includes coarse materials (Table 4.7). In addition to mere classification, it enables experienced geotechnicians to estimate the range of the mechanical properties and of the permeability of soils.

Cohesive soils are classified according to the diagram shown in Figure

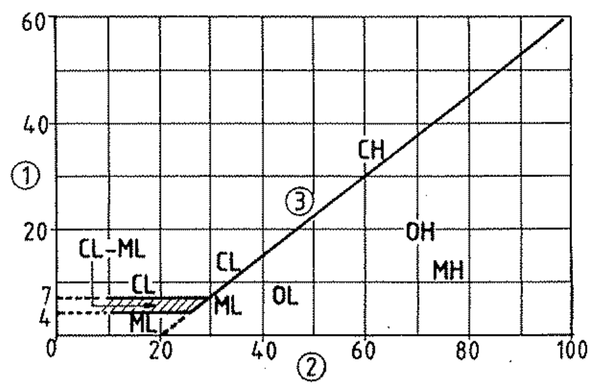


Figure 4.20. Plasticity chart (extract of the Unified Soil Classification Chart, USBR 1974).

1 Plasticity index
2 Liquid limit (%)
3 A-line
(For further information see Table 4.7).

Table 4.7. Group symbols and typical names of soils (extract of the Unified Soil Classification Chart, USBR 1974).

Group symbols	Typical names and laboratory classification criteria
GW	Well graded gravels, gravel-sand mixtures; little or no fines. C_U greater than 4
GP	Poorly graded gravels, gravel-sand mixtures; little or no fines. Not meeting all gradation requirements for GW
GM	Silty gravels, poorly graded gravel-sand-silt mixtures. Atterberg limits below A-line, or PI less than 4
GC	Clayey gravels, poorly graded gravel-sand-clay mixtures. Atterberg limits above A-line with PI greater than 7
SW	Well graded sands, gravelly sands; little or no fines. C_U greater than 6
SP	Poorly graded sands, gravelly sands; little or no fines. Not meeting all gradation requirements for SW
SM	Silty sands, poorly graded sand-silt mixtures. Atterberg limits below A-line, or PI less than 4.
SC	Clayey sands, poorly graded sand-clay mixtures. Atterberg limits above A-line with PI greater than 7
ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity
CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
OL	Organic silts and organic silt-clays of low plasticity
MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, plastic silts
CH	Inorganic clays of high plasticity, fat clays
OH	Organic clays of medium to high plasticity
PT	Peat and other highly organic soils

Atterberg limits = liquid limit (LL) and plastic limit (PL).
PI = Plasticity index (LL minus PL).
C_U = Coefficient of uniformity d_{60}/d_{10}.

4.20. The A-line separates clays from silts. Only organic clays OL and OH are found on the silt side (below the A-line). They are not relevant in dam engineering, as they are not suitable as construction materials.

The Unified Soil Classification System is used all over the world. It includes 15 group symbols. The number of symbols can be extended using double symbols like SC-PL, SP-SC, CL-ML. The system covers all types of soils which are suitable in dam engineering.

4.3.1.3 *Compaction*

It is the water content which controls the compactability of cohesive soils and hence the need to moisten or to dry them out before being incorporated in an embankment. In the laboratory a tamper with a drop guide is used for compaction (Proctor 1933). Table 4.8 gives the details of the equipment and the compaction conditions to achieve the 'standard' and the 'modified' Proctor density. The compaction test gives the relationship between the water content of the soil and the achievable dry unit weight at a defined compacting effort (Fig. 4.21).The optimum water content related to the

Table 4.8. Compaction test after Proctor. Equipment and details.

		Standard Proctor density			Modified Proctor density		
Weight of sample	(kg)	≥ 2	≥ 9	≥ 40	≥ 2	≥ 9	≥ 40
Diameter of cylinder	(mm)	100	150	250	100	150	250
Height of cylinder	(mm)	120	125	200	120	125	200
Max. grain size	(mm)	20	31.5	63	20	31.5	63
Diameter of tamper	(mm)	50	75	125	50	75	125
Weight of tamper	(kg)	2.5	4.5	15	4.5	4.5	15
Drop height	(mm)	300	450	600	450	450	600
Number of layers		3	3	3	5	5	5
Number of blows per layer		25	22	22	25	59	59
Compaction effort per m^3	(mkg)		60,000			270,000	

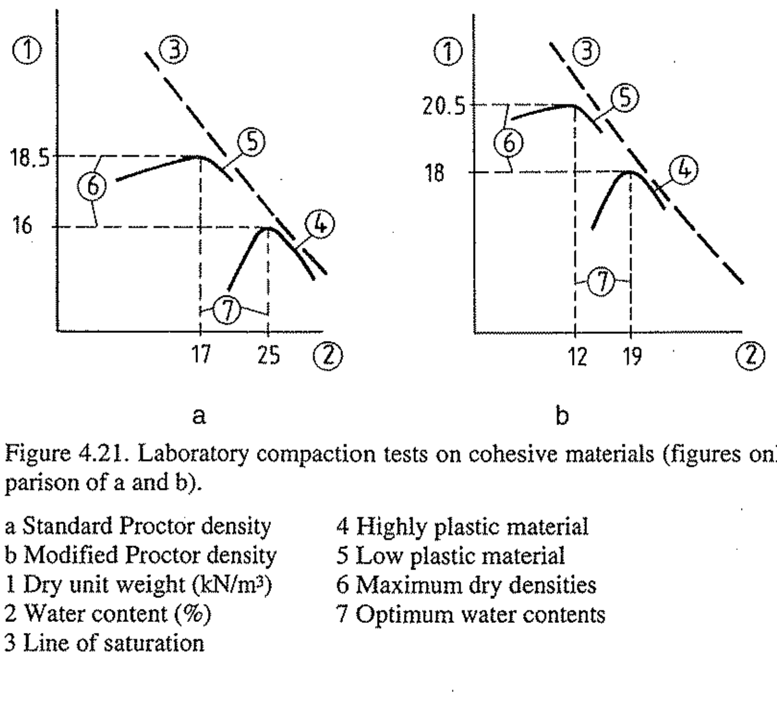


Figure 4.21. Laboratory compaction tests on cohesive materials (figures only as a comparison of a and b).

a Standard Proctor density
b Modified Proctor density
1 Dry unit weight (kN/m^3)
2 Water content (%)
3 Line of saturation
4 Highly plastic material
5 Low plastic material
6 Maximum dry densities
7 Optimum water contents

modified test is less than that related to the standard test, and the maximum dry density related to the modified test is higher than that related to the standard test. The laboratory compaction test is the basis for the technical specifications for field compaction, and for quality control of the readily compacted cohesive materials in the embankment.

Usually, the modified density is achieved with the use of up-to-date machinery. In spite of this the standard density is frequently used as a criterion. Densities of more than 100% standard density of the soil can often be achieved by common field compaction. The preferred reference to the standard density relates to the desirable water content of the fill 'on the wet side' (see Sections 7.3.2.1 and 9.4.1).

4.3.1.4 *Deformation*

The deformation behaviour of cohesive soils is investigated mainly by triaxial compression tests. The results provide the input for all computations of stability, stresses and strains. They are used to design the dam zoning and the geometry of the sealing element. Examples of material data are given in Table 4.10. Other typical material properties have been compiled by Idriss & Duncan (1988). These data are derived from several hundred triaxial tests on more than 50 types of soil, cohesive and non-cohesive.

Usually, triaxial stress conditions exist in the dam body. Therefore, triax-

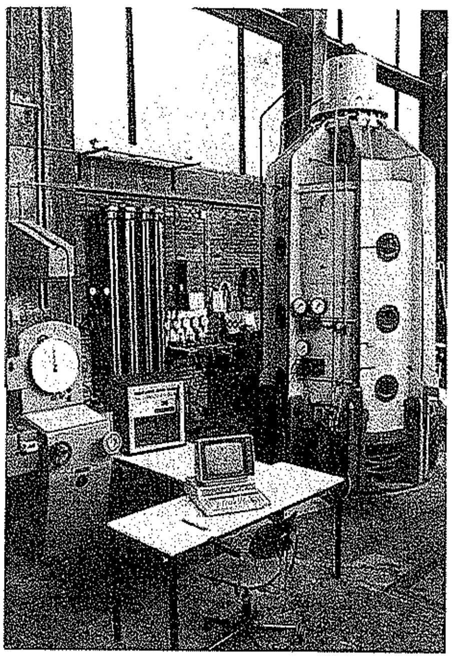


Figure 4.22. Triaxial cell and accessories (courtesy of IBF).

Left: Control panel for stress and strain dependent test procedures
Center: Pipes and pressure gauges for pore-water and cell pressure
Right: Triaxial cell

ial testing is of great importance. Due to advanced techniques of triaxial testing, developed in the last decades, uniaxial tests have lost ground.

The diameter of cohesive soil samples for triaxial tests is about 36 mm. Larger samples need tests which are more time-consuming and more costly. Up-to-date triaxial equipment for large samples is shown in Figure 4.22. The load increase is stress or feed controlled. All data measured are automatically printed out. The data can be processed by computer and be graphically plotted (Fig. 4.23).

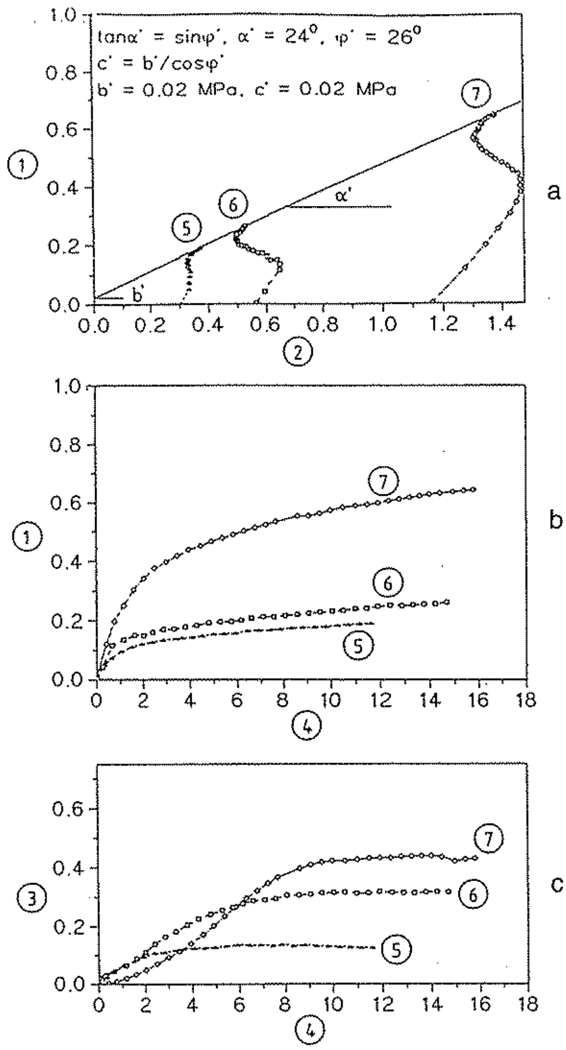


Figure 4.23. Plots of CU-triaxial tests on clayey sandy silt (courtesy of IBF).

a Stress paths
b Axial strain
c Pore-water pressure
1 Deviator stress $(\sigma_1 - \sigma_3)/2$ (MPa)
2 Mean principal stress $(\sigma_1 + \sigma_3)/2$ (MPa)
3 Pore-water pressure du (MPa)
4 Axial strain dl/l (%)
5 $\sigma_3 = 0.3$ MPa
6 $\sigma_3 = 0.6$ MPa
7 $\sigma_3 = 1.2$ MPa

The triaxial tests must cover the stress conditions of all load cases. Therefore, the following tests have to be distinguished:

– CD-tests (consolidated, drained) which are relevant for the load cases of reservoir operation,

– CU-tests (consolidated, undrained) which are relevant for the load case 'end of construction',

– UU-tests (unconsolidated, undrained) which reflect the conditions of compacted layers before the consolidation starts.

Consolidated conditions are established, prior to the increase of the normal stress σ_1, by equal vertical and confining pressures ($\sigma_1 = \sigma_2 = \sigma_3$). Drained and undrained conditions are established by allowing the pore water to escape or not to escape, respectively, through the end sections of the sample. So, the U-tests lead to the development of excess pore-water pressure in the sample, and related loss of shear strength. With U-tests the samples remain constant in volume. Volume changes of medium dense or loose samples apply to CD-tests. The confining pressures $\sigma_2 = \sigma_3$ should be selected so as to comply with the actual stresses in the dam body. Accordingly, confining pressures of about 0.3, 0.6 and 1.2 MPa make a full set of triaxial tests for a dam 100 m in height (the rate of lateral pressure taken as 0.6).

In the load case 'end of construction' the lower portion of the dam core reflects CU conditions, and the upper portion UU conditions. The location of the borderline depends on the geometry of the core, on the consolidation properties of the material and on progress in constructing the core.

4.3.1.5 *Permeability*

The permeability k after Darcy is determined in the course of compaction or triaxial tests. Necessarily, it is a key-date with respect to the suitability of a given soil for sealing purposes. In an embankment dam there exist different permeabilities in vertical and horizontal directions. In the laboratory, in most cases the vertical permeability is determined, i.e. the permeability related to the direction of compacting the soil with the Proctor test or related to the direction of the vertical principal stress with the triaxial test.

The horizontal permeability of an embankment consisting of individually compacted layers can be assessed by percolation tests (Section 4.2.6.2a).

4.3.1.6 *Dispersivity*

Dispersivity is a special property of some types of clay. Dispersive clays disperse or deflocculate in the presence of water which becomes turbid with the progress of dispersion. Flowing water carries the dispersed particles away. It is called colloidal erosion. If the velocity of the water is low the swelling potential of the clay may block the seepage paths and prevent colloidal erosion. The origin of dispersivity is seen in the predominance of sodium cations in the pore water while calcium or magnesium cations prevail in the pore water of non-dispersive clays (Sherard et al. 1976b).

Indications of the presence of dispersive soils can be seen in the field: deep erosion gullies, features of piping, turbid water after rain, and others. Also geological indications are known: clays of alluvial origin, soils derived from shale and claystone laid down in a marine environment, and others (Fell et al. 1992). Such indications, however, are not sufficient for the identification of dispersivity.

Five generally accepted tests have been developed for the laboratory identification (Knodel 1988, Fell et al. 1992). The most applied tests are:

– The Emerson crumb test (SAA 1980),

– the pinhole test (Sherard et al. 1976a, b),

– the double hydrometer test (developed from a previous procedure after Volk 1937).

The crumb test uses cubical crumbs of the soil placed in distilled water to observe eight classes of dispersivity according to the turbidity of the water after several time intervals. Class 1 relates to highly dispersive soils and Class 8 to non-dispersive soils. The test does not provide a measure of erodibility. It is useful as a first check on dispersivity (Fell et al. 1992).

The pinhole test uses a 25 mm thick sample of clay with a 1 mm hole. Water flowing through the hole at different velocities becomes turbid and erodes the hole at a sufficiently high velocity. Six classes of dispersivity and erodibility are distinguished according to the turbidity of the water, the diameter of the hole at the end of the test and the discharge of water. The first class D1 is related to highly dispersive soils and the two last classes ND2 and ND1 to non-dispersive and completely erosion resistant soils (Sherard et al. 1976a, b, Fell et al. 1992).

The double hydrometer test consists of a standard hydrometer analysis of the soil and a second hydrometer analysis made in parallel without mechanical agitation and without the use of a chemical dispersant when the soil is dispersed in distilled water for testing. The measure of dispersivity is the ratio of particles > 0.005 mm of the parallel test to the standard test, expressed as a percentage. For typical dispersive soils it may be 30% (Knodel 1988).

The dispersivity observed in the tests depends on the density of the soil and on the chemistry of the water. There are limits to the reliability and accuracy of the test results. So it is recommended to apply more than one type of test and to use distilled and non-distilled water.

As a protective measure against colloidal erosion the dispersive clay can be modified by the addition of lime (Thomas 1976). According to research, well designed filters are also able to prevent this type of erosion.

4.3.2 *Non-cohesive soils*

Non-cohesive soils are the construction material for dam shells and for filters. Tests for property determinations are made essentially on disturbed samples because undisturbed sampling is almost impossible. Furthermore, the properties of non-cohesive soils are always changed when the soil is excavated.

The size of the test samples is related to their maximum particle size. As a rule of thumb, the sample diameter of a well graded soil should be about five times the maximum particle diameter, and the sample diameter of a poorly graded soil should be eight to ten times the maximum particle diameter. Accordingly, a sandy gravel up to 60 mm grain size needs sample diameters of 30 to 50 cm. Respective tests are listed in Table 4.5. For test details refer to the known manuals.

In addition to common testing under static loads it is necessary to determine the deformation behaviour of non-cohesive soils under dynamic loads. This applies mainly to sand and non-cohesive silt, which require special attention in dam engineering because of the risk of liquefaction.

4.3.2.1 *Mineralogical composition, soluble and organic contents*

The mineralogical composition is an indication of the nature of the material in general. The material resistance to mechanical attack is related to the mineral hardness. Soluble contents, such as lime and gypsum, must be identified and evaluated in respect of being harmful or not. Any organic content must be close to zero. Wooden material and roots might lead to post-construction settlement.

4.3.2.2 *Grain size distribution and permeability*

Non-cohesive materials are classified according to the grain size distribution (Table. 4.7). The permeability is related to it as follows:

$$k = C \times (d_{10})^2 \tag{4.3}$$

where: k = permeability
C = factor related to the uniformity d_{60}/d_{10}
d_{10} = particle size for which 10% is finer (effective grain size).

Equation (4.3) was initially given by Hazen (1892) for poorly graded sand. It was confirmed by tests on well graded sandy gravel (Beyer 1964). The resulting relation between factor C and the uniformity is shown in Figure 4.24. Fell et al. (1992) recommend that the use of Equation (4.3) 'should be limited to initial estimates of permeability in non-critical cases'. More accurate values of the permeability can be obtained from tests in the compaction

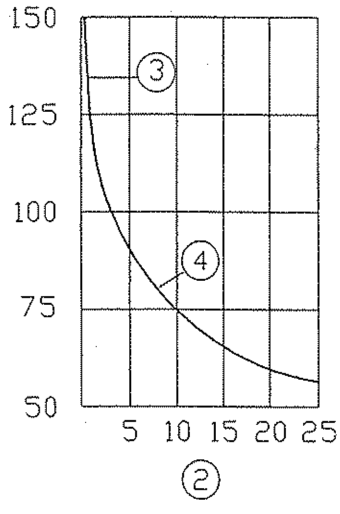


Figure 4.24. Factor C of Equation (4.3) versus coefficient of uniformity of sand and sandy gravel.

1 Factor C of Equation (4.3): $k = C \times (d_{10})^2$
2 Coefficient of uniformity $U = d_{60}/d_{10}$
3 After Hazen (1892) for $U \leq 5$
4 After Beyer (1964) for $U > 5$

mold or in the triaxial cell, which regard also the effects of the particle shape and the sample density.

4.3.2.3 *Compaction*

The state of density of non-cohesive soils is defined as relative density D using the loosest and the most compact states of the soil:

$$D = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100\ (\%) \tag{4.4a}$$

where: e_{max} = void ratio of the soil in loosest state
e = void ratio of the soil in place
e_{min} = void ratio of the soil in most compact state.

The relative density D can as well be expressed by:

$$D = \frac{\gamma_{max}(\gamma - \gamma_{min})}{\gamma(\gamma_{max} - \gamma_{min})} \times 100\ (\%) \tag{4.4b}$$

where γ_{max}, γ, γ_{min} are the respective unit weights of the soil (USBR 1974). In Europe the relative density is defined also using the void volumes n_{max}, n and n_{min} according to Equation (4.4a). It is noted that the two densities using e and n differ by 5 to 10%.

The data of the loosest state are obtained by pouring the ovendried soil in a steady stream from a scoop or a spout into a cylindrical unit weight measure and then calculating e_{max} or γ_{min}, respectively. The method is somewhat dependent on the personnel and on the device, that means on the scoop and spout used for a soil with a given maximum grain size, and on the method of pouring.

The data for the most compact state are obtained by shaking the soil specimen in the unit weight measure for some minutes on a vibratory table. The unit weight measure is fixed to the table, and the soil is loaded by a surcharge weight. The method and the details are regarded as accurate by USBR (1974) to obtain reproduceable data. When Equation (4.4b) is applied the dry unit weight as well as the wet (saturated) unit weight may be established by the test.

It is mentioned that similar data from the vibratory test have been defined also in Europe (Leussink & Kutzner 1962), but, the test using a vibratory table to obtain the most compact state is not always applied, in favour of a less accurate method of compacting the soil by using a hand-operated hammering device.

From non-cohesive soils there is no equivalent to the maximum dry density of cohesive soils. The typical relation between the achievable density and the water content, as in Figure 4.21, does not exist because of the higher permeability of non-cohesive soils, enabling the pore water to escape during compaction. The lines of water content versus density would rather be horizontal than curved. Only sand would show such a curve when being compacted at low effort. The curve reflects the effect of an apparent cohesion which has its maximum at a low water content and which is zero at dry and saturated conditions. In practice, the apparent cohesion does not effect the compactability due to the strong impact of heavy machinery.

4.3.2.4 *Deformation and liquefaction*

The deformation behaviour is investigated by CD-triaxial tests under static loading, which corresponds with CD-tests on cohesive soils. Greater samples, about 30 to 50 cm in diameter, are used. The determination of the appropriate sample size and the preparation of samples are described in Section 4.3.3. Because of the prevailing triaxial stress conditions the uniaxial (frame) shear test is less important. Such tests may be applied in simple cases to determine the shear parameters, not the deformation parameters.

Special attention has to be paid to the deformation behaviour of saturated non-cohesive soils under dynamic loading, which may lead to the development of excess pore-water pressure and related loss of shear strength, i.e. to liquefaction (Fig. 4.25). When the soil is partly saturated the pore water can escape to the air filled pores, yielding diminished pore-water pressure and loss of strength.

The liquefaction potential is investigated by cyclic triaxial tests and by resonant column tests. The typical variety of amplitudes and accelerations

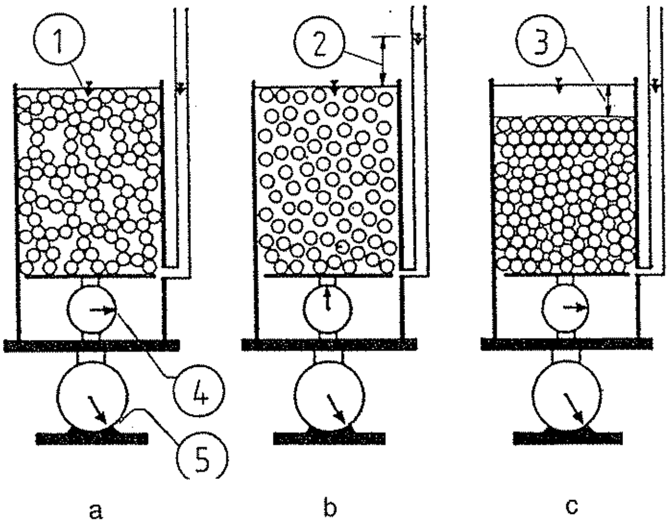


Figure 4.25. Liquefaction of sand (adapted from Ishihara 1985).

a Initial conditions	2 Excess pore-water pressure
b Liquefaction	3 Settlement
c Final conditions	4 Effective vertical stress
1 Water surface	5 Total vertical stress

related to an earthquake in nature (Fig. 6.9) is replaced in the tests by a cyclic loading as shown in Figure 4.26a. Under the cyclic load pore-water pressures develop in the saturated sample, reaching a maximum after a small number of cycles, and being constant at the maximum level during further cyclic loading (Fig. 4.26b). The shear deformation increases continuously (Fig. 4.26c). Under unfavourable conditions the increase may be overproportional to the number of cycles. A rate of 5% shear deformation is an agreed criterion of failure.

An overview of soils prone to liquefaction is shown in Figure 4.27. Most endangered soils are loose, poorly graded fine sands and non-cohesive silts. With such soils, in a loose state, the dynamic load causes compaction which is demonstrated also in Figure 4.25. The low permeability of the soils prevents the pore water from rapid escape. Therefore, pore pressure develops under the effect of compaction. The risk of liquefaction decreases with increasing density of the soils.

The risk of liquefaction decreases also with an increasing plasticity index of the fines, i.e. with increasing cohesion, because cohesion prevents the particle-to-particle pressure from being reduced. This complies with the fact

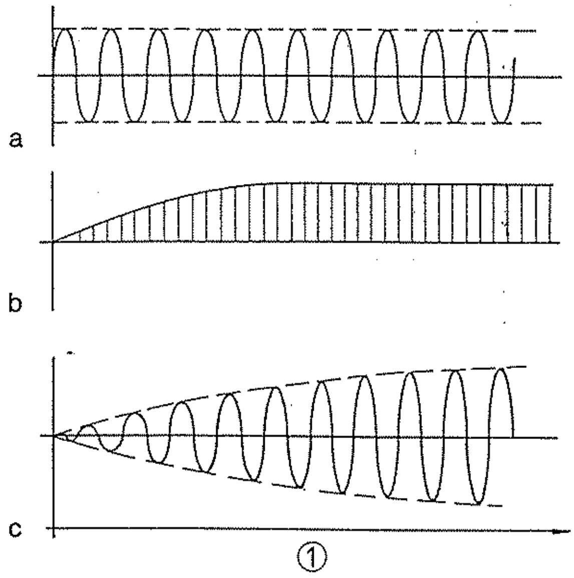


Figure 4.26. Cyclic triaxial test on saturated sand.

a Cyclic shear stress
b Excess pore-water pressure
c Shear strain
1 Number of cycles

that the soil must be loosened before compaction under dynamic loading is effective. Cohesion counteracts the loosening effect.

The lower limit of liquefiable soils shown in Figure 4.27 is seen as conservative. It includes soils with some content of clay. Apart from this it coincides approximately with a lower boundary for 'potentially liquefiable soils' given by Tsuchida (1970). It is noted by Ishihara (1985) that the boundary is floating due to the range of cohesion of the fines.

Given coarser soils permeability increases rapidly, and hence the risk of liquefaction decreases. Again, the boundary is floating depending on the permeability and the density of the soil. After Tsuchida the upper boundary for 'potentially liquefiable soils' coincides with a very poorly graded gravelly sand with $d_{10} \approx 1.0$ mm and $d_{60} \approx 2.0$ mm. The permeability k of this soil is about 10^{-2} m/s. According to this boundary only poorly graded soils would be liquefiable. This is not confirmed by tests on the well graded slightly silty and gravelly sand from Chico (no 10 in Figure 4.27). This soil showed pore-water pressure after a few cycles of a cyclic triaxial test. Therefore, the author prefers to attach an upper boundary for liquefiable soils to poorly graded sands and to well graded sandy gravel with $d_{10} \approx 0.4$ mm and $k \leq 10^{-3}$ m/s, as shown in Figure 4.27.

In addition, the liquefaction potential depends on the stress conditions of the soil. In the test, the number of cycles required for liquefaction and critical deformation increases with increasing confining pressures (Fig. 8.6). In

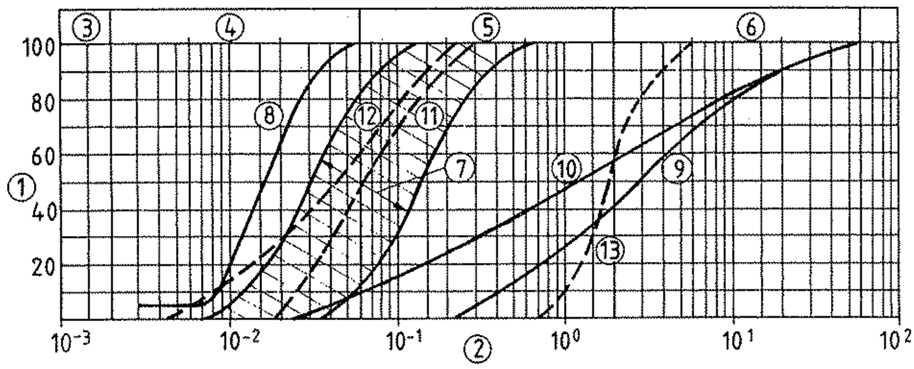


Figure 4.27. Grain size distributions of soils prone to liquefaction.

1 Percent finer by weight
2 Grain size (mm)
3 Clay
4 Silt
5 Sand
6 Gravel
7 Range of soils highly susceptible to liquefaction
8 Conservatively assumed lower boundary
9 Conservatively assumed upper boundary
10 Example of slightly silty sand-gravel (Chico)
11 Example of silt and fine sand (Iran, Breth 1976)
12 Example of silt and fine sand (river deposit Kinda)
13 Upper boundary for potentially liquefiable soils (Tsuchida 1970)

the field, this effect prevents sand layers at a greater depth from being liquefied.

The cyclic details of the tests are of minor importance. The wide range of frequencies and amplitudes of earthquakes exceeds the limits which are given in testing. In a practical sense it is sufficient to confirm qualitatively the tendency towards liquefaction in order to classify non-cohesive soils, when saturated, as suitable or unsuitable for dam construction.

4.3.2.5 *Water absorption potential and abrasion resistance*

The water absorption potential of rock is related to its pore volume. It is about 1% with strong and durable rock. Higher pore volumes indicate reduced strength which is again related to the degree of saturation. In an embankment such material may lead to post-construction settlements after impounding. These settlements due to saturation must be minimized by appropriate construction methods, for instance by addition of water while the material is being compacted.

The abrasion resistance of sand and gravel is an indicator of durability and resistance to mechanical attack. The percentage of abrasive and friable material is determined by an abrasion test. Such test and its equipment is defined by USBR (1966) and ISRM (1977), known as the 'Los Angeles test'. The friable material will usually disintegrate during compaction. Disintegration after construction, for instance due to consolidation, stress variation or weathering, would lead to undesirable post-construction settlements. The permeability might also be reduced by such a process which is also undesirable.

4.3.3 *Rockfill material*

Rockfill material is suitable for dam shells, for riprap as a wave protection and for selected zones of all types of dams. Alluvial gravel is an excellent 'rockfill', but usually rockfill material has to be quarried. It consists – ideally – of strong and durable rock fragments in the size of gravel and cobbles.

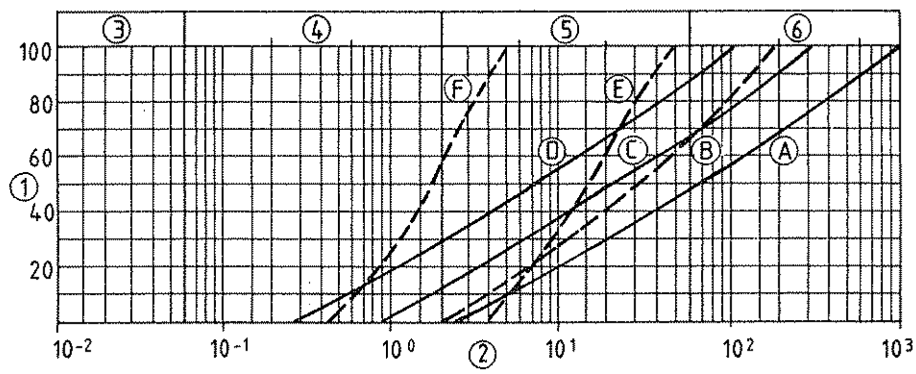


Figure 4.28. Grain size distributions of processed rockfill materials, appropriate for triaxial testing (see Table 4.9).

1 Percent finer by weight
2 Grain size (mm)
3 Silt
4 Sand
5 Gravel
6 Cobbles
A Well graded rockfill material, $U > 5$, permeability k_A
B Material A, 30% scalped at 200 mm
C Modelled material A, permeability $k_A/10$
D Modelled material A, permeability $k_A/100$
E Poorly graded gravel, $U < 5$, permeability k_E
F Modelled material E, permeability $k_E/100$

The material must be well graded. An excellent material for dam construction has the following grain size distribution (material A in Fig. 4.28):

– Not more than 5% below 5 mm,

– not more than 30% below 20 mm,

– maximum particle size 600 to 1000 mm, depending on the rock strength and the tendency towards particle breakage.

Numerous dams have, of course, been built of rockfill material with other than such 'ideal' properties. Again it is a challenge for the designer to adjust his design to the available materials and to make use of them with the aim of achieving safety over the lifetime of the structure. Many examples of different types of rockfill embankments are discussed in this book.

The permeability of competent rockfill material is at least 10^{-2} m/s. It is free-draining and, therefore, no problems arise from earthquakes when the material is saturated. The shear strength is high. It may be reduced by breakdown at high stresses and due to latent fractures in the rock fabric. As to limitations of the permeability and the homogeneity of rockfill embankments see Section 9.4.3.

Materials of that size can hardly be handled in the laboratory. Only individual pieces of rock can be tested. The grain size distribution of graded rockfill material as a whole must be simulated by particle mixtures which are reasonable in size for testing. The maximum particle size must be limited to about 1/5 of the sample diameter of well graded material and about 1/10 of the sample diameter of poorly graded material. There are two methods of producing graded materials appropriate for testing (Fig. 4.28 and Table 4.9):

1st Method: The material is scalped at 1/5 or 1/10 of the sample diameter, respectively (material B in Fig. 4.28). The mechanical properties of the scalped material will not differ significantly from those of the original material, provided the scalped part does not exceed 25 to 30%. This is because the large pieces, screened out, are floating in the total mix without contacting each other. It was repeatedly confirmed by tests, e.g. made by Leussink et al. (1964) on mixed soils.

The shear strength of the scalped material may be slightly reduced if the material which is screened out amounts to more than 30%. This is because the pore volume of the scalped material is slightly increased in comparison to the original material. This tendency was confirmed e.g. by Idel (1960) for material made of spheres of equal diameter and by Marsal (1973) for sandy gravel with 200 mm maximum grain size. With natural materials this tendency will be superimposed by the effects of break down and crushing of particle edges which applies mainly to the large pieces (see below).

2nd Method: A new material is produced, reduced in size, with the grain size distribution being parallel to that of the original material (materials C and D in Fig. 4.28). The newly modelled material must consist of the same

Table 4.9. Processed rockfill material, appropriate for testing (compare Fig. 4.28).

Material to be tested	d_{max} (mm)	d_{60} (mm)	d_{30} (mm)	d_{10} (mm)	C_U[1] (mm)	C[2] (mm)	k[3] (mm)	Diameter of sample for further tests (mm)
A Rockfill material, well graded, $k = k_A$	1000	120	20	5	24	60	1.5×10^{-1}	–
B Scalped material A, 30% scalped from A	200	50	12	4	12.5	70	1.0×10^{-1}	1000
C Modelled material A, $k = k_A/10$	300	40	6	1.7	24	60	1.5×10^{-2}	1000[4] 500[5]
D Modelled material A, $k = k_A/100$	100	12	2	0.5	24	60	1.5×10^{-3}	500 300[6]
E Gravel, poorly graded, $k = k_E$	50	20	9	5	4	100	2.5×10^{-1}	500
F Modelled gravel E, $k = k_E/100$	5	2	≈ 1	0.5	4	100	2.5×10^{-3}	>50 100[7]

[1] $C_U = d_{60}/d_{10}$
[2] According to Figure 4.24
[3] According to Equation (4.3)
[4] After scalping of about 10% at 200 mm
[5] After scalping of about 20% at 100 mm
[6] After scalping of about 15% at 50 mm
[7] Recommended minimum

rock as the original because the shear strength of the graded material is considerably affected by the friction at the points of particle-to-particle contact. This was confirmed by tests (Idel 1960). With regard to permeability, it is advisable to limit the reduction in size to about $d_{10} \approx 0.5$ mm. At $d_{10} < 0.5$ mm problems of pore-water pressure development may arise (Fig. 4.27).

The strongest effect on the shear strength and deformation modulus of the modelled material is attributed to the crushing of particle edges. Usually, this type of breakdown applies to the larger pieces which are, at least partly, screened out. Therefore, the shear strength and the deformation modulus of the modelled material tend to increased values. This tendency was confirmed by tests on various rockfill materials, for instance on the Oroville material no 9 in Figure 4.32 (Marachi et al. 1972).

The laboratory tests relevant for rockfill materials are listed in Table 4.5. These tests refer to individual pieces of rock and to graded materials. The compressive strength is related to the mineral hardness and to the water absorption potential. The strength must be determined for dry and saturated samples. Such information is needed to decide on the dumping and compacting method in the embankment, with or without the addition of water.

The deformation behaviour is determined by static triaxial tests. Usually, the samples are saturated. Such tests provide the shear parameters and the stress-strain diagrams to develop the input data for FE-computations. Some data are listed in Table 4.10. More information can be taken from Leps (1988b).

Since about 1960 large triaxial cells have been used for testing samples 1.0 m in diameter (Fig. 4.29). Confining pressures amount up to 2.5 MPa (Marsal 1973). The smallest recommendable sample diameter of modelled rockfill material is 500 mm (Table 4.9), to obtain sufficiently accurate test results. Modelling to smaller sizes might exclude the effect of particle edge crushing at high stresses, leading to increased shear strength of the modelled material in comparison to the original material.

Table 4.11 gives a tentative programme of tests to be done on the shell material of a 100 m high rockfill dam, 1000 m in length. Again this pro-

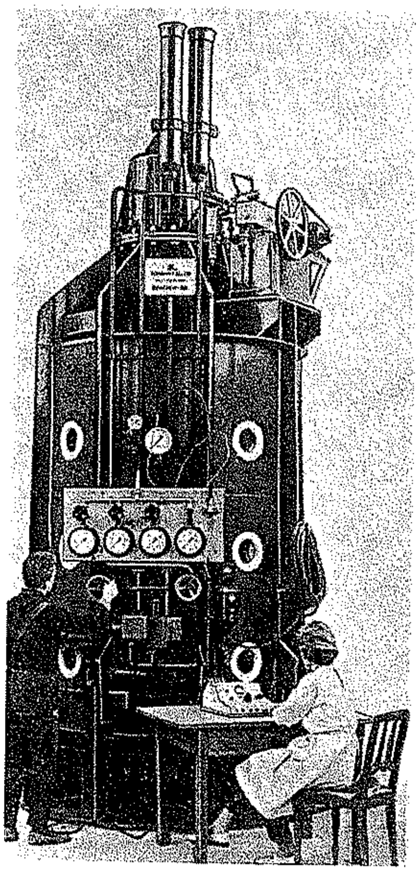


Figure 4.29. Large triaxial cell, sample diameter 1.0 m (Leussink 1960).

Table 4.10. Properties of construction materials.

No	Rock/soil	Weight (kN/m³)	C_U[1]	d_{max} (mm)	< 0.06 (%)	Friction (°)	Deduction (°)[3]	Cohesion (kPa)	Dimensionless parameters R_1	K	n	K_0	m	ν[2]	Remarks (Reference)
1	Siltstone/ sandstone	21.5	33	60	10	40	6	0	0.7	600	0.5	200	0.4	0.15	No 12 Fig. 4.32, Chico (2)
2	Andesite/ diorite	22.5	8	600	–	48	8	0	0.7	800	0.4	400	0.25	0.24	Chico (2)
3	Andesite	22	12	1000	–	42	8	0	0.75	290	0.35	140	0.35	0.15	No 11 Fig. 7.39, Kinda (3)
4	Grano-diorite	22		700	≤ 5	48		0	0.42	1300	0.0				Table 7.10 Finstertal (1)
5	Greywacke/ siltstone	22		600		42		0	0.92	1200	0.27				Grosse Dhünn (1)
6	Amphibolite, rounded	21	> 50	150	–	46	5	0	0.75	1780	0.25				No 9 Fig. 4.32, Oroville (4)
7	Gravel	21	10	150		35	4	0	0.75	175	0.35	110	0.35	0.23	No 10 Fig. 7.39, Kinda (3)
8	Moraine	23.2		100		42		0	0.44	1000	0.45				Table 7.10 Finstertal (1)
9	Silt, sandy, clayey	16.5	> 50	30	20	30	0	0	0.76	220	0.6	70	0.5	0.26	Chico (2)
10	Silt-sand, clayey	20	> 50	25	20	17	0	50	0.75	135	0.35	135	0.35	0.33	No 8 Fig. 7.39, Kinda (3)

References:
1 Breth & Arslan (1990)
2 Kutzner & Hönisch (1985)
3 Kutzner et al. (1988)
4 Marachi et al. (1972)

[1] $C_U = d_{60}/d_{10}$
[2] ν = Poisson's ratio
[3] Logarithmic deduction of $d\sigma_3$

Table 4.11. Example of testing programme on the shell material of a 100 m high rockfill dam with earth core. Dam length about 1000 m, favourable conditions as in Table 4.2.

Test/property	Number of tests/determinations
Mineralogical composition	5
Determination of soluble contents	5
Gradation	5
Specific gravity	5
Water absorption potential	15
Uniaxial compression test (drill cores)	15
Point load test	30
CD triaxial test (diameter ≥ 50 cm):	
– 3 confining pressures σ_3	3 × 3 individual tests
– max. confining pressure σ_3	2 × 1 individual test[1]
Abrasion resistance	5[2]
Slake durability test	3[2]

[1]Tests to round off scattering results
[2]Strong rock, strength > 100 MPa, otherwise more tests required

gramme corresponds with Tables 4.2 and 4.6 and the assumption that the total amount of material is of a common nature and is available from only one borrow area.

Frequently, 'dirty' rockfill material is available instead of material consisting exclusively of strong and durable rock fragments. Dirty material may contain also sand, silt and clay. Such mixed material is not free draining. The shear strength is not as high as that of pure rockfill material. Mixed materials are also used for dam construction, given consideration of problems related to pore-water pressure development with dynamic loading. Typical examples are materials of sandstone, siltstone, mudstone and schist, weathered to some extent and exploitable by ripping. An example is the Prims dam, Germany, where the dam body consists of clay slate with the following parameters:

- Fill unit weight 21 kN/m^3,
- friction angle 37.5°,
- cohesion 50 kPa,
- permeability 5×10^{-4} to 5×10^{-5} m/s.

For more examples refer to Fell et al. (1992). Attention must be given to the particle shape. The shape should be such as to allow the establishment of an 'isotropic' embankment, in a practical sense. A system classifying the particle shape was developed by Zingg (1935) with the nominations 'flat', 'cubical', 'flat stalked' and 'stalked' (Fig. 4.30). The material shown is similar to that of Prims. Typically, the particle shape tends from cubical to stalked with increasing particle size. USBR (1974) classifies the particle shape with the terms bulky or equidimensional (rounded, subrounded,

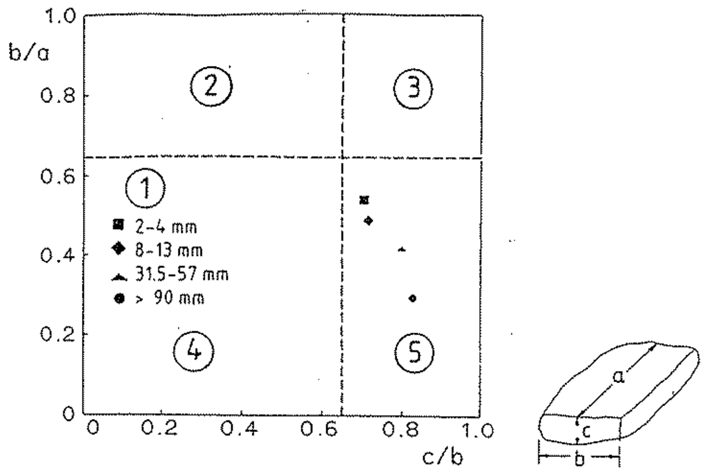


Figure 4.30. Particle shape of slaty rockfill material. Classification after Zingg (1935) (courtesy of IBF).

1 Grain size
2 Flat
3 Cubical
4 Flat stalked
5 Stalked

subangular, angular), flaky or platelike and elongated. Flaky and elongated shapes apply mainly to fine-grained soils.

4.3.4 *Mineral filter materials and concrete aggregates*

This section deals with the two materials because a geotechnician is frequently employed for the identification of both in the field, and within the same working period. Often the sources of the materials and the quality requirements are the same. The materials are natural sand and gravel from deposits or crushed sand and gravel from quarries.

The following tests are usually done to confirm the suitability of filter materials:

– Determination of the mineralogical composition,
– determination of the specific gravity,
– point load test on coarse gravel and cobbles. The compressive strength should be more than about 70 MPa,
– determination of the water absorption potential,
– abrasion test (Los Angeles, ISRM 1977, or equivalent),
– slake durability test (ISRM 1979, or equivalent),

– determination of the particle shape of natural materials. Cubical and spherical shape are preferred. Flat and stalk-shaped particles tend to segregation,

– determination of deleterious and soluble contents.

In some cases an analysis of the chemistry of filter material may be needed to confirm the compatibility with seepage water from the reservoir.

It should be remembered that filter materials of natural deposits only in rare cases show the grain size distribution required to guarantee proper filter function. The materials have, in most cases, to be separated in fractions and then to be newly composed. It is, therefore, recommended to analyze the grain size distribution of a great number of samples to enable a correct estimation of the amount of material which has to be extracted from the deposits to render sufficient material of the specified grain size distribution.

Usually, the standards to be set for concrete aggregates are more stringent than those for filter materials. The strength of individual pieces should be about 100 MPa. This may be relaxed according to the strength requirement of concrete. As a rule, the following tests and determinations are made:

– Mineralogical composition,

– grain size distribution, with respect to the amount of material needed, workability and strength,

– particle shape, with respect to workability, see Section 4.3.3,

– specific gravity,

– water absorption potential,

– resistance to frost,

– resistance to weathering (sodium sulfate test),

– resistance to abrasion (Los Angeles abrasion test or equivalent),

– deleterious contents, such as fines, organic and soluble matters, chemicals,

– alkali reactivity in respect of the durability of concrete.

The tests and determinations and the conclusions to be drawn from the results are described in the standards and in respective manuals (e.g. USBR Concrete Manual).

4.3.5 *Ground water and surface water*

Samples of ground water and surface water must be analyzed chemically to check the aggressiveness to concrete and other construction materials. Ground water sampling must cover all existing phreatic nappes which may be separated from each other.

Samples of surface water should be collected repeatedly in different seasons and at different discharge levels. For wide rivers it is recommended to take water samples close to both river banks and from the middle of the river.

4.4 AVAILABILITY AND SUITABILITY OF NATURAL CONSTRUCTION MATERIALS

Prior to completion of tender documents and prior to contracting procedures, the availability of the required amount of construction materials and their suitability must clearly be evidenced. The costs of identification and of investigations increase with the amount of materials. Working in the field often demands extreme effort from the staff involved, under hard conditions in remote areas, because the work has to be done in an early phase when no infrastructure has developed.

As an international rule it is usual to identify an amount of 200% of the calculated demand for material. This amount takes regard of all imponderables, e.g.:

– Inaccurate calculation of the available amount in the borrow areas. It is noted that the lower boundary of deposits can be determined only pointwise,

– inaccurate calculation of the required amount,

– unexpected additional excavation in the foundation area,

– unsuitable material in the borrow areas which had not been detected by the field investigations.

Such imponderables have in practice proved the 200% rule to be realistic. Any missing material will lead to additional costs, to delays in construction, to claims or to a need to re-design as described in Section 5.2 for the Agus IV dam. It is recommended to all designers not to relax the effort of field identification to meet the 200% requirement.

The suitability of natural materials for various purposes gives some free play to the designer. The designer must be prepared to innovate when the materials approach the boundaries for their suitability. Table 4.12 gives an overview of the utility of natural materials. Their manifold use can be seen from the dam types presented in Chapter 5 and from numerous existing dams. Quality can be improved by material processing, but limits are set by the costs in comparison to the use of artificial materials.

The range of use of natural materials for sealing purposes is demonstrated in Figure 4.31, with examples of existing dams. The grain size curves represent natural soils. Besides 'typical' materials the extreme ends are given by non-plastic soils with a permeability of about 10^{-5} m/s (no 7) and by the wet core of highly plastic clay (no 12), marked by low compactability and high deformability.

Figure 4.31 should not be interpreted in the sense that the suitability of a soil for sealing purposes would depend only on the grain size distribution. In addition, the plasticity, the permeability, the compactability and the workability have to be checked, as well as the geological history, i.e. the degree of weathering. Examples are the decomposed granite in Figure 4.31 and the mudstone and weathered materials thereof in Figure 4.16. It should be noted

Table 4.12. Typical earth and rockfill materials in dam engineering.

Zone of dam	Earthfill/rockfill material	Examples	Typical range of properties
Impervious zone	Cohesive, plastic soils	Cohesive river sediments (loam) Residual soils Laterite Moraine Colluvial soils Slope debris Tertiary marl	Water content between $w_{opt} - 10\%$ to $w_{opt} + 10\%$, PI = 10 to 50, cohesion > 20 kPa, $k < 10^{-7}$ m/s
	Low plastic soils	Residual silt Alluvial silt Highly weathered rock Rippable rock Slope debris	PI = 0 to 5, cohesion near zero, friction angle ≥ 28°, $k < 10^{-6}$ m/s
Shell	Non-cohesive soils	River alluvions Slope debris Slightly weathered rock Rippable rock Moraine	Friction angle ≥ 35°, cohesion = 0, or friction angle ≥ 33°, cohesion ≈ 20 kPa
	Rockfill material	Quarry-run rock of igneous, volcanic, sedimentary and metamorphic origin	Compressive strength > 70 MPa
Filter	Non-cohesive soils	River alluvions Pure sand and gravel of other deposits	Quality equal to concrete aggregates
	Crushed rock	All types of rock	Quality equal to concrete aggregates
Wave protection	Rockfill material	Selected rock of igneous, sedimentary and metamorphic origin	Compressive strength > 100 MPa

that all the materials in Figure 4.31 are well graded. The quality of sealing materials can be improved by processing (Section 9.3). Necessary compaction and related material breakage are not considered as processing.

The wide range of use of natural materials for dam shells is demonstrated by the grain size distributions in Figure 4.32. The upper boundary is given by the weight of individual blocks which can reasonably be handled on site. Such blocks embedded in graded material are used as riprap for wave protection. The maximum weight is about 2000 kg, corresponding to a block of about $1.1 \times 0.8 \times 1.0$ m in size with rounded edges.

The lower boundary for non-cohesive materials for shells is given by the risk of liquefaction at saturated conditions. Endangered material can be used only in the downstream shell, which is not saturated. Here, the boundary is

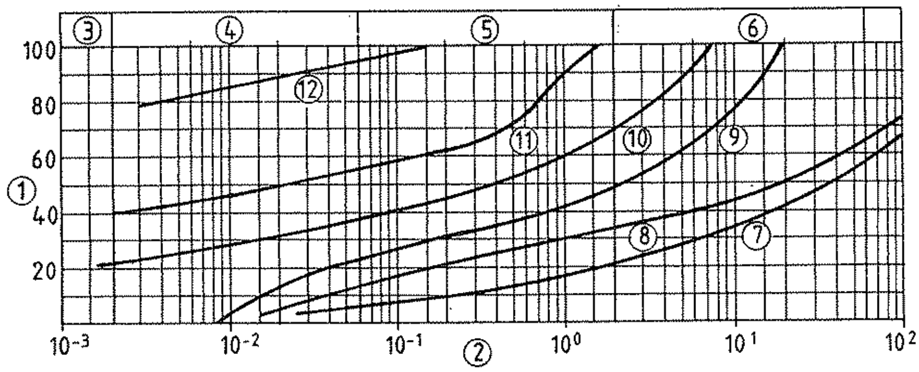


Figure 4.31. Examples of sealing materials.

1 Percent finer by weight
2 Grain size (mm)
3 Clay
4 Silt
5 Sand
6 Gravel
7 Coarsest non-plastic sealing material used in Swedish dams (Bernell 1982)
8 Coarsest sealing material of Messaure dam, fine-grained moraine (Bernell 1982)
9 Coarsest sealing material of Chawalla Gorge dam, decomposed laterite (Lazany 1986)
10 Finest sealing material of Kinda dam
11 Finest sealing material of Agus IV dam, laterite
12 Mean gradation of wet core material of Monasavu dam, halloysitic clay (Knight et al. 1982)

set by the shear strength of friction and cohesion which is required for slope stability. Such soils are to be seen as a transition to cohesive soils, which are the only material of homogeneous dams.

As before, the suitability of shell materials does not depend only on the grain size distribution. Particle strength, breakage, resistance to weathering and workability are other criteria. The workability is related to the uniformity in the sense that poorly graded materials are not easy to compact. Accordingly, all the materials in Figure 4.32 are well graded.

Processing of shell materials to improve quality has stricter limitations than processing of cohesive materials. It is always intended, and in most cases achieved, to use the materials 'quarry-run', i.e. without any processing after blasting. The costs of further processing would have to be compared with the costs of a concrete structure. In contrast, filter materials must usually be processed, which means washed, screened and newly composed according to the specified grain size distribution.

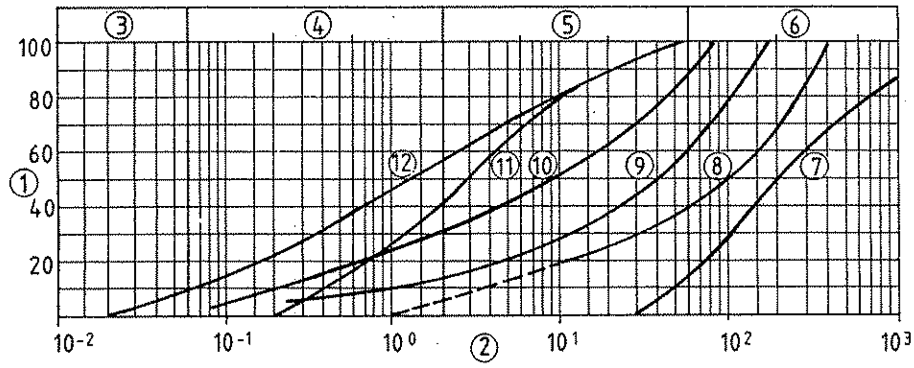


Figure 4.32. Examples of shell materials.

1 Percent finer by weight
2 Grain size (mm)
3 Silt
4 Sand
5 Gravel
6 Cobbles
7 Coarse rockfill material
8 Mean gradation of Aabach dam shell material (Idel 1981)
9 Mean gradation of Oroville dam shell material (Marachi et al. 1972)
10 Finest material of Cethana dam material (Thomas 1976)
11 Approximate coarsest material developing pore-pressure under cyclic load
12 Mean gradation of Chico dam shell material, downstream part, transition to sealing materials

4.5 SELECTION OF THE TYPE OF DAM

Usually the selection of the dam type is based on the availability of materials and on the costs.

The choice between a natural and an artificial sealing is made mainly in respect of the availability of material and the costs of its processing, hauling, dumping and compacting. Dams with artificial sealing demand far higher technical standards than dams with natural sealing. Therefore, preference is often given to natural sealing, particularly in remote areas with little or no infrastructure.

Artificial sealings are more sensitive to settlements of the substrata and – in the case of face sealings – of the upstream shell. An advantage is that a short construction period is attributed to face sealings, related to the reduced volume in comparison to the volume of an embankment with an earth core. This advantage was optimized with concrete face rockfill dams by incorporating the upstream cofferdam and managing the risks of temporary inundation or overtopping during construction (Cook 1984).

The Aabach dam is an example of selection of the type of dam after a cost comparison. Initially, the dam was designed as a rockfill dam with a face of asphaltic concrete (Idel 1981). Test blastings revealed a rapid disintegration of the potential rockfill material of siltstone. After laboratory and field compaction tests the material proved, in mixture with colluvium, to make a good core material. Cost estimates could not clarify whether a design according to Figure 5.2 would lead to lower costs than a filldam with an asphaltic face sealing, because construction delays due to the weather conditions had to be expected. The tender documents asked for bids for both variants. The design according to Figure 5.2 proved to be the more economic version because it was possible to suspend a continuous grout curtain, due to the long seepage path below the sealing blanket.

A cost comparison is also advisable if two or more proper designs result from the available materials. As an example, the tender documents for the Kinda dam included a rockfill dam with earth core and a zoned earth embankment. The rockfill version revealed the lower costs, and it was constructed (Figure 5.4).

Usually, selection of the dam type is made in the course of the field investigations (Section 4.2), prior to the completion or even prior to the beginning of laboratory investigations. The foundation conditions which may affect this selection will have been evaluated previously, namely at the time when the dam location was selected.

Selecting the dam type prior to or at the beginning of laboratory tests has a precondition: the experts involved must be experienced enough to confirm the suitability of potential materials after only visual inspection. In complex cases large scale field tests will be needed, as described in Section 4.2.6, to evaluate the suitability of a part of the materials and to estimate the costs of their processing, if required. An example is the Bakun rockfill dam in its early design phase and the related field compaction test on potential core material (Section 4.2.6.2a). In this case the laboratory results had, in part, been available when the dam type was selected.

The following basic consideration should be mentioned: the selection of the dam type does not mean voting for the 'best' type against others of lesser quality. All dams designed and constructed according to the state of the art are equivalent in their standard. The task is to find the most appropriate version as an optimum of all technical and economic aspects, under consideration of the bounding conditions of the project. The variability of potential designs and the challenge for the designer originate in this.

CHAPTER 5

Types of dams

5.1 HOMOGENEOUS DAMS

The dams called 'homogeneous dams' are the prototype of dams constructed for millennia to protect people and cultivated land from inundation and to create water reservoirs. They are essentially of only one type of soil (Fig. 5.1). Such dams are not homogeneous in the literal sense, since the construction method in subsequent layers leads to different permeabilities in horizontal and in vertical directions up to $k_H = 10 \times k_V$. In our times filter layers are attached to the dams for safety reasons, designed to constitute favourable hydraulic conditions.

The main construction material must be impervious in a practical sense. That means, mainly cohesive soils like silt and clay are considered, having a permeability after Darcy of 10^{-7} m/s or less. Such soils do not develop high shear resistance, so the outer slopes are flat in the range of 1V:2.5H to 1V:5H.

The flat slopes are related to wide foundation areas and hence to large dam volumes. So the height is limited, for practical reasons, to about 30 to 35 m. Exceptions may be two dams 70 m in height now almost completed in India.

5.2 ZONED DAMS WITH NATURAL SEALING

Zoned dams consist of more than one type of construction material. The sealing member does not cover the whole cross-section. Other zones of the dam are made of materials of higher shear strength. This results mainly in two advantages:

– Non-cohesive soils can be used, which are easier to handle on site. They allow an increased daily production in building the embankment,

– the higher shear strength of non-cohesive materials results in steeper

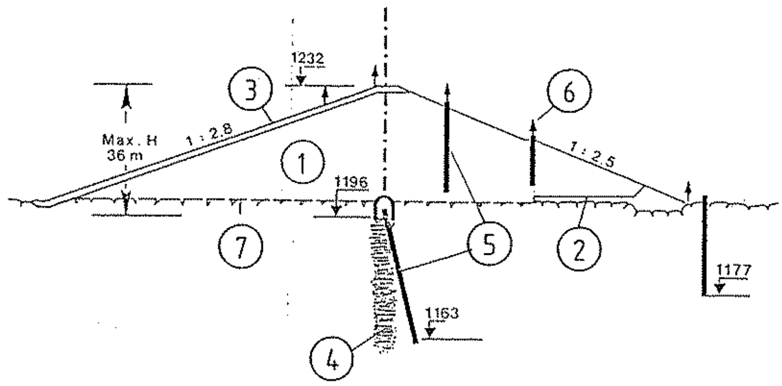


Figure 5.1. Typical section of a homogeneous dam. Example Shen river dam, Nigeria 1979.

1 Dam body of laterite
2 Filter
3 Riprap
4 Grout curtain and inspection gallery
5 Piezometer
6 Surface survey point
7 Foundation of granite

outer slopes and in a reduction in dam volume, compared to homogeneous dams.

The zoning follows mainly from the type and availability of the materials. An optimum is a zoning where permeability increases from the inner parts to the outer parts. This is, usually, related to decreasing deformability from the inner to the outer parts. Both are achieved, for instance, with the Aabach dam in Figure 5.2.

A more complex zoning is shown in Figure 5.3. This 32 m high dam Agus IV consists of six different materials. The as-built section had to be developed during construction, because of insufficient investigations – due to regional security problems – in the design phase. The quality of the rockfill material available from two long power tunnels proved to be worse than expected. The design was adjusted in two steps to the real conditions when this quality, and later a quantity, problem arose. Only the cohesive material for dam sealing was available in a virtually unlimited amount.

Initially, the dam was designed as a rockfill dam with earth core. Relics of this are the rockfill zone in the downstream shell (zone 2 in Fig. 5.3) and the steep downstream sloping of zone 1. The upstream impervious blanket was introduced after the foundation proved to be semi-permeable. This blanket is responsible for the flat upstream slope of the dam which – vice versa – enabled increased use of clayey silt in the upstream dam body and decreased use of rockfill material there.

The example demonstrates the flexibility in material selection and use

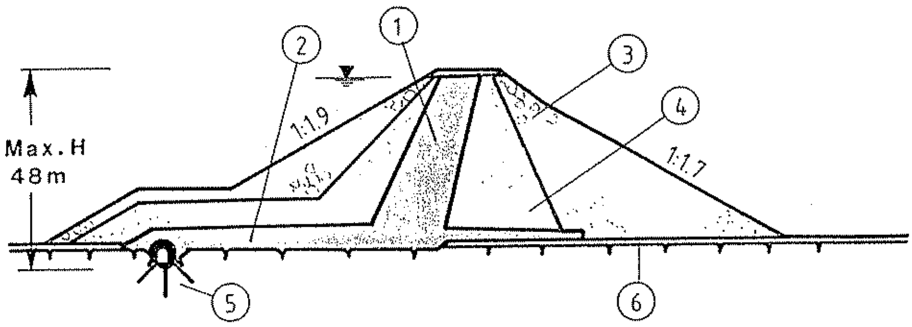


Figure 5.2. Typical section of a zoned rockfill dam. Example Aabach dam, Germany 1979 (adapted from Idel 1981).

1 Impervious core
2 Impervious blanket
3 Rockfill, max. grain size 600 mm
4 Transition zone of sand-gravel with cobbles, max. grain size 150 mm
5 Inspection gallery and contact grouting
6 Drainage

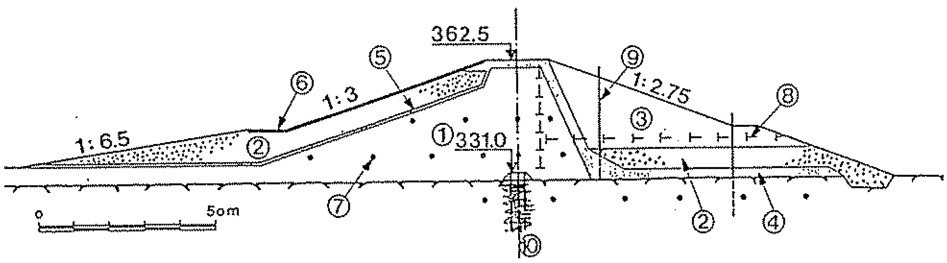


Figure 5.3. Typical section of a zoned earthfill dam. Example Agus IV dam, Philippines 1984.

1 Impervious zone of clayey silt with upstream impervious blanket
2 Rockfill
3 Fill of weak basalt
4 Coarse and fine filters, 1.2 m each
5 Coarse and fine filters, 0.5 m each
6 Riprap
7 Pore pressure gauge
8 Measuring instruments for horizontal and vertical displacements
9 Piezometer
10 Grout curtain and grout cap

which may be required, on the one hand, but is justified on the other. It is an instructive example of the philosophy 'design as you go'.

5.3 ROCKFILL DAMS WITH EARTH CORE

This type of dam is an almost ideal version of large dams (Fig. 5.4). Besides

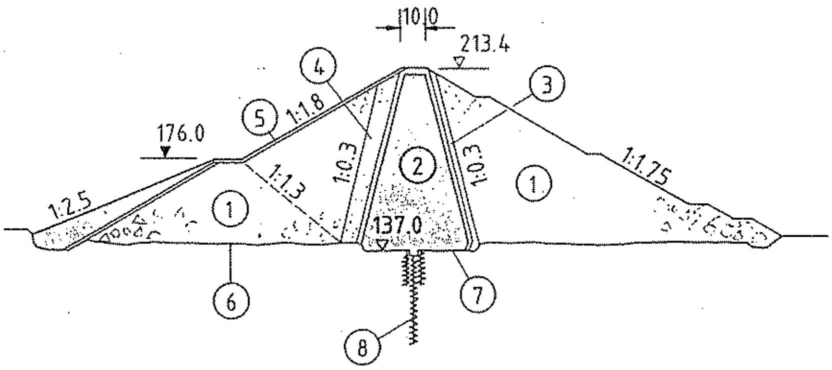


Figure 5.4. Typical section of a rockfill dam with vertical earth core. Example Kinda dam, Burma 1987.

1 Rockfill, max. size 1000 mm
2 Impervious core of sandy, clayey silt
3 Coarse and fine filters, 2.0 m each
4 Transition zone, max. grain size 150 mm
5 Riprap
6 Foundation of weathered rock
7 Foundation of fresh rock
8 Grout curtain

the impervious core and free-draining rockfill materials, only gravel and sand for filters are required. Frequently, the typical section is modified by using other materials from excavations, such as diversion, spillway, power house and the foundation excavation. The dam design must be adjusted to the quality of these materials.

The prototype is the dam of high quality free-draining rockfill material with an impervious earth core. Two variants exist:

– The core is arranged symmetrically around the center line of the dam (Fig. 5.4). This is called a dam with a vertical core, because the center line of the core is vertical,

– the core is inclined, with the arrangement shown in Figure 5.5. This is called a dam with an inclined core, because the center line of the core is inclined.

The core sloping results from the deformation conditions between the core and the shells and from the desired hydraulic gradient of the water seeping across the core. Typical sloping of the vertical core on both sides is 1V:0.25H to 1V:0.33H. Typical sloping of the inclined core is given in Figure 5.5. The hydraulic gradient is about $i \leq 2$.

The outer slopes of dams with a vertical core correspond to the high shear strength of rockfill material, provided the shear strength of the foundation is higher than or equal to the rockfill's shear strength. Otherwise the dam slopes must be flattened or stabilized by berms.

The upstream slope of dams with an inclined core must be slightly flatter than that of dams with a vertical core. This is independent of the shear

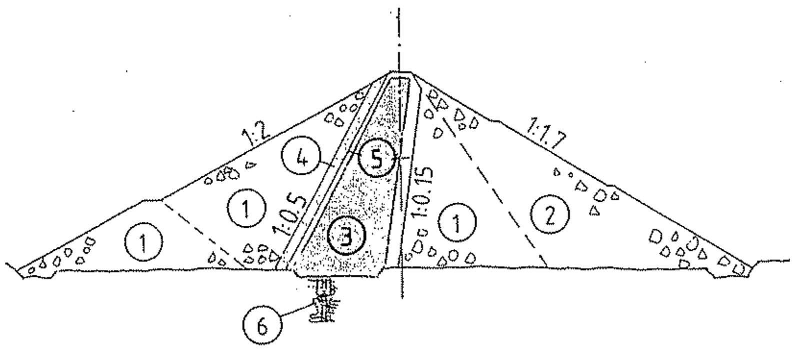


Figure 5.5. Typical section of a rockfill dam with inclined earth core (scheme).

1 High quality rockfill, max. grain size 600 mm
2 Rockfill, max. grain size 1000 mm
3 Impervious core
4 Transition zone
5 Filter
6 Grout curtain

strengths of dam and foundation. This follows from the fact that critical failure planes cut the dam core over a longer section, in principle reducing the total shear resistance along the failure plane.

The transition zones and the filters on both sides of the core serve to guarantee hydraulic safety and to compensate for differential settlements between the core and the shells (Section 7.3.3).

At present, the highest rockfill dams with an earth core are the Nurek and Rogun dams, 317 and 325 m in height, respectively. Variants of rockfill dams, equivalent in safety, are the Oroville and Mica gravel dams, 228 and 244 m in height, respectively. Rockfill and gravel dams with earth cores are highly resistant to earthquakes (Seed 1979, Thomas 1976).

5.4 DAMS WITH ARTIFICIAL INTERNAL SEALING

In former years dams were constructed with a concrete wall as a core sealing. Such concrete structure serves to replace cohesive soils if they are missing. Disadvantages are the complex deformation conditions between the rigid concrete structure and the dam shells, and the concrete wall itself. It is a hindrance to the progress of construction because it must be constructed between formwork ahead of the neighbouring dam portions.

About 1980 this type of dam again came into discussion, since the stresses and strains on the concrete structure could now be better computed and controlled with the aid of finite elements (Schober 1982, 1988). A recent example is the Bockhartsee dam, 31 m in height (Schober & Lercher 1985).

As a prototype of large dams, the rockfill dam with an internal sealing of

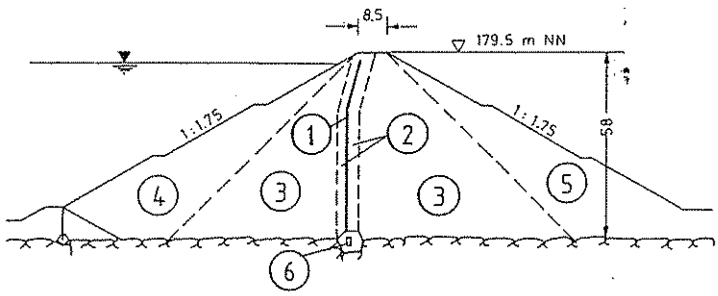


Figure 5.6. Typical section of a rockfill dam with internal sealing of asphaltic concrete. Example Grosse Dhünn, Germany 1980 (adapted from Breth & Arslan 1990).

1 Asphaltic concrete, 0.6 m
2 Filter and transition zones
3 Fine rockfill
4 Coarse rockfill
5 Random rockfill
6 Concrete structure with inspection gallery

asphaltic concrete was developed after World War Two (Fig. 5.6). The sealing membrane is located on top of a concrete structure, frequently with an inspection gallery. The asphaltic concrete is constructed at the same time as the lateral dam portions, both being at the same elevation. Its properties enable it to make the sealing membrane-like at widths between some decimeters and about 1.5 m. For reasons of deformations, transition zones are needed on both sides of the membrane. The downstream transition zone serves also for seepage control.

The dam shells are made of all types of rock and earth materials which guarantee stability and safe hydraulic conditions of the structure. The largest existing rockfill dams of this type are the High Island dams in Hong Kong with heights of 109 and 101 m (Lehnert & Geiseler 1979).

5.5 DAMS WITH FACE SEALING

It is an old idea to seal an embankment dam by placing impervious material on its upstream slope. Such a design leads to the minimum possible width of the dam's base because the whole dam body and the water load itself contribute to activate shear resistance in the foundation hence constituting sliding safety of the dam. Impervious soil may be used as a face sealing for small dams and for dams of short lifetime, such as cofferdams.

For large dams a layer of flexible asphaltic concrete is appropriate which follows the deformations of the dam without cracking. A prototype is the rockfill dam with face membrane of asphaltic concrete (Fig. 5.7). Its development to a high standard started in Germany after World War Two. Settle-

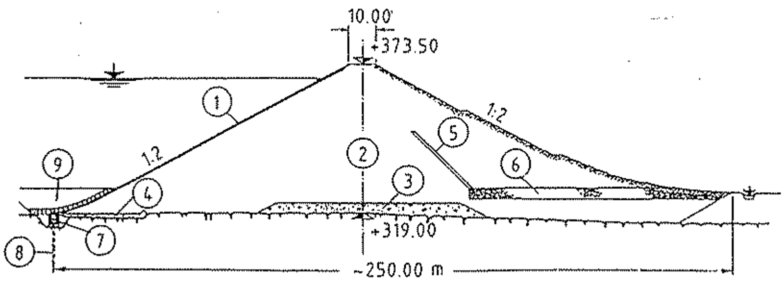


Figure 5.7. Typical section of a rockfill dam with face sealing of asphaltic concrete. Example Obernau dam, Germany 1972 (adapted from DNK & DVWK 1987).

1 Asphaltic concrete, 21 cm, controlled
2 Rockfill of clayey slate and sandstone
3 Valley gravel
4 Filter
5 Inclined filter
6 Drainage
7 Inspection gallery
8 Grout curtain
9 Random fill

ments and deformations are minimum when rockfill is dumped and highly compacted on a practically incompressible foundation.

The membrane is placed in several layers, without joints, from bottom to top, to a total thickness of about 50 to 200 mm. It is underlain by a layer of sand-gravel acting as a transition between the membrane and the rockfill dam body and as a leakage limiting element in case of perviousness of the membrane. Typical sloping is 1V:1.5H to 1V:2H. At the perimetric joint the membrane is connected to a concrete structure which is, in Europe, usually designed as an inspection gallery.

The height of existing dams is less than 100 m. Dams up to 50 m in height have been constructed of other than typical rockfill materials (e.g. Prims, Germany 1982, Lorson et al. 1983) and on other than rock foundations (e.g. Castello, Italy, Bigalli et al. 1980).

After about 1965 cement concrete was favoured instead of asphaltic concrete. A prototype is the concrete face rockfill dam (Fig. 5.8). Its development to a high standard started in the USA, while European engineers still have some doubts about the flexibility and the tightness of the system. Typically, progress in development was dictated by empirical rather than theoretical considerations.

The concrete face is placed in vertical sections from bottom to top to a thickness of about 0.3 to 0.6 m. It is underlain by a transition and leakage limiting layer of specified grading. During embankment construction, this layer is stabilized by shotcrete or bitumen as a protection against erosion and damage by mechanical attack. Typical sloping is 1V:1.4H. The vertical joints between the sections and the perimetric joint at the toe slab (plinth) are

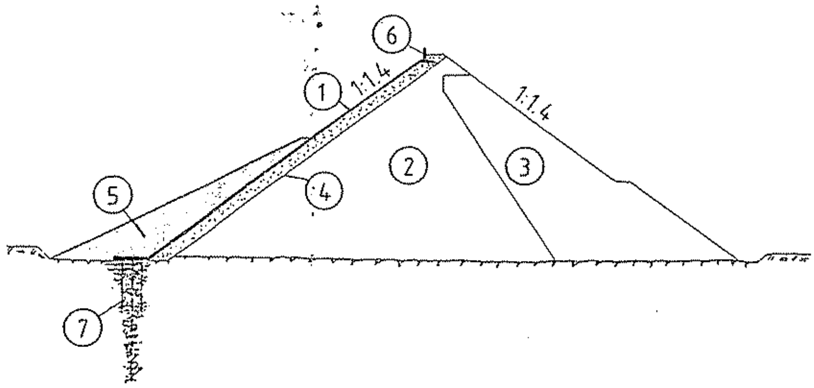


Figure 5.8. Typical section of a concrete face rockfill dam (scheme).

1 Concrete face, 30 to 60 cm	5 Earth fill
2 Rockfill, max. grain size 600 mm	6 Parapet wall
3 Rockfill, max. grain size 1500 mm	7 Plinth and grout curtain
4 Transition zone, max. grain size 200 mm	

sealed by a system of waterstops. The lower portion of the face is frequently covered by earth and random fill. The function of this fill is self-sealing of defects of joints and of cracks in the concrete.

The height of existing dams is up to 160 m (Foz do Areia, Brazil 1980). A 200 m high dam is presently under construction (Bakun, Sarawak). Design concepts are created to a height of 250 m (Cooke 1984).

CHAPTER 6

Assessment of the performance of dams

6.1 STATIC LOADS

6.1.1 *Deformation due to dead load*

All deformations of the dam body depend on the dam zoning and on the properties of the construction materials. The deformations can be computed by means of the finite element method and measured with the help of embedded instruments.

Usually, only the dead load of the dam acts during the construction period until the typical load case 'end of construction'. The sequence of dam construction, by placing individual layers, causes settlements of the previously dumped layers and of the foundation. Pore-water pressure develops in all fine-grained materials, in the construction materials as well as in the foundation. This pressure increases until the end of construction and dissipates subsequently. The process is superimposed by the reservoir impounding, irrespective of its start during construction or after completion. In the following discussion – for easier understanding – the impounding is assumed to start soon after the end of construction.

Considering an embankment dam with earth core, the cohesive material of the core is frequently the most compressible material. Settlements of the core will be maximum. Typical settlements of a vertical core in a rockfill dam during construction are shown in Figure 6.1. With respect to the properties of common construction materials, the maximum settlement at the end of construction will be about 2% of the dam height H. The location of the maximum is between 0.5 and $0.7 \times H$, as shown. This location, not at the dam crest but below it, follows from the fact that the lower portion of the dam consolidates partly or completely, with the development of respective settlements, while the upper portion is still under construction.

The dam shells settle also, usually at lesser rates than the core. Trimming of the slopes may compensate for such settlements, if required.

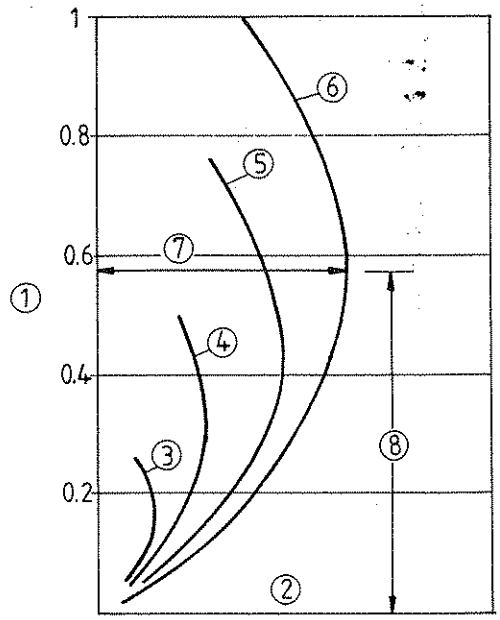


Figure 6.1. Rockfill dam with vertical earth core on incompressible rock. Development of core settlements due to dead load.

1 Temporary dam height in parts of total height *H*
2 Settlement
3 Height 0.25 *H*
4 Height 0.5 *H*
5 Height 0.75 *H*
6 End of construction
7 Max. settlement about 0.02 *H*
8 Location of max. settlement 0.5 to 0.7 *H*

The distribution of computed settlements of the Kinda dam (Fig. 5.4) at the end of construction is shown in Figure 6.2. Measured settlements of the core were 130 cm and 63 cm of the downstream shell; computed values are 180 cm and 100 to 125 cm, respectively.

The greater settlements of the weak core, in comparison to the stiff shells, correspond to an irregular distribution of stresses. This is shown for the maxima of stresses at the bottom of the dam, i.e. in the foundation line, in Figure 6.3. Equal unit weights and moduli of the materials would give the regular distribution shown, proportional to the dam height. Given a weak core, the upper portion of the core might 'hang up' via friction at the interface of the core and the shells, while the lower portion settles according to consolidation. This arching effect leads to reduced vertical stresses in horizontal planes across the lower portion of the core and in the foundation line. The respective stress distribution shows the stress deficit below the core and the effect of stress transfer from the core to the shells below the neighbouring zones. Horizontal cracks might develop at the outer zones of the core or – in extreme cases – across the core at too high a deficit of vertical stresses. The problem of arching is discussed in detail in Sections 7.3.2 and 9.9.3.

Protective measures against arching are mainly wide filter and transition zones between the core and the shells, and reasonable sloping of the core. To some extent, the deformation conditions of the transition zones can be controlled by appropriate compaction. Moderate density will help to make the

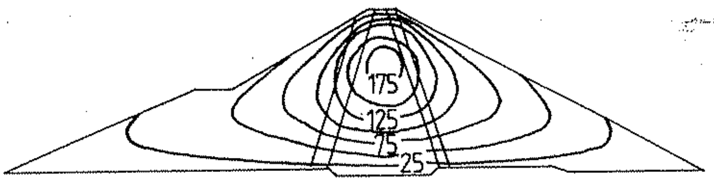


Figure 6.2. Kinda rockfill dam as in Figure 5.4. Computed lines of equal vertical settlements (cm) at end of construction (Kutzner et al. 1988).

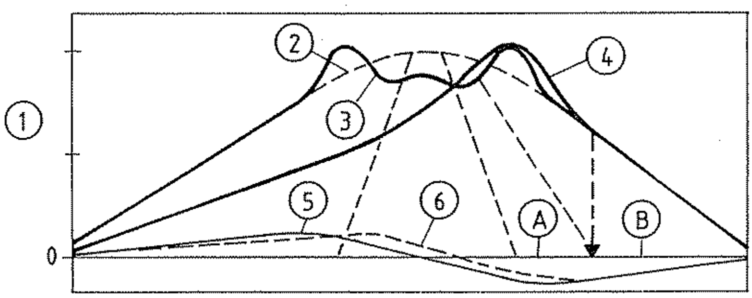


Figure 6.3. Rockfill dam with vertical earth core on incompressible rock. Stress distribution at the bottom of the structure.

1 Stresses
2 Vertical stresses at end of construction,
equal unit weights and moduli of shell and core materials
3 Vertical stresses at end of construction, weak core
4 Vertical stresses after impounding
5 Shear stresses at end of construction
6 Shear stresses after impounding
A Foundation area with increased stresses after impounding
B Foundation area with unchanged stresses after impounding

transitional function effective. For the Kinda dam a measured and computed deficit of vertical stresses in the range of 20% did not reveal an intolerable risk of arching. This may be taken as an approach to this problem with other dams.

Settlements of the foundation should be kept as small as possible. Protective measures are proper foundation preparation and pre-consolidation of compressible zones. At the end of construction the foundation should be consolidated to a high degree. Exceptions are dams on a weak foundation where an appropriate dam zoning will help to exclude core damage.

6.1.2 *Deformation due to impounding*

The stress and strain conditions of the dam change under the effect of the water load. Such change is in principle different for dams with an internal sealing and dams with a face sealing. This follows from the different locations of the resultant of the hydrostatic pressure (Fig. 6.4).

Water from the reservoir seeps through the internal core. Given simplified assumptions, the resultant of the hydrostatic pressure acts at the upstream face of the core as a force consisting of horizontal and vertical components. The horizontal component presses the core towards the compressible downstream shell, causing a horizontal and a tilting movement of the core. Given a rigid foundation, the displacement increases from the bottom to the crest (Fig. 6.5). The displacement rate depends on the material properties, the height and length of the dam and the curvature of the center line in plan. Rates at the crest in the order of several decimeters are acceptable.

The lateral displacements, across the dam section as shown in Figure 6.6, increase along the line a-a from the upstream slope towards the center of the core and increase from there towards the downstream slope. The maxima of lateral displacements due to impounding are located in the center of the dam. The lines of equal displacements show an upward inclination from the center to both outer slopes. The lines cut the downstream slope approximately at right angles. Inclination and cutting angle reflect the tilting tendency of the core instead of a sliding tendency at the bottom. The rate of lateral dis-

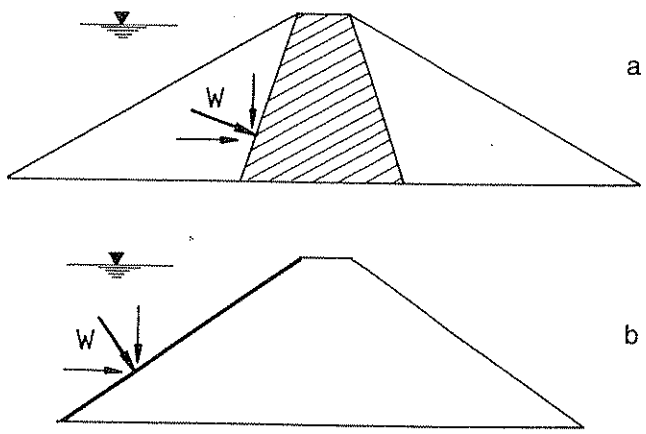


Figure 6.4. Location of the resultant force due to the water head after impounding (simplified).

a Rockfill dam with internal sealing
b Rockfill dam with face sealing
W Resultant force with vertical and horizontal components

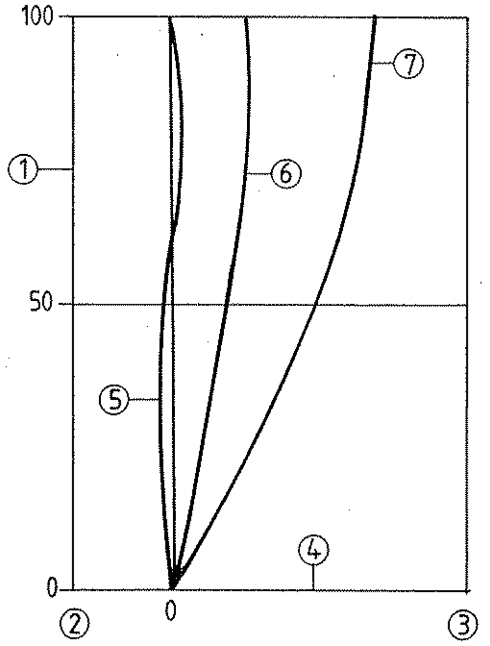


Figure 6.5. Rockfill dam with vertical earth core on incompressible rock. Development of horizontal core displacements due to dead load and impounding.

1 Dam height H (%)
2 Upstream side
3 Downstream side
4 Horizontal displacement
5 End of construction
6 Reservoir partly impounded
7 Reservoir fully impounded

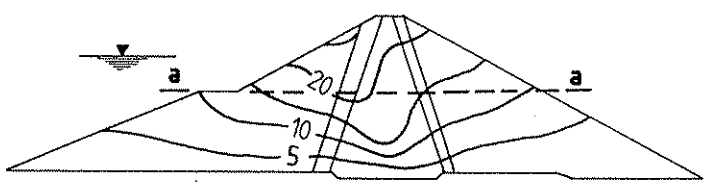


Figure 6.6. Kinda rockfill dam as in Figure 5.4. Computed lines of equal horizontal displacements after impounding to 0.7 H (Kutzner et al. 1988).

placement of the downstream toe may be several decimeters. A rate of a few meters is regarded as unacceptable. The measured and computed rates of the Kinda dam downstream toe were in the range of 4 to 6 cm.

The vertical deformations due to impounding are, usually, considerably less than those due to the dead load. Many dams on rock foundations did not show any additional settlement. The relation of the three components, namely construction settlement, post-construction settlement and foundation settlement, remains unknown since in many cases the consolidations due to dead load and due to impounding overlap each other. This can be traced from the pore pressure development. Dascal (1987) reports on a number of dams in Canada where crest settlements after the end of construction remained less than 1% of the dam height over a period of several years of operation.

Impounding causes uplift of the upstream dam body. The vertical stresses in the foundation line below the upstream portion are reduced to about half of the original value (Fig. 6.3). Due to the water load the maximum of stresses is shifted downstream. The stress deficit under the core disappears. Within the downstream shell the water load is effective only to the inner portion and the related section of the foundation (A in Fig. 6.3). Only there additional deformations may occur due to impounding. The outer part of the shell is not affected.

Due to the changes in stresses and stress distribution, differential settlements occur between the core and the shells. They must be bridged by the transition zones, which are designed to provide the required potential of shear deformation. The differential settlements at the upstream interface of core and shell may be greater than those at the downstream interface. This is because of the interchange of dead load and uplift conditions. Therefore, the transition zone is occasionally made wider upstream than downstream. The dimensions and the function of transition zones are in detail discussed in Section 7.3.3.2.

It can be deduced from Figures 5.4 and 5.5 that an inclined core can follow the differential deformations easier than a vertical core. The support of the downstream shell given to an inclined core is more compatible than that given to a vertical core, and, vice versa, the shear deformation between core and upstream shell can be bridged more easily. This advantage of an inclined core over the vertical core should be considered in the light of the disadvantage, namely the need to flatten the outer slope upstream (see Section 5.3).

Impounding does not cause uplift and related changes in stresses in dams with a face sealing. It leads to stress increase in the dam body, to a rotation of the principal stresses downstream and to related dam deformations. The stress increase is again restricted to upstream and center portion of the dam. The outer downstream portion is not affected. The dam tends to settle and to move downstream. The face membrane must follow the deformations of the supporting zone. The axial strain of the face between the toe and the crest will be negligible. Therefore, shear deformations will occur between the face and the dam body, which must be bridged by a transition layer.

6.1.3 *Slope stability*

Slope stability is controlled by the shear strengths of the dam materials and the foundation. The proof is according to the failure theory of Coulomb (1776) which is applied – modifying his approach – to circular or otherwise curved or to polygonal slip planes. The failure theory proved to be applicable to common construction materials.

With a homogeneous dam on an equivalent foundation (Fig. 6.7a) the slope itself is the critical slip plane, provided the shear strength is given by friction only. In contrast, the critical plane cuts through the dam if the shear strength is given by friction and cohesion.

With a zoned dam with earth core on a foundation of equal or higher shear strength, the critical slip plane touches the foundation line (Fig. 6.7b). In the vicinity of the dam toe the tangent is horizontal. Given reduced strength of the foundation the critical deep slip plane cuts through the dam and the foundation (Fig. 6.7c). It outcrops at the surface outside of the dam. Due to the horizontal component of the stresses in the foundation line, the dam toe tends to move away from the center of the dam. The resultant of the horizontal stresses is called spreading force. With too large a displacement the slope must be flattened or a stabilizing berm must be added to the toe. Spreading forces also exist but are negligible or almost negligible, given equal shear strengths of dam and foundation.

Failure occurs when the driving forces along the slip plane exceed the resisting forces (or driving and resisting moments, see Section 8.2.1). The mass above the slip plane slides (Fig. 6.7c). Large deformations occur. The sliding movement comes to rest when a new configuration of the system constitutes a new equilibrium of the forces. Sliding movements along circular or almost circular planes have frequently been observed in nature.

Polygonal slip planes have to be considered where there is an unfavourable stratification of the foundation, with layers of low shear strength. An example is the Mornos dam (Fig. 6.8) where weak mylonites at a shallow depth necessitated a flat slope of 1V:2.0H (26°) and a stabilizing berm. The friction angle of the dam material was at least 35°.

The proof of the slope stability of dams with a face sealing is made in the same way. The location of slip planes is according to Figure 6.7. The foundation is preferably strong rock where the critical slip plane cuts the dam and touches the foundation line tangentially. Foundations on weak rock are rare. Examples are known from Italy, according to the prevailing geological conditions of some regions there (Bigalli et al. 1980). It is noted that the face sealing will not be able to follow greater horizontal movements without cracking.

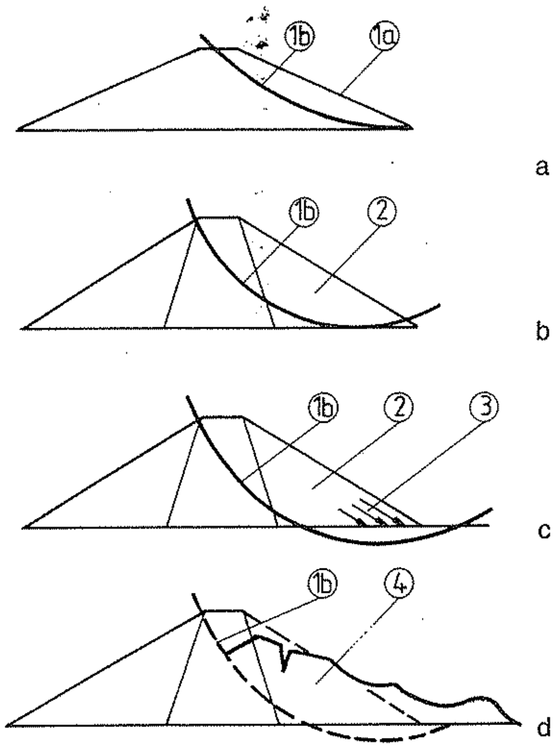


Figure 6.7. Slope stability. Location of critical slip planes.

a Dam and foundation homogeneous with equal shear strength
b Zoned dam, the shear strength of the foundation is greater than that of the dam body
c Zoned dam, the shear strength of the foundation is lower than that of the dam body
d Failure conditions
1a Critical slip plane, friction only
1b Critical slip plane, friction and cohesion
2 Shell
3 Spreading forces
4 Sliding mass

6.1.4 *Sliding stability*

The sliding stability of the dam body with reference to the foundation has to be considered, in addition to the slope stability and the stress-strain conditions. A proof is shown for straight line and polygonal failure planes in or at the surface of the foundation (Fig. 6.8). The shear resistance activated in the failure plane must be in equilibrium with the driving horizontal forces, with regard to the safety factor. The proof is based on the failure theory, as is the slope stability proof. Large displacements are attributed to the failure which

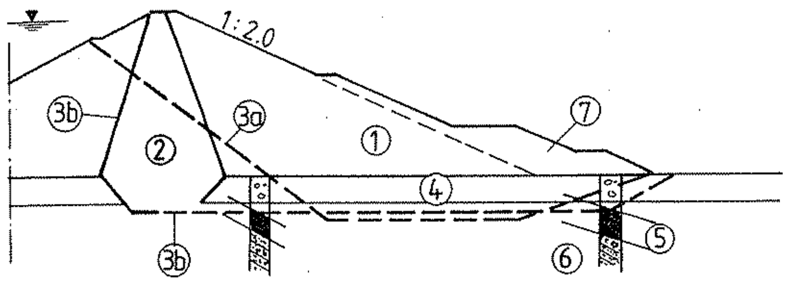


Figure 6.8. Sliding stability. Example of Mornos dam with mylonites in the foundation.

1 Downstream shell
2 Impervious core
3a Failure plane to compute slope stability
3b Failure plane to compute sliding stability of the dam body
4 Alluvions, sand and gravel
5 Mylonite
6 Stratified sandstone
7 Stabilizing berm

exceed those attributed to spreading forces, and to horizontal forces due to impounding.

A look at Figure 6.4 makes the following obvious: for the same sliding safety, the volume of a dam with internal sealing must exceed the volume of a dam with face sealing. With this type of dam, the weight of the whole dam body and the vertical component of the hydrostatic force contribute to activating shear resistance at the bottom of the dam. In contrast, for a dam with internal sealing only the weights of the downstream and the center portions of the dam contribute to shear resistance, i.e. essentially the dam portion downstream of the phreatic line which is not subjected to uplift.

6.2 DYNAMIC LOADS

Dams are subjected to dynamic loads by earthquakes. Shock waves propagate through the dam from bottom to top. The original frequency and amplitude of the waves are changed on their way through the dam, according to the dam geometry and the dynamic properties of the dam materials.

Earthquakes which are related to the particular dam location are assessed by an earthquake analysis. The analysis results in a maximum credible earthquake (MCE) and an operational basis earthquake (OBE) or design basis earthquake (DBE) which is considered to occur at least once during the lifetime of the structure. Usually the lifetime is taken as 100 years.

Every earthquake is marked by a peak ground acceleration which is transferred to the dam body at the foundation line. This acceleration and the related amplitude are magnified, in a tower-like effect, from bottom to top, while the related frequency is reduced (Fig. 6.9). Typical ground accelerations of strong earthquakes are in the range of 0.4 to 0.8 times the acceleration due to gravity *g*. The crest acceleration is in the range of 1.5 times the ground acceleration.

Earthquakes may have the following effects on structures which are incorrectly designed with respect to the foundation, the dam materials, the geometry or the zoning:

– Excessive settlement, loss of freeboard and subsequent overtopping,
– excessive displacement of the dam toes or slopes,
– local failure at the crest with the risk of subsequent overtopping,
– landslide in the reservoir with the development of a wave overtopping the dam (example Mornos dam in Section 4.2.2.1),
– shear failure at the abutments and the interface of dam and concrete

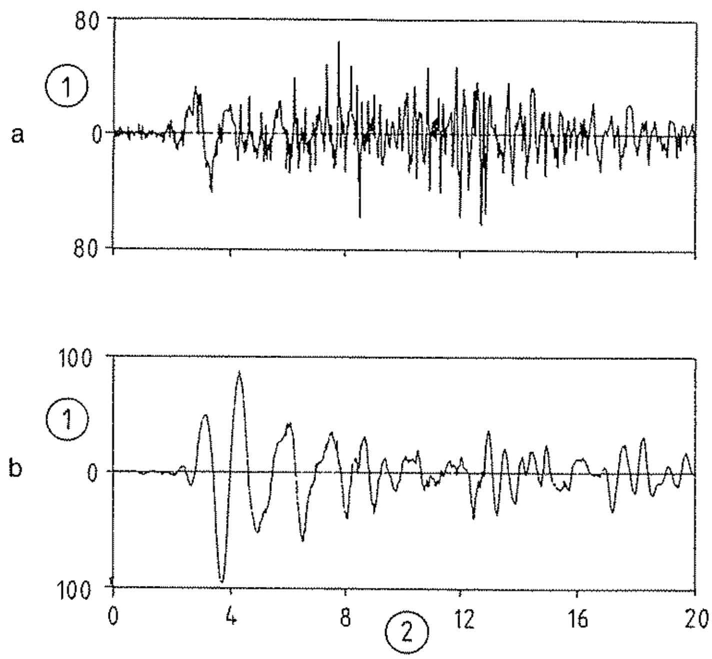


Figure 6.9. Earthquake response of a rockfill dam with earth core, 160 m in height (Kutzner & Hönisch 1985).

a Assumed design earthquake at the location of the dam,
peak ground acceleration 0.65 g, frequency 10 Hertz
b Computed resulting earthquake at the dam crest,
peak crest acceleration 0.96 g, frequency 1 Hertz
1 Acceleration (% of g = acceleration due to gravity)
2 Time (s)

structures with subsequent excessive leakage. Such failure is assumed to be related to the different natural motions of dam and abutment or dam and structure,

– damage to structures like the spillway or inspection galleries, again with the subsequent risk of excessive leakage,

– damage to face sealings at the perimetric joint with subsequent excessive leakage,

– damage to face sealings at the crest, with a risk of overtopping.

Loose saturated sands and silts might be liquefied with the loss of shear strength and related damage to the dam. From this it is obvious that liquefiable soils must be removed from the foundation and cannot be used as construction materials. Examples are known from the USA (Seed 1973, 1979).

Design criteria for earthquake-resistant dams have been developed in the last decades based on the respective state-of-the-art experience (ICOLD 1975, Londe 1983). Attention to these criteria has surely contributed to preventing earthquake damage to dams which have been completed since then. ICOLD (1983) stresses that 'it is the experience with existing dams which forms the basis for the design of new dams'.

6.3 HYDRAULIC LOADS

6.3.1 *Seepage through the dam*

Water seeps through each dam. It is one of the tasks of design and construction to make the structure functional in the sense that the water is properly drained away and that the quantity of drained water is tolerable and small. That means:

– Seeping water must not endanger the dam's stability. Erosion of fines must be excluded. Erosion might result in piping, in progressive magnification of the permeability of related dam zones, in intolerable leakage and deformation and in failure.

– The quantity of seepage must not affect human life or any activity in the downstream area, such as agriculture and water supply. The quantity must not adversely affect the economic operation of the reservoir. It is obvious that the tolerable quantity does not depend only on the conditions of the system of dam and foundation, but also on the total amount of water to be managed and on the reservoir volume.

The phreatic line in a homogeneous dam is shown in Figure 6.10, after the dam and the foundation below the phreatic line have been saturated. It cuts the downstream slope at about 1/3 of the hydrostatic head h. The location is independent of the permeability of the material which constitutes the homogeneous dam. The permeability controls the quantity of seepage, which is proportional to the permeability, according to Darcy's law. In most existing dams – as in natural sediments – the horizontal permeability is greater than the vertical permeability, in the order of $k_H = 2$ to $10 \times k_V$. In this case the actual location of the phreatic line is shifted accordingly (Fig. 6.10).

The seepage, usually, causes instability of the slope and piping. The slope must be protected and stabilized by a drainage berm as shown, or the phreatic line must be drawn down by filters inside the dam as shown in Figure 7.10. The filters must prevent passage of fines. They must be of sufficient size to allow free drainage of the seeping water.

The phreatic line in zoned dams with a vertical core is shown in Figure 6.11. The filter at the interface of core and shell must prevent erosion of

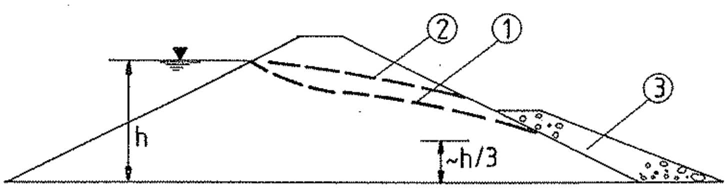


Figure 6.10. Phreatic surface in a 'homogeneous' dam.

1 Phreatic surface, $k_H = k_V$
2 Phreatic surface, $k_H > k_V$
3 Drainage berm

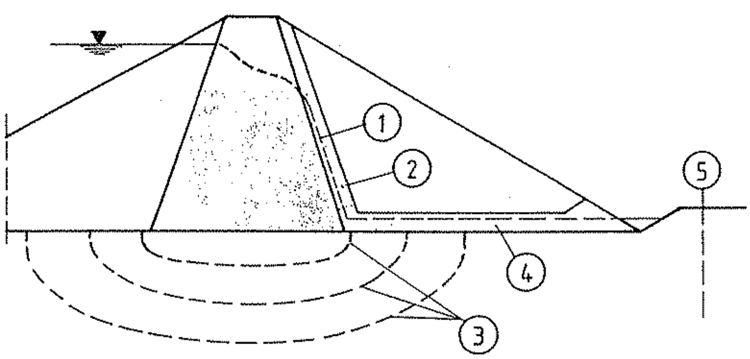


Figure 6.11. Rockfill dam with vertical earth core. Seepage through dam and foundation after impounding.

1 Phreatic line in the dam
2 Filter
3 Flow lines in the foundation
4 Filter blanket
5 Ground water observation hole

fines and guarantee free drainage of the seepage. If the shell material is less permeable than the core filter an additional filter blanket at the bottom is needed for free drainage.

With existing dams the quantity of seepage covers a wide range. For large dams it is in the order of 0.1 to 2.0 l/min and per meter of dam length, including seepage through the foundation (see the next section). As a rule, seepage does not result in critical stress and deformation conditions. Irregularities will be indicated by deformations which do not rapidly consolidate, by solids in or turbidity of the seeping water.

In membrane-like sealings there is no consistent phreatic line. Seepage is concentrated at defects and joints. Face membranes may be deteriorated by ageing or be damaged by violent attack, i.e. sabotage or war. In some countries it is standard design to enable very large quantities of water to flow through the dam without reducing the stability to critical values.

6.3.2 *Seepage through the foundation*

Water seeps through the foundation below and to the side of each dam, irrespective of artificial sealing measures such as grouting or diaphragm. Again, design and construction are liable for the functions of free drainage of tolerable quantities of seepage and of erosion stability. The filter blanket in Figure 6.11 at the bottom of the dam serves to prevent erosion of fines from the underlying material and of joint fillings. If this filter is the most permeable layer between the shell and the foundation it must be of the right size to drain the water seeping through the dam and seeping through the foundation.

Filling the reservoir leads first to saturation of the foundation and the substrata in the vicinity of the dam, then to seepage through the foundation and a rise of the ground water table downstream of the dam. A rise of several meters has been observed (Kanchanaphol et al. 1982, Kutzner 1988). Seepage through the foundation must not lead to harmful deformation of the dam and within the foundation.

6.3.3 *Reservoir operation*

At the same time as filling, the upstream portion of dams with an internal sealing starts to be saturated. The dead weight turns to the uplift weight. The process may cause settlements, as far as the water causes movements of individual particles and further compaction of the dam material. Such settlements will occur particularly with material having less strength in a saturated than in a non-saturated state. The settlements will contribute to the differential deformations between the shell and the core (Section 6.1.2).

The rise of the water level in the reservoir may cause heaving of the upstream shell due to the reduction of the vertical stresses. This process also contributes to differential deformations at the interface of shell and core. It is noted that computations, usually, reveal such heavings, since the input of such computations does not take property changes into account.

The saturation of and the water flow through the core superimpose the ongoing dissipation of pore-water pressure caused by the unconsolidated dead weight, until the hydrostatic pressure and the final phreatic line have developed. Because of the low permeability the development of hydrostatic pressure conditions covers a long period. It may take years, and will repeatedly be delayed by the fluctuation of the reservoir level due to operation.

Lowering the reservoir water level causes a water flow from the core towards the upstream shell (Fig. 6.12a). A protective filter is needed at the interface to prevent erosion. It must at least cover the section between maximum and minimum operating water levels. In conditions of normal or rapid drawdown a flow pressure *S* will develop in the shell (Fig. 6.12b) if – due to low permeability of the shell – the water level in the shell cannot immediately follow the water level in the reservoir. Then, the slope stability will considerably be reduced. The calculated reduction is 50%.

Apart from stability problems the upstream slope must be protected from erosion by seepage. Erosion stability is usually achieved by a transition layer between the shell of rockfill material or gravel and the riprap of coarse and heavy rock fragments.

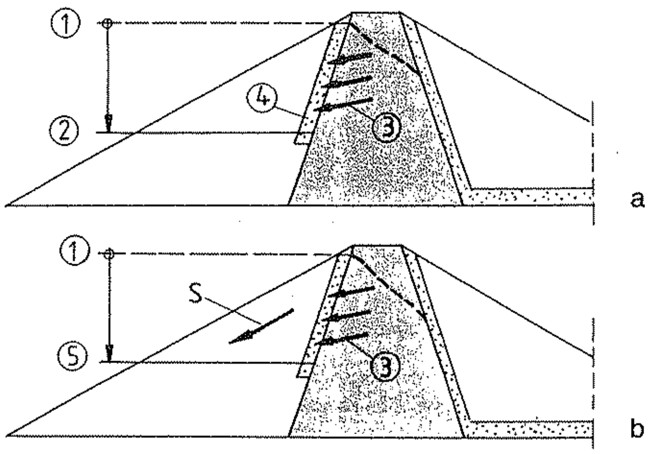


Figure 6.12. Rockfill dam with vertical earth core at reservoir operation.

a Normal (slow) drawdown of the reservoir level
b Rapid drawdown of the reservoir level
1 Max. operating water level
2 Min. operating water level
3 Flow from the core towards upstream
4 Filter
5 Target of rapid drawdown
S Seepage force during delayed drawdown within the dam body

CHAPTER 7

Design of earth and rockfill dams

7.1 GENERAL DESIGN CRITERIA

7.1.1 *Overview*

Considering the variety of potential dam locations, and the number and variability of potential construction materials, it is obvious that each dam is a unique structure. Accordingly, the design is individual for each dam, and it is not easy to define general design criteria. An attempt is made here, whereby the background of each of the criteria must, necessarily, be generalized, expressing rules of engineering which do not apply only to dam engineering but also to other engineering disciplines:

– All water-retaining dams must be impervious in a practical sense. Inevitable seepage through dam and foundation must not endanger the safety and the durability of the structure. In addition, seepage must be considered from the viewpoint of reservoir economy and environmental impact,

– the permeability of the foundation and of the abutments must be low. Otherwise those involved must establish favourable flow conditions by proven means of impermeabilization,

– the foundation area must be made regular and smooth. The shape and the properties of the foundation should not cause excessive deformation of the structure on top,

– dam zoning allows the use of different materials, each of them at the place where the properties fit best with the function,

– the incorporation of transition layers between different zones serves to bridge differential settlements between parts of the dam. Regular lines of equal stresses and deformations indicate compatible conditions,

– deformations must not lead to cracks in the sealing element. This is the more important the shorter the seepage path across the element,

– internal earth pressures in the sealing element must exceed the hydrostatic pressure of the retained water. This applies to pressures in all directions and in the center as well as in the outer zones of the element. Otherwise hydraulic fracturing and piping cannot be excluded,

– the construction materials should not be overstressed. Plastified zones are acceptable to some extent, but this is subject to individual approval.

A 'Committee on Earthquakes' (ICOLD 1975) elaborated recommendations for the earthquake resistant design of embankment dams, reflecting experiences with dams and earthquakes at that time. These recommendations are still relevant. It is noted that in the last decades earthquakes have been registered in areas which had formerly been considered as 'safe'. Necessarily, these recommendations are generalized, but it is worthwhile to present these ideas here with a short comment.

– For soft foundations the dam body and the foundation should be considered as an interacting system. Additional material at the toe of the dam will be effective as a stabilizing berm.

This recommendation refers to the non-elastic behaviour of such foundations and related excessive deformations due to earthquakes. It is noted that sealing blankets are prone to shear failure because of their geometry. They tend to develop tension cracks given large horizontal displacements.

– Sufficient foundation treatment is requested.

Measures in this respect are taken mainly to safeguard proper bond between the sealing element and the foundation. Such a bond will help to exclude piping.

– It is considered effective to make the axis of the dam slightly curved and to make the width of the dam somewhat greater towards the abutments.

This recommendation aims to protect the interface of dam and abutment from excessive leakage, which might develop due to the different natural motions and related shear movements of the two media. In addition, the curvature is deemed to create a slight arching effect which helps to compensate for potential tensile stresses at the downstream side of the crest due to the horizontal forces.

– The dam crest should be made wide.

This refers to the magnified acceleration and amplitude at the crest triggering local sliding along shallow failure planes. Shallow planes will not cut through the whole crest with the risk of overtopping if the crest is widened.

– An extra allowance of freeboard should be provided for settlement of the dam and its foundation.

This recalls the vertical component of earthquakes and the related risk of settlements leading to unacceptable loss of freeboard and subsequent overtopping.

– The construction materials must be selected very carefully.

Careful material selection will help to compose the dam of zones which are homogeneous. It is noted that maximum compaction gives maximum strength to the shells and minimum permeability to the sealing element.

– The slopes of the dam should be made gentle.

Irregularities in the outer slopes or steep and abrupt slopes might lead to unfavourable stress and strain conditions with the risk of failure.

– The thickness of the impervious zone should be increased.

The wide sealing offers additional safety against penetrating cracks. It will reduce the quantity of seepage due to the decrease of the hydraulic gradient.

– The utmost care is required in the work of construction.

This recommendation reminds us of the need for careful construction supervision to make sure that all design details are truly realized in the structure, and that the soils are made as dense as possible.

With reference to known dam failures due to sliding, overtopping and seepage, Londe (1983) proposed a '10-point plan for dam safety'. These points are equivalent to general design criteria derived from complex conditions which had caused the failures. His recommendations for safe dam design refer to the following problems (three more problems refer to the reservoir operation):

– Site investigations should watch clay soils. Residual strength parameters should be used, particularly with overconsolidated clays,

– loose, silty and sandy soils and their liquefaction potential must be considered,

– ample and well graded filters and drains are vital for preventing piping through the dam and foundations,

– the clay soils should be tested for their dispersion potential,

– instrumentation is a vital part of dam design,

– inspection galleries are invaluable for direct observation,

– thorough surveillance of dams is necessary, preferably on a 24-hour basis, together with automatic recording instrumentation.

These and other design criteria will be discussed in the following sections.

7.1.2 *Freeboard*

The freeboard is the vertical distance between the reservoir water level and the crest of the dam without camber. The freeboard serves to protect the dam from overtopping. The freeboard must take regard of the wave height, the wave run-up, the wind set-up and an ice set-up, if appropriate. According to DIN 19 700 an additional distance has to be added for safety reasons. This surplus is selected according to the reliability of hydrological data, the type of flood control structures, and the risk of settlements due to earthquakes.

The elevation of the reservoir water level is specified. DIN 19 700 sets the elevation at the water level which results from routing the design flood. Accordingly, the freeboard is the calculated distance plus the safety margin mentioned above. USBR (1973) defines a normal and a minimum freeboard.

The normal freeboard refers to the normal operating water level or full supply level. The minimum freeboard refers to the water level that results from routing the design flood which may be the probable maximum flood (PMF). For an uncontrolled spillway the minimum freeboard is always less than the normal freeboard. For a controlled spillway the two may be identical.

There is no rational formula to compute the freeboard. But there are empirical rules to assess the rates of the components which form the freeboard. Details for developing the freeboard are shown in Figure 7.1. The wave height is related to the wind velocity, the wind duration, the fetch, the angle between the wind direction and the dam axis and the reservoir depth in the vicinity of the dam. The fetch is the distance along which the blowing wind is in contact with the water surface in the reservoir. Subsequent waves of different heights and lengths following each other are evaluated to define a significant wave height h_S as a base to determine the freeboard. The significant wave height can be taken from Figure 7.2 for waves that touch the center line of the dam at 90°.

Poweleit (1985) suggests adding a safety margin to compensate for different resistance of the dam slope to wave run-up. It leads to the design wave height h_D which is:

- for the slope of rockfill dams $h_D = 1.0\ h_S$
- for the slope of earth dams:

cemented	$h_D = 1.1\ h_S$
moderately cemented	$h_D = 1.2\ h_S$
not cemented	$h_D = 1.3\ h_S$ and
average	$h_D = 1.25\ h_S$

according to Thomas (1976).

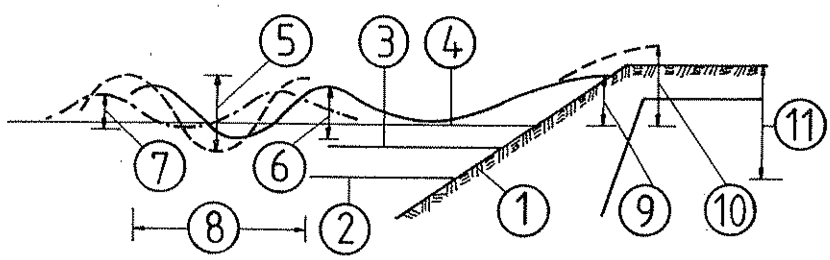


Figure 7.1. Details to develop the freeboard.

1 Embankment slope
2 Full supply level
3 Flood surcharge
4 Wind set-up
5 Maximum wave height
6 Design wave height h_D
7 Significant wave height h_S
8 Wave length
9 Run-up h_L by design wave
10 Run-up by maximum wave
11 Freeboard

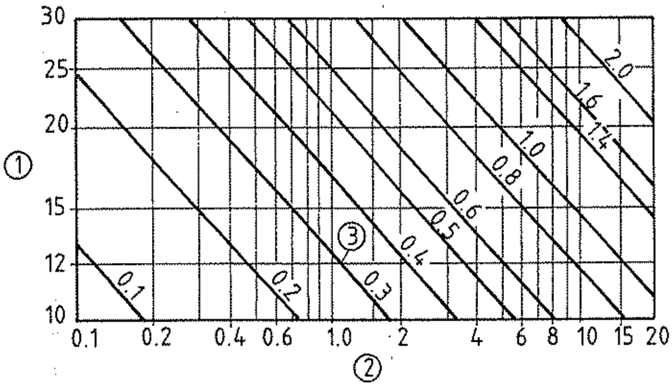


Figure 7.2. Determination of the significant wave height h_S (adapted from Poweleit 1985).

1 Wind velocity v (m/s)
2 Fetch L (km)
3 Significant wave height h_S (m)

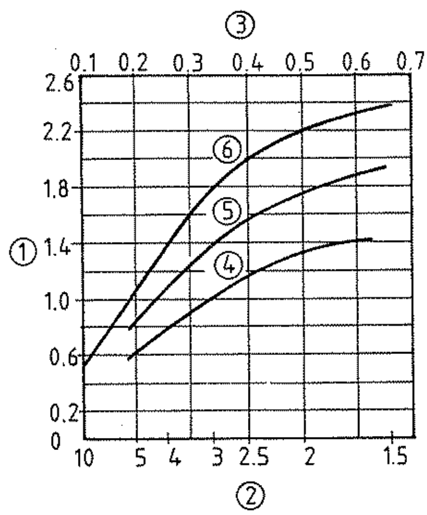


Figure 7.3. Determination of the run-up h_L by design wave h_D (adapted from Poweleit 1985).

1 h_L/h_D
2 Parameter m of the slope 1V:mH
3 Tangent of the slope angle
4 Very rough surface
5 Moderately rough surface
6 Smooth surface

The resulting wave run-up h_L can be found from Figure 7.3. As an example: for $h_S = 1.0$ m after Figure 7.2 and a rockfill dam sloping 1V:1.8H with very rough surface it is $h_D = 1.0\ h_S$, $h_L/h_D = 1.4$ m after Figure 7.3 and $h_L = 1.4$ m. The same wave would run up the moderately cemented and moderately rough slope 1V:2.0H of an earth dam by $h_L = 1.0 \times 1.2 \times 1.75 = 2.1$ m.

The wind set-up h_W is commonly determined after the empirical Zuider Zee formula

$$h_W = \frac{v^2 \cdot L}{63{,}000 \cdot T} \cdot \cos\alpha \tag{7.1}$$

where: h_W = wind set-up (m)
v = wind velocity (m/s)
L = fetch (m)
T = average reservoir depth (m)
α = angle between wind and fetch directions.

As an example: the wind set-up is 1.0 m for v = 90 km/h, L = 10 km, T = 100 m and α= 0°.

The relation of the freeboard and the top elevation of the sealing is not specified in the references. USBR (1973) points to the effect of the regional climate and says: the normal freeboard 'must be sufficient to prevent seepage through a core which has been loosened by frost action or which has cracked due to drying out'. Figure 7.4 shows a reasonable design for a rockfill dam with earth core. Flood surcharge, plus wind set-up, plus wave run-up, exceed the top elevation of the core but leave a safety surplus of 0.6 m to the crest. A short-term overtopping of the core is acceptable in many cases. The following is noted: such a design is not fully compatible with the recommendations of DIN 19 700. This specifies having the top elevation of the core above the wave run-up. The crest elevation would have to be raised by 1.0 m if the protective margin of 1.5 m between crest and core is to be

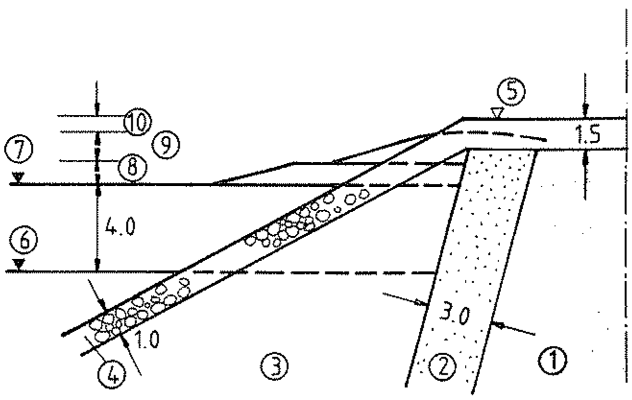


Figure 7.4. Rockfill dam with vertical earth core. Development of the freeboard (dimensions in m).

1 Dam core
2 Filter and transition zone
3 Shell
4 Wave protection
5 Dam crest, assumed at 200 m a.s.l.
6 Full supply level, assumed at 193 m a.s.l.
7 Maximum level at PMF, assumed at 197 m a.s.l.
8 Wind set-up 1.0 m
9 Wave run-up 1.4 m
10 Safety surplus 0.6 m

maintained. The design would be in accordance to DIN 19 700 if the flood surcharge of the design flood is less than that of PMF, as in Figure 7.4.

With face sealed dams the sealing should connect tightly with a wave controlling structure, as in Figure 7.6, or with the impervious pavement of the crest.

7.1.3 *Dam crest*

7.1.3.1 *Design*

The dam crest must resist all mechanical attacks and the effects of the climate. Potential mechanical attacks are wave action, ice pressure, traffic accidents and the action of animals, if appropriate. Also sabotage and war activity have to be regarded. Sabotage as an attack mainly refers to dams with face sealing.

The surface of the crest is usually prepared for mobile traffic. Figure 7.5 shows the crest of the Mornos dam as an example, with one traffic lane for each direction. The accessories, such as single or continuous curb stones, sidewalk, parking bays, guide boards, illumination etc. are selected according to local standards. The pavement is inclined upstream and drained by trenches and pipes.

The surface of internal sealings is inclined upstream for drainage. The sealing element needs a cover of coarse material as a protection from the

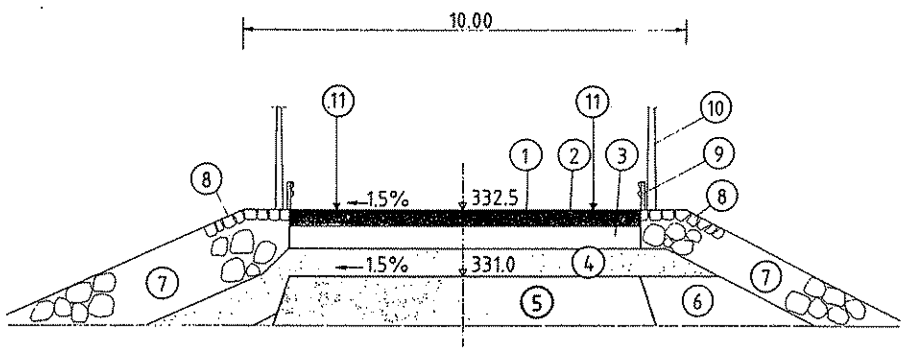


Figure 7.5. Rockfill dam with vertical earth core. Example of crest design (dimensions in m, courtesy of LI).

1 Asphaltic concrete and protective coating
2 Bituminous base layer
3 Crushed rock
4 Sand-gravel
5 Dam core
6 Filter zone
7 Riprap
8 Mortared pavement
9 Guide board
10 Lamp posts, spaced 25 m
11 White lines

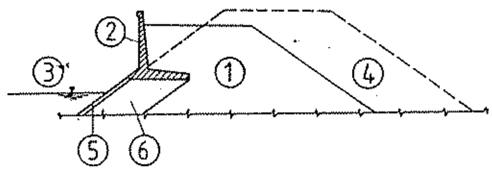


Figure 7.6. Dam crest with retaining wall.

1 Rockfill
2 Retaining wall (parapet)
3 Full supply level
4 Saving of fill
5 Concrete face
6 Transition zone

effects of weather, such as frost, drying out, moistening and weakening by precipitation. The cover is 1.0 to 2.0 m thick. The thickness should be adjusted to the frost conditions. The base layer of the road on the crest should be free of frost to prevent frost heavings and subsequent damage by traffic. In regions of deep frost penetration complete protection would lead to an extremely high freeboard. McConnell et al. (1973) report on dams in Canada with a frost penetration of 3.4 m and a selected freeboard of 2,4 m. The thickness of the protective layer on the crest is 0.6 m only.

It was proposed and agreed to stabilize the crest by a retaining wall, as in Figure 7.6. The wall protects the dam crest from wave run-up. Such a design reduces the amount of construction material considerably. It is logically used where the run-up is high, namely with concrete faced dams (as shown). The wall rests on strong rockfill material with minimum settlements after the end of construction. The design has not been used with dams with natural internal sealing. In the opinion of the author, the wall would form an undesirable foreign body on the crest of such dams.

7.1.3.2 *Width*

The width of the crest is related mainly to its purpose, to serve the traffic, as a motorway, road, agricultural or footway. Irrespective of the purpose the width should be increased with the dam height. The minimum width follows from the construction activities. At least a small compactor and a truck should be able to pass each other. This results in a minimum width of 4.0 m. In Germany the minimum width of river dikes is 3.0 m to enable maneuvering of heavy equipment for dike protection in emergency cases.

Assessments to be found in literature lead to the widths shown in Figure 7.7. The selected width should ensure that local slidings due to earthquakes do not cut the whole crest. Respective failure planes should be restricted to one side only and leave enough space of the crest undamaged to exclude overtopping. The acceleration at the crest is about 1.5 times the peak ground acceleration.

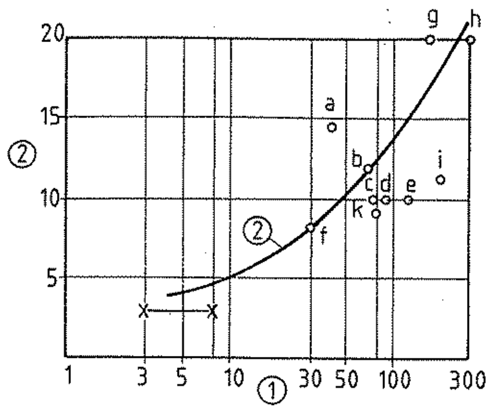


Figure 7.7. Average crest width.

1 Dam height (m)
2 Crest width (m)
a Aabach
b Alcova
c Kinda
d Teton
e Mornos
f Shen river
g Chico
h Nurek
i Bakun 1986, 1996
k Frauenau
x-x River dikes in Germany

7.1.3.3 *Camber*

The camber serves to compensate for crest settlements after the end of construction. Such settlements may originate from the dam body and from the foundation. The camber should be high enough to exclude any loss of freeboard after complete consolidation. A permanent slight camber from the abutments towards the center is desirable for aesthetic reasons.

Examples of crest settlements of dams with natural and with artificial sealings are listed in Table 7.1. The data comply with the expected behaviour of the dams:

– The settlements of natural internal sealings are maximum. The rate decreases with the plasticity of the material. The total settlements until the end of construction – not at the crest – make about 2 to 3% of the dam height *H*. The remaining settlements, due to further consolidation, impounding and long-time reservoir operation, are commonly not more than 0.01H.

– The settlements of dams with face sealing are smaller because the rockfill or equivalent material is less compressible than cohesive core material. The remaining crest settlement will not exceed 0.005H.

– Rockfill dams with artificial internal sealing show minimum settlements. This is because the vertical component of the water load does not contribute to the consolidation of the upstream shell, as it does with face

Table 7.1. Examples of crest settlements.

Project	Sealing element	Construction material	Deformation modulus (MPa)	Settlements until end of construction[1] (cm)	Settlements until end of impounding (cm)	Settlements during operation (cm) (years)	References
Peruca	Earth core	Clay	Highly plastic			86 (25) 0.014 H	Nonveiller (1987)
Kinda	Earth core	Silt, sandy, clayey	PI = 15 to 30	130 0.02 H	42	46 (1.5) 0.006 H	Archives LI
Canadian dams	Earth core	Moraine	Non-plastic			(2 to 2.5) <0.0035 H	Dascal (1987)
Foz do Areia	Concrete face	Basalt ≤ 600 mm	30 to 50	358 0.02 H	42	45 (1.5) 0.003 H	Pinto et al. (1982)
Alto Anchicaya	Concrete face	Hornfels ≤ 600 mm	100 to 170		6	14.3 (7) 0.001 H	Regalado et al. (1982)
Finstertal	Asphaltic concrete core[2]	Granodiorite ≤ 700 mm	100 to 150	25 to 30 0.0025 H	6	10 (3) 0.0008 H	Schwab (1984), Pircher & Schwab (1988)
Wupper	Asphaltic concrete core	Siltstone ≤ 400 mm	40 to 60	1.0	0.2	1.2 0.003 H	Archives LI

[1]Maximum settlement inside the dam
[2]Dam height 120 m

sealed dams. Remaining settlements after the end of construction should be considered as 0.001 to 0.003H, related to the rockfill quality.

These considerations fit with the conditions of an incompressible foundation. Expected foundation settlements must be taken into account in relation to the camber. The assessment of the appropriate camber should be made in the course of the design work. It should be adjusted to requirements when the actual construction settlements can be evaluated. The camber is compensated by steepening the outer slopes in the uppermost section of the dam. As an example: a design slope of 1V:1.8H is steepened to 1V:1.5H at the uppermost 5 m of the dam to compensate for 1.0 m of camber. The steep slope will be partially re-adjusted after complete consolidation of the dam and its foundation.

7.1.4 *Curvature of the dam axis*

The value of a curvature of the dam axis in plan is under discussion. It will not be possible to evidence the advantages which are seen by a number of experts. It may be worthwhile to list these advantages because they are, in principle, agreed, but without evaluation. The curvature constitutes:

– Favourable stress distribution on the downstream side, by establishing compressive strains, fully or partially, instead of tensile strains. Tensile strains might lead to cracks in the upper portion of an internal natural sealing.

– Favourable conditions at the abutments, reducing the risk of excessive leakage after different movements of the dam and the abutment (ICOLD 1975).

– Favourable stress distribution in face sealings. Related compressive stresses will help to maintain vertical joints intact. The neighbouring slabs of concrete faces may damage each other if the radius of the curvature is too small. Face sealings of asphaltic concrete may tend to local buckling.

In addition, a slightly curved axis is more aesthetically appealing than a straight line axis.

Frequently used radii are in the range of 1000 to 3000 m, the center of the circle being downstream. The radii should be adjusted to the topographical conditions. Also, reverse curvatures have been used with the center upstream. Such curvature applies to all artificial pools, as, for instance, pump storage plants. There the radii are usually less than 100 m.

7.1.5 *Slope protection*

7.1.5.1 *Upstream slope*

The upstream slope must be protected mainly from wave action. Other attacks are the weight and the action of ice. The protective layer covers the slope from the crest to the elevation of the minimum operating water level with an addition of a few meters to compensate for wave suction. It is useful to place the lower end at a berm as, for instance, the cofferdam crest to protect it from undermining during the first filling.

The common material for wave protection is riprap which is a graded material of heavy rock fragments. The effectiveness and the durability depend mainly on the rock strength, on the composition in terms of size, weight and shape of the individual pieces and on the durability of the slope below the protective layer. Angular fragments interlock better than rounded fragments.

The material must be tight, strong and durable. Igneous and metamorphic rock and strong limestone are good materials. The uniaxial compressive strength should be at least 100 MPa. The quality requirements and related tests are the same as those for concrete aggregates. Frequently, it is a problem and it is costly to provide proper material. Long hauling distances must be taken into account. For such cost reasons the thickness of the protective layer should be carefully designed.

The thickness follows mainly from the intensity of the wave action. Accordingly, the literature gives empirically selected dimensions in relation to the wave height (Taylor 1973, Iverson & Ringheim 1973, McConnell et al. 1973), or to the fetch (USBR 1973). The data of Table 7.2 for the design of protective layers are developed on the base of these references. The weight of the smallest pieces should not be less than one fourth of the average weight. The weight of the largest pieces should be up to four times the average. The number (not the weight) of pieces with an edge length more than the average should be 50% of the total.

The maximum wave height given in Table 7.2 is about 1.8 times the significant wave height according to Figure 7.2. The layer thickness indicated in

Table 7.2. Dimensions of riprap for wave protection on slopes up to 1V:1H.

Maximum wave height (m)	Layer thickness (cm)	Mean values of individual pieces (kg)	(cm)[1]	Maximum values of individual pieces (kg)	(cm)[1]	(cm)[2]
< 0.6	40	20 to 50	20 to 25	80 to 100	30 to 35	40
0.6 to 1.2	50	50 to 100	25 to 35	150 to 300	40 to 50	55
1.2 to 1.8	60 to 70	150 to 250	40 to 45	400 to 800	55 to 65	75
1.8 to 2.4	80 to 90	250 to 500	45 to 55	1000 to 1500	70 to 85	95
2.4 to 3.0	90 to 120	500 to 1000	55 to 70	1800 to 2500	85 to 100	115
> 3.0	≥ 120	1200 to 1500	75 to 85	3000 to 4000	100 to 115	135

[1]Equivalent edge length of cube, unit weight = 27 kN/m³
[2]Equivalent diameter of spherical pieces of mean max. weight, unit weight = 27 kN/m³

Table 7.2 should be increased if mainly rounded pieces are used. This is to compensate for the reduced interlocking effect of rounded material. Accordingly, the equivalent diameters of rounded pieces in the last column of Table 7.2 are slightly greater than the indicated layer thicknesses. The thickness may be reduced if the individual pieces are placed like a mosaic, by hand or by the bucket of an excavator.

Durable conditions of the base below the protective layer require that the wave action does not wash particles of the base into the voids of the protective layer. If a transition layer is needed it must be designed according to proven filter rules. Usually it is a layer of crushed rockfill material or gravel, 5 to 100 mm in diameter or edge length. Such a transition layer is not needed between typical rockfill material and typical riprap, as for instance no 11 in Figure 7.39. It may also be dispensed with homogeneous dams and moderate or slight wave action, due to the cohesion of the dam material. An example is the Shen river dam (Fig. 5.1). Dam bodies of sand, gravel or slightly plastic materials will always need a transition layer.

Occasionally, the protective layer is stabilized by mortar. In this case the base layer must be drained downstream or through permeable, un-mortared areas of the top layer. Such drainage is required to prevent the development of hydrostatic pressure behind the top layer after drawdown operations.

Other kinds of upstream slope protection are pavements of concrete, asphaltic concrete or soil-cement. Details of soil-cement can be obtained from ICOLD (1986c). Such pavements need careful drainage of the base. Their maintenance over several decades is reported to be elaborate and costly. Protection against extreme wave attack is effected by solid concrete bodies, such as tetrapodes. On the contrary, for small dikes which are not permanently exposed to water and waves, a grass cover may be sufficient.

7.1.5.2 *Downstream slope*

The downstream slope must mainly be protected from damage and erosion by rainfall. Other attacks may be caused by digging animals and wind. If appropriate, a protective layer must be placed from the dam crest to the toe or to the minimum tail water level, again with a surplus for wave suction.

The most usual protective layer is a grass cover. The type of grass must be selected according to the climate and the type of dam material. In zones of temperate and cold climate a layer of topsoil is needed to allow the establishment of a dense and durable grass cover. The layer thickness should not be more than 10 to 20 cm to enable the roots of the grass to create a good bond between the topsoil and the dam. Details of plantation and maintenance can be seen from Sykes & Doister (1979). Informative booklets from suppliers explain how to make the slope rapidly green with the use of stabilizers. Many tropical soils used for dam construction can be seeded without spreading a topsoil layer.

An alternative to a grass cover is a layer of rockfill material up to about 25 cm in edge length. The layer thickness should not be less than 0.3 m, but dumping and levelling of 0.5 m may be easier.

The downstream slope must be carefully drained. It is usual to establish berms at about 10 m vertical spacing or – better – to connect the berms to constitute a continuous road from the dam toe to the crest. The road serves for inspection and dam maintenance. Berms and road cut the slope in sections which are individually drained. A drainage ditch or pipe must be incorporated along the dam toe to collect the water from the slope and from the blanket filter, or from the bottom of the dam if no consistent filter is existing. This ditch has also to collect the water which seeps from the abutments close downstream of the dam to the bottom. It is mentioned that respective abutment areas are usually stripped of vegetation, being an unprotected catchment area of the toe ditch.

Planting of low bushes on the downstream slope is acceptable provided the roots do not penetrate that part of the slope which is required for slope stability. Bushes should be widely spaced. The lower third of the slope should not be planted, to enable easy visual control of leakages.

7.2 HOMOGENEOUS DAMS

Homogeneous dams without an individual sealing element consist of cohesive soil of low permeability. A typical dam section is shown in Figure 5.1. The outer slopes are made according to the shear strength of the dam material and of the foundation.

An example is the Shen river dam (Fig. 7.8). The dam material is laterite

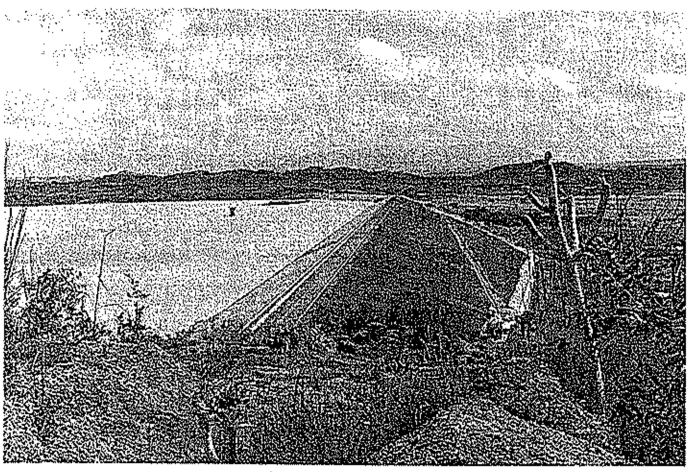


Figure 7.8. Shen river dam 1979 (courtesy of LI).

which is a typical tropical soil. The foundation consists of slightly weathered granite. In soil mechanical terms the laterite is a residual plastic clay (CL), a weathering product of the granite. The volume of the dam is about 1.7×10^6 m^3. It had to be constructed within one dry season from October to March 1978/1979, after foundation preparation and the completion of the major part of the grouting works. The selected method of compaction (see Section 4.2.6.2a) enabled it to meet the schedule. The average production rate was 10,000 to 12,000 m^3 per day. In the middle part this corresponds to about one layer per day over the whole dam area. There was no problem from the development of excess pore-water pressure, due to the construction progress and the properties of the soil, which are given in Table 7.3.

The contractor was able to place the protective layers on both slopes prior to major rainfall. This prevented the slopes from being eroded. The layers consist of riprap upstream and topsoil and grass downstream (Fig. 7.8). There is no transition layer placed between riprap and laterite, though the material's grain size compositions do not meet the geometrical criteria of filter stability. The transition layer was omitted due to the cohesion of the laterite. This was justified by experience with similar dams in the region.

A contrasting example is shown in Figure 7.9. It is typical for tropical areas with heavy rainfall. The unprotected slope was eroded in the construc-

Table 7.3. Examples of tropical soils used to construct homogeneous dams.

Project		Shen river[1]	Agus IV[2]	Punchina[3]
Type of soil		Inorganic clay (Laterite)	Plastic silt (Laterite)	Sandy silt (Saprolite)
Parent rock		Granite	Basalt	Diorite
Group symbol		CL	MH	SM
Content < 0.002 mm	(% by weight)	20 to 30	20 to 40	6.4
Content < 0.074 mm	(% by weight)	40 to 60	40 to 55	42.7
Max. grain size	(mm)	2	3	6
Liquid limit	(%)	25 to 45	70	35.6
Plasticity index		10 to 20	35	10.2
Specific gravity	(kN/m^3)	26.9	27.0	27.4
Standard Proctor density	(kN/m^3)	17 to 19	11.8	16.5
Opt. water content	(%)	13 to 20	44.2	20.1
Natural water content	(%)	20 to 24	44 to 47	24.2
Mean fill water content	(%)	opt. + 2	opt.	opt. + 4
CD-angle of friction	(°)	30	34	31
CD-cohesion	(kPa)	20	60	15
Permeability, compacted	(m/s)	5×10^{-10}	10^{-9}	$\leq 10^{-7}$

[1]Kutzner (1982a)
[2]Archives LI
[3] Villegas (1982)

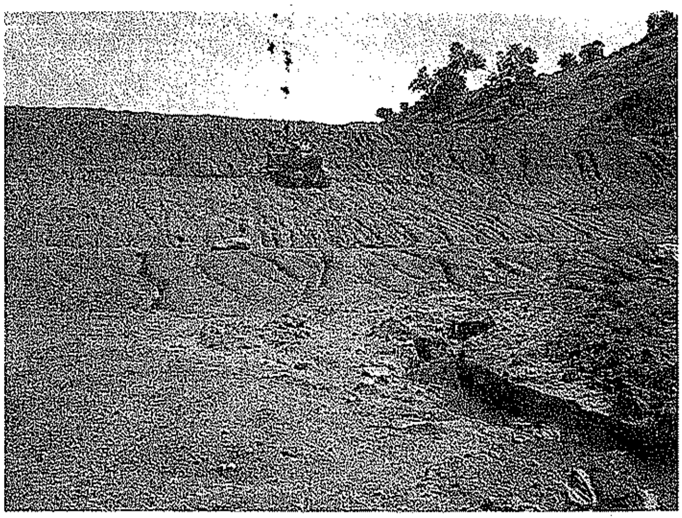


Figure 7.9. Homogeneous dam of laterite. Erosion gullies at the slope (India 1990).

tion break during the rainy season. Existing gullies must carefully be refilled. The material must be selected so as not to form new drainage trenches leading to repeated erosion.

Seepage through the Shen river dam is controlled by a filter blanket (Fig. 5.1). Other, more frequently used filter arrangements are shown in Figure 7.10. The composition and dimensions of filters are described in detail in Section 7.3.3. For the Shen river dam the location of the phreatic surface after long-time impounding and hence the length of the filter was analyzed by use of an electro-analogous model. The analysis comprised different horizontal and vertical permeabilities. Such inhomogeneities (precisely: anisotropies) are always observed with so-called homogeneous dams. Unfavourable conditions should be checked using such analyses.

The author was able to follow up the seepage conditions until 20 months after start of impounding and about 15 months of full supply level (Kutzner 1982a). At that time the location of the phreatic surface in the inner downstream dam portion was 16 m below the computed final location. This indicates low permeability and fairly isotropic conditions.

As is known, the development of the final phreatic surface and the first appearance of seeping water at the downstream toe or in the filter depend on the degree of saturation of the penetrated material. In fully saturated material the phreatic surface will immediately reach its final location corresponding to the reservoir level, and seepage will occur in the filter. The development is delayed by air filled pores of the dam material (Mallet & Pacquant 1951). With the Shen river dam, the development of final conditions was obviously

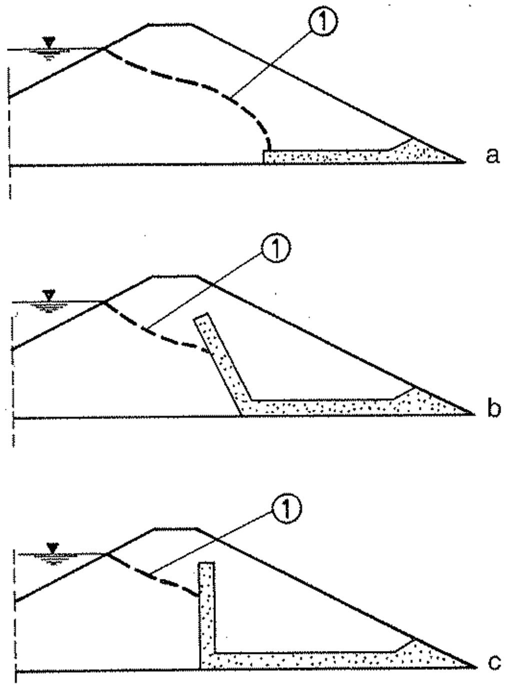


Figure 7.10. Homogeneous dam. Arrangement of filters.

a Filter blanket
b Inclined filter
c Chimney filter
1 Phreatic surface

delayed by more than 15 months, due to the conditions of permeability, isotropy and the degree of saturation.

In contrast, the permeability of the foundation is marked by joints. Saturation of the joints followed shortly after the start of impounding. So, water seeping through the grout curtain was quickly observed in the piezometers downstream. The flow rate in the range of 1 to 2 l/min in two piezometers close to the valley section of the dam was tolerable.

Laterites as a tropical weathering product of igneous rock have proven to be good dam construction materials. There is a considerable number of homogeneous dams made of laterites. It is noted that the soil mechanical properties of laterites from different tropical regions vary within a wide range. Table 7.3 gives examples. The low density and the high optimum water content of the soil from Agus IV in the southern part of the Philippines should be noted, irrespective of the high shear strength and plasticity. It is known from other types of laterites that the values of the liquid limit and the plasticity index depend on the method of drying and wetting of the material for testing (Gidigasu 1976, Morin & Todor 1976).

Villegas (1982) reports on stability problems during construction due to pore-water pressure. The 45 m high Punchina cofferdam (Fig. 7.11) was constructed of sandy silt with the properties given in Table 7.3. The soil be-

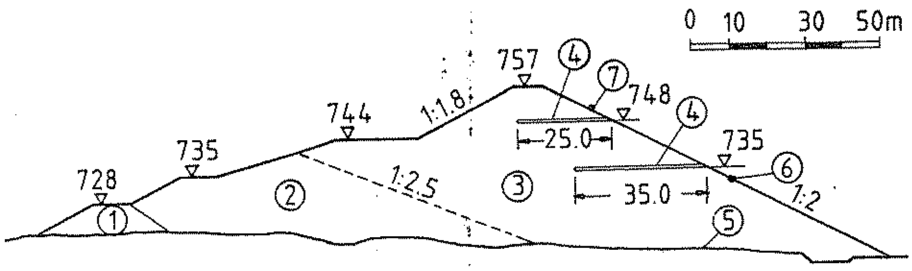


Figure 7.11. Punchina cofferdam, simplified (elevations m a.s.l., adapted from Villegas 1982).

1 Rockfill dike
2 First stage cofferdam
3 Compacted sandy silt
4 Drainage blankets
5 Final foundation surface
6 Location of max. horizontal displacement (1.5 m)
7 Location of max. settlement (0.45 m)

longs to the group of saprolites which are also tropical weathering products of igneous rock. Saprolites exist in many tropical regions as residual soils.

Construction progress had to be adjusted to the weekly precipitation. The cofferdam was to be completed prior to the beginning of the rainy season in April 1980, so only 60 to 75 days could be used. The risk of dam failure emerged due to high pore-water pressure in the upper portion of the downstream slope. Displacements up to 1.5 m horizontal and 0.45 m vertical occurred, at pore pressures in the center portion of 60 to 70% of the dead weight (Fig. 7.11).

7.3 DAMS WITH NATURAL SEALING

7.3.1 *Dam zoning*

Dam zoning aims to guarantee the dam's safety with respect to stability, seepage and cracking. Frequently, different sizing and shaping of the zones results in the same safety. The selection is made with regard to the availability of the materials and their most economic handling. The technically best zoning results in a dam section which is marked by the following:

– The permeability increases from the center, both upstream and downstream,

– the shear strength of the materials increases in the same sense.

This principle is very clearly followed in the design of the Aabach dam (Fig. 5.2). It is to be seen in many rockfill dams with internal natural sealing, where the two shells consist of the same material, for instance the Kinda dam, Figure 5.4.

Designers may be obliged to modify such simple zoning for reasons of material availability and economic handling, without any loss of safety. One step towards economic construction is dividing the downstream shell into an inner zone of very low deformability and an outer zone of slightly more deformability, as can be seen in Figure 5.5 and described in Section 6.1.2. The materials have the same strength. They are compacted at different layer thicknesses, for instance 0.8 m for the inner zone and 1.2 m for the outer zone, to accelerate the construction progress and to decrease the compacting impact in the outer zone. This is justified because the stresses in the outer zone do not increase after impounding, in contrast to the stresses in the inner zone. The principle of increasing permeability from inside to outside is maintained.

The downstream shell is often formed of materials of reduced quality which are available from excavations. Reduced quality is reflected by increased deformability or decreased permeability, both in comparison to the properties of high quality rockfill material. An example is the 160 m high Talbingo rockfill dam (Fig. 7.12). The inner zone of the downstream shell consists of high quality rockfill material, the outer zone of material of lesser quality. In such cases it is recommended to confine this outer zone by a layer of high quality material at the slope. This also serves for protection from erosion. The zone of high quality rockfill is enlarged near the crest to cover the safety requirements in case of earthquakes. It is enlarged also at the dam base to establish high shear strength there.

Another example is the previous tender design of the Bakun rockfill dam (1986), 200 m in height (Fig. 7.13). The outer zone of the downstream shell was designed to consist of greywacke and up to 50% mudstone, the inner zone of 100% greywacke. The outer zone was confined by high quality ma-

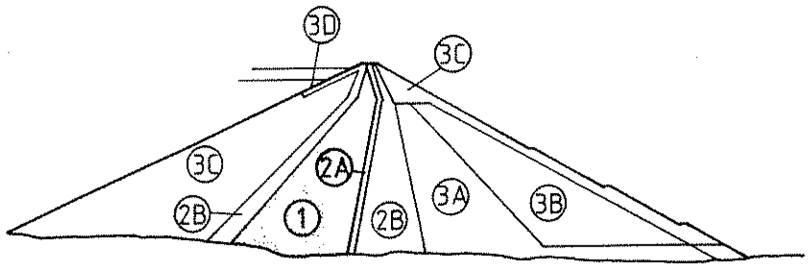


Figure 7.12. Talbingo rockfill dam with inclined earth core (adapted from Wallace & Hilton 1972).

1 Impervious core
2A Processed fine filter
2B Natural coarse filter
3A High quality rockfill
3B Lesser quality rockfill
3C High quality rockfill
3D Riprap

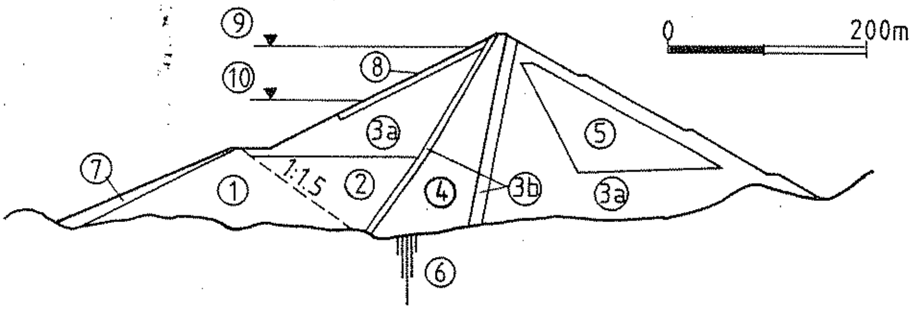


Figure 7.13. Bakun rockfill dam with inclined earth core. Previous tender design 1986 (courtesy of LI).

1 Cofferdam of greywacke with up to 30% mudstone
2 Greywacke with up to 30% mudstone
3a High quality rockfill, greywacke
3b Transition zone and filter, crushed greywacke
4 Core, weathered mudstone
5 Greywacke with up to 50% mudstone
6 Grout curtain
7 Face sealing of the cofferdam, impervious soil
8 Riprap
9 Full supply level
10 Min. operating water level

terial of around 12 m thickness normal to the slope and at least 10 m thickness at the base. Due to the location of the minimum operating water level and the low earthquake risk of the region, making the upstream cofferdam and the lower portion of the upstream shell of lesser quality material than 100% greywacke was justified. The greywacke was blended with up to 30% mudstone. This design was best adjusted to the available materials.

An extreme example to demonstrate the use of different materials, and hence the variability of dam zoning, is the tender design of the Chico rockfill dam, 160 m in height, shown in Figure 7.14. The background is described in Section 4.2.6.1. The dam was to be constructed in a region of the Philippines which is highly prone to earthquakes. The peak ground acceleration of the design earthquake was taken as 0.65 g. A large amount of construction materials was to be taken from excavations for economic reasons. It is a mixture of sandstone and disintegrated siltstone (Figs 4.13 and 4.32) with a permeability in its most compact state in the range of 10^{-7} to 10^{-6} m/s, i.e. close to the permeability of the sealing material. Seepage must be drained exclusively by the filters at the downstream face of the core and at the bottom of the shell.

The upstream dam body is subjected to varying stress and strain condi-

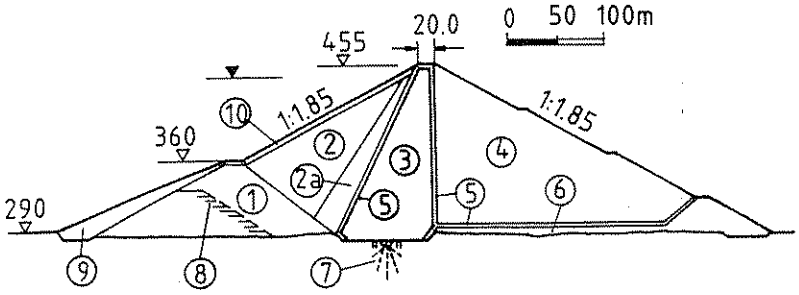


Figure 7.14. Chico rockfill dam with earth core, tender design 1984 (courtesy of LI).

1 Cofferdam of high quality rockfill, 2 stages
2 High quality rockfill
2a Transition zone of high quality rockfill
3 Core, impervious soil
4 Disintegrated sandstone and siltstone (no 10 in Fig. 4.27)
5 Filter
6 High quality rockfill
7 Inspection gallery and grout curtain
8 Reinforcement of 1st stage cofferdam
9 Face sealing of cofferdam, impervious soil
10 Riprap

tions, due to the reservoir operation. Therefore, the use of different materials there is limited. Earthquakes demand free-draining material all over the upstream shell, at a permeability of $k \geq 10^{-2}$ m/s.

In regions of low earthquake risk the upstream shell may be designed as shown in Figure 7.15, provided there is no free-draining material available or it is too costly. The major part of the upstream shell consists of semi-pervious material. Drainage of the upper section between full supply level and drawdown level is supported by highly permeable drainage layers. The outer shell must consist of free-draining material to achieve safety at conditions of rapid drawdown. The interface of the zones (2) and (3) must meet common filter rules.

An example is the 68 m high Alcova zoned earthfill dam (Fig. 7.16) with its semi-pervious zone upstream of the sealing and a thin protective rockfill layer on the flat slope. After a rapid drawdown operation noticeable pore pressures have been measured in the semi-pervious zone. Drain layers in this zone would enable it to steepen the outer slope.

A particular case of dam zoning emerges from an integrated upstream cofferdam (Fig. 7.17). For economical reasons such a design should be adopted whenever possible. The steepest inner sloping of the cofferdam is 1V:1.3H (Fig. 7.18). For a friction angle of 40° the calculated safety is 1.1 and

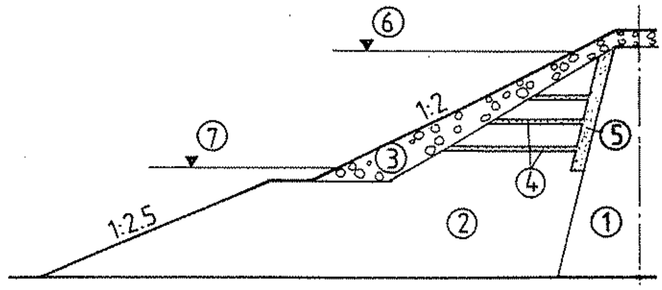


Figure 7.15. Scheme of upstream dam zoning for regions of low earthquake risk.

1 Impervious zone
2 Semi-permeable zone
3 Free-draining zone
4 Drainage layers
5 Filter
6 Full supply level
7 Water level after rapid drawdown

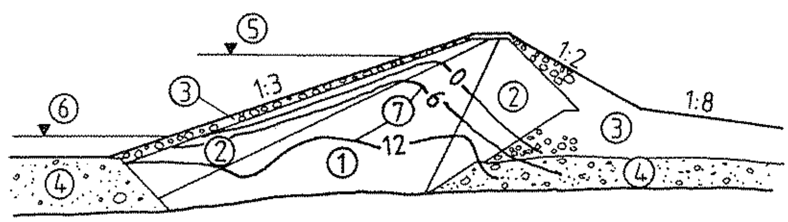


Figure 7.16. Alcova zoned earthfill dam (adapted from USBR 1973).

1 Impervious zone
2 Semi-permeable zones
3 Free-draining zones
4 Alluvial river deposits
5 Full supply level at 1655 m a.s.l.
6 Water level after rapid drawdown at 1619 m a.s.l.
7 Lines of equal pore pressure after rapid drawdown (pressures in m of water head)

slightly below 1.0 under moderate earthquake load. Such safety factors will usually be tolerable for the cofferdam slope.

The inner toe of the cofferdam must be sufficiently apart from the dam core to enable proper establishment of the adjacent zones. The distance of the cofferdam from the core may be equal to the width of the transition zone. A cofferdam crest 10 m in width as a permanent construction road allows a cofferdam 48 m in height if the main dam is 100 m high, according to Figure 7.17. A higher cofferdam or another slope of the main dam demands a wider cofferdam crest. As an example: it is 42.5 m wide with the configuration shown in Figure 7.13. There, the cofferdam is around 90 m in height.

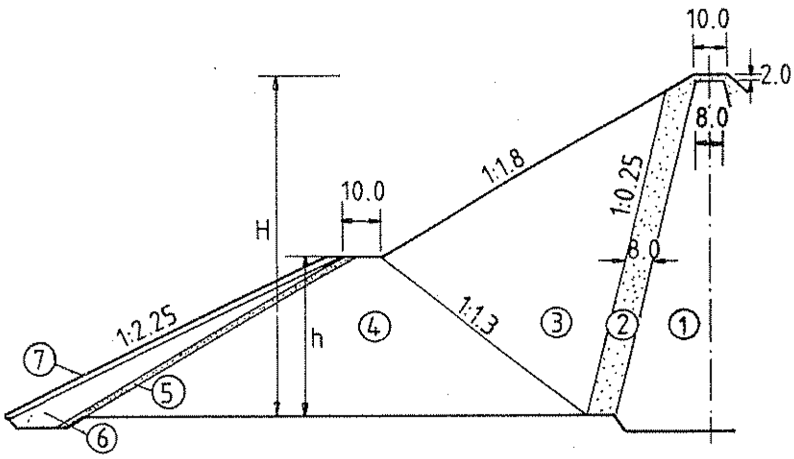


Figure 7.17. Scheme of upstream zoning of rockfill dam with vertical core and integrated cofferdam (dimensions in m).

1 Dam core
2 Transition zone and filter
3 Shell
4 Cofferdam
5 Filter
6 Face sealing
7 Wave protective layer

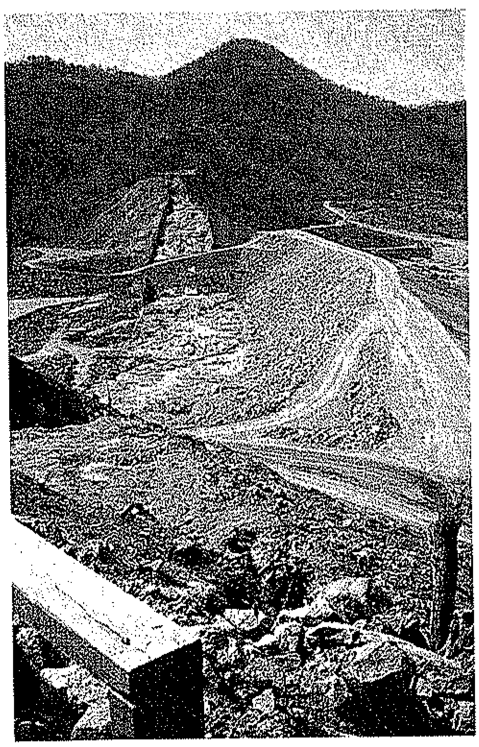


Figure 7.18. Kinda project. Photograph showing the cofferdam with its downstream slope 1V:1.3H and the core trench excavation in 1982 (courtesy of LI).

7.3.2 *Sealing elements*

7.3.2.1 *Internal sealing of soil*

At first the position of the sealing within the dam body must be considered. The axis of the sealing may vary between the vertical and several inclined positions. The vertical core, arranged symmetrically to its axis, yields sufficient core width without flattening the outer slopes. A wide core constitutes a long seepage path which is particularly desirable at the interface of dam and foundation. There, two completely different media, namely cohesive soil and rock, have to come to tight contact. The width should be selected so as not to exceed a hydraulic gradient of $i = 2$.

The internal vertical and horizontal stresses in the core must at any point be greater than the hydrostatic pressure of the water in the reservoir. This applies also to the interfaces of dam and foundation and structures. Otherwise hydraulic fracturing and subsequent erosion and piping might occur. The horizontal stresses are critical. They amount to only a part of the vertical stresses. Therefore, high vertical stresses are desirable.

The distribution of stresses follows from the geometry of the core and the adjacent zones and from the properties of the dam materials. This was clearly explained by Penman (1982), using the model of a narrow rectangle supported by shear forces along its vertical sides. The vertical stresses are:

$$\sigma_v = h\left(W - \frac{c_u}{a}\right) \tag{7.2}$$

where: h = vertical height above the base
W = unit weight of the core material
a = half width of the core
c_u = undrained shear strength of the core material.

The horizontal stresses are:

$$\sigma_h = K_0\,(\sigma_v - u) + u \tag{7.3}$$

where: K_0 = coefficient of earth pressure at rest
u = pore-water pressure.

The vertical stresses are maximum for $c_u = 0$ and $a = \infty$:

$$\max \sigma_v = W \times h \tag{7.2a}$$

The horizontal stresses are maximum for full pore-water pressure ($u = \sigma_v$):

$$\max \sigma_h = \sigma_v \tag{7.3a}$$

They are minimum after complete consolidation ($u = 0$):

$$\min \sigma_h = K_0 \times \sigma_v \tag{7.3b}$$

For better understanding, a numerical evaluation of Equations (7.2) and (7.3) is demonstrated in Figure 7.19. The following can be concluded:

– The vertical stresses increase with the core width. The greater the shear strength the more obvious is this tendency,

– the vertical stresses increase with decreasing shear strength. That means, low shear strength is desirable which is related to higher water contents of the core material. At $c_u = 0$ the core width does not affect the vertical stresses,

– the horizontal stresses increase with the vertical stresses and with the coefficient of earth pressure at rest,

– the horizontal stresses increase with the pore-water pressure. This points also to the advantage of a high fill water content.

The diminishing factor c_u/a of Equation (7.2) may cause inadequate ver-

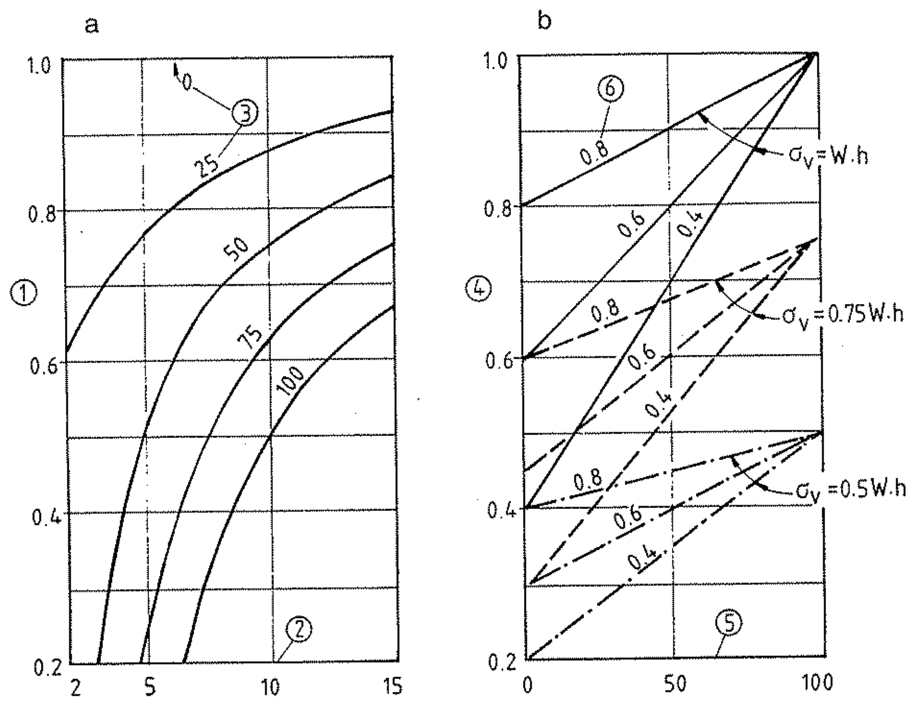


Figure 7.19. Computed stresses due to dead load in a rectangular core section with vertical outer faces, according to Equations (7.2) and (7.3) after Penman (1982). Unit weight of the core material 20 kN/m^3, dam height 50 m.

a Vertical stresses, Equation (7.2)
b Horizontal stresses, Equation (7.3)
1 Vertical stress σ_v (MPa)
2 Half width of the core (m)
3 Undrained shear strength c_u (kPa)
4 Horizontal stress σ_h (MPa)
5 Pore-water pressure u (% of σ_v)
6 Coefficient K_0 of earth pressure at rest

tical stresses and related low horizontal stresses in the lower portion of the core. In addition, the upper portion may laterally 'hang up' by friction at the interface of core and adjacent zones while the lower portion still settles down. This arching effect may lead to horizontal cracks.

The tendency towards arching can be reduced by sloping the faces and hence widening the core. The usual slopes of symmetrical vertical cores are 1V:0.25H to 1V:0.4H. Such sloping does not noticeably affect the stability of the outer dam slopes. Many existing dams show an excess sloping of the core at the lower portion, e.g. Figure 7.20.

The core may be narrowed asymmetrically if the quality of the material and the foundation conditions permit, as designed with the Chico dam (Fig. 7.14) and realized with the Mica dam (Fig. 7.20). Respective steepening of the face is made downstream to maintain the favourable effect of the inclination upstream. Such a design may be used also where there is a lack of core material or because of construction reasons (see below). The designers of the Mica dam concluded (after Webster 1970):

– A near vertical, near central core provided the greatest stability during earthquake loading,

– deformations during earthquake loading would be less, and less likely to be serious,

– the rockfill zones were large enough to permit rock placing when work on the core was stopped by inclement weather.

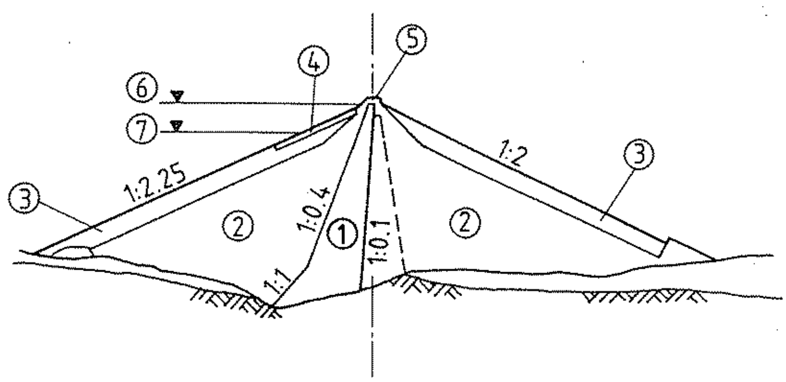


Figure 7.20. Mica gravel dam with earth core (simplified after Webster 1970).

1 Glacial till in 25 cm layers
2 Sand and gravel in 30/45 cm layers
3 Outer shell, sand and gravel or rock in 60 cm layers
4 Drawdown zone; gravel, cobbles, boulders or rock in 60 cm layers
5 Crest width 11 m
6 Full supply level
7 Min. operating water level

According to Thomas (1976) 'the core was made wider to reduce the possibility of propagation of any transverse or longitudinal cracks; it was also widened towards each abutment over a length of 168 m, i.e. in the area where longitudinal tensile strains would be most likely to occur'.

The question on the resistance to hydraulic fracturing initially created two contrasting answers:

– The core should be made stiffer than the shells. In this case the shells would rest on the core, establishing additional vertical stresses. That means in practice placing the core at water contents below the optimum,

– the core should be made weak. In this case it is unlikely that part of the weight is transferred to the shells favouring the development of arching. That means in practice placing the core at water contents above the optimum.

Both the tendencies have a limitation: handling the material is more and more difficult – dryer than optimum as well as wetter than optimum – the greater is the difference between the optimum and the fill water content. The dry material tends to become brittle and more susceptible to cracking, which leads to anisotropy in terms of permeability. The wet material tends towards failure under the load of the construction machinery, which leads to the condition that the machinery cannot maneuver. In both cases the compactability of the material is reduced, with subsequently decreased shear strength and increased permeability after final consolidation.

Considering the stresses as demonstrated by Figure 7.19 it is recommended to dump and compact core materials with water contents greater than optimum. It means to make use of the favourable effects of reduced shear strength and increased pore-water pressure in unconsolidated conditions. Increased pore-water pressure corresponds to increased safety against hydraulic fracturing. During impounding the pore pressure dissipates. It should approach the lower hydrostatic pressure from a higher stress level. This will result in total stresses in the core which exceed the hydrostatic pressure.

At high pore-water pressure the core is likely to act as a dense liquid and to produce pressure to the adjacent shell which is greater than that imposed by the water of the reservoir. This pressure will deform and consolidate the shell according to its modulus of deformation. The tendency of the core to widen at the lower part is related to the development of spreading forces. As far as the deformation of the core is not connected to a change in volume, no tensile strains with the risk of cracking will occur. The post-construction deformation of the downstream shell due to the hydrostatic pressure from the reservoir is negligible. Such considerations have been confirmed by measurements on two rockfill dams with earth core (Penman & Charles 1973).

The conclusions to be drawn from that are obviously accepted by the profession, since a considerable number of large rockfill dams with symmetrical

Table 7.4. Examples of fill water contents of core materials. Embankment dams with vertical and slightly inclined cores.

Project	Height (m)	Core material Groupe symbol	PI	w_{opt} (%)	Fill water content (± % of w_{opt})	Core axis u/s slope	References
1 Srinagarind	140	Sand, clayey SC	10 to 25	12 to 17	0 to +2.5	Vertical 1V:0.2H	Champa & Mahatharadol (1982)
2 Kinda	75	Silt, sandy, clayey CL	15 to 25	12 to 16	0 to +2	Vertical 1V:0.33H	Kutzner (1985), Kutzner et al. (1988)
3 Dartmouth	180	Sand, silty Sand, clayey SM, SC	8 to 18	13 to 16	–0.5 to +1	Vertical 1V:0.4H 1V:0.5H	Murley & Cummins (1982)
4 LG-4	125	Moraine SM	Non-plastic	7 to 8	–1.0 to +2	Vertical 1V:0.25H	McConnell et al. (1982)
5 Aabach	48	HW siltstone, loam	Low plastic	11	+2 to +4	Inclined 1V:0.5H	Idel (1981), Idel & Stöhr (1982)
6 Pueblo Viecho	130	Clay, silty CL	18 to 20	23.5	–2 to +2	Inclined 1V:0.4H	Balissat & Wilhelm (1986)
7 Colbun	116	Silt, clayey, sandy, gravelly GC	5 to 20		0 to +4[1]	Inclined 1V:0.3H	de Pablo & Cruz (1985)
8 Scammondon	70	Clay CL, CH	15 to 30		+7 to +20	Inclined 1V:0.7H	Penman (1982), Penman & Charles (1973)
9 Monasavu	85	Clay (Halloysite) CH	43 to 62	40	+20	Inclined 1V:0.5H	Knight et al. (1982, 1985)

[1]Modified Proctor density

or asymmetrical cores have been constructed in the last decades. Good performance was confirmed by comprehensive measurements on them (e.g. ICOLD 1985a). Data of typical dams with a vertical core are listed in Table 7.4 (upper part). The respective core materials include plastic and non-plastic soils. As can be seen, the majority of the soils were compacted with water contents slightly greater than optimum. Unusual examples are the wet cores of nos 8 and 9, which are dealt with at the end of this section.

The core should be made of at least three different zones (Fig. 7.21). Rajcevic (1970) recommends fill water contents up to 10% above the plastic limit at the interface of dam and foundation. The plastic material is flexible enough to adjust to all irregularities of the surface and to give a good bond there. This applies also to interfaces of the dam and incorporated structures. High pore pressure there is the more important the steeper the surface of the structure. A moderate water content is recommended at the outer ends of the dam crest. This is to overcome the risk of tensile cracks there. The mass of the core should be compacted at water contents approximately equal to the plastic limit. This corresponds to the limits of the fill water content usually specified between optimum and optimum plus 2%. The suggestions according to Figure 7.21 – equally or slightly modified – are generally accepted and applied.

We are now going to look at the system known as 'embankment dam with inclined core'. The inclination of the upstream face of the core is limited for stability reasons of the outer slopes. Simple slip circle computations show that the reduction in stability of the outer slope is within 10% if the core slope is not flatter than 1V:0.7H. Flatter core sloping requires flatter dam sloping, irrespective of the high shear strength of the shell material. This is because of the position of the critical slip circle which cuts the core of low shear strength over a long section.

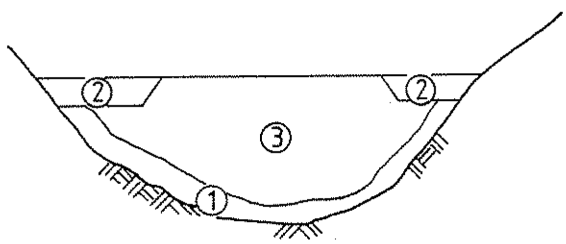


Figure 7.21. Recommended distribution and plasticity of core material (adapted from Rajcevic 1970).

1 Interface of dam and foundation: moisture content up to 10% above plastic limit
2 Dam shoulders: moisture content up to 6% above plastic limit
3 Center: moisture content approximately at plastic limit (close to optimum)

The Nantahala rockfill dam is an instructive example (Fig. 7.22). The core slopes at 1V:1.4H which corresponds to the steepest reasonable slope of the rockfill shell. Stability is achieved only by sloping the shell 1V:2.5H and dumping a stabilizing berm, which makes the slope 1V:3H in general. According to Thomas (1976), at the time of construction in 1942 the flat slope of the sealing was regarded as advantageous for constructional reasons: the design enabled construction of the large downstream portion and placing the sealing after the material was consolidated. Great settlements had been expected as a result of dumping but not compacting the rockfill. Actually, at the time when the sealing was placed, the settlement of the shell amounted to only 40% of the final settlement, which was in the range of 80 cm after 8 years.

In our times, with techniques to establish high moduli by heavy compaction this aspect does not need to be noted. However, it should be considered that adverse weather conditions may cause a delay in dumping and compacting the sealing element. In this case it is still advantageous to continue the construction of large portions of the downstream shell, which is facilitated by an inclined core.

For geometrical reasons the inclined core is considered to create favourable deformation conditions between the core and the shells, with reduced risk of arching and subsequent cracking. Maksimovic (1973) has investigated the relation of arching and inclination by applying simplified computations to models of dams with vertical and inclined cores, the upstream

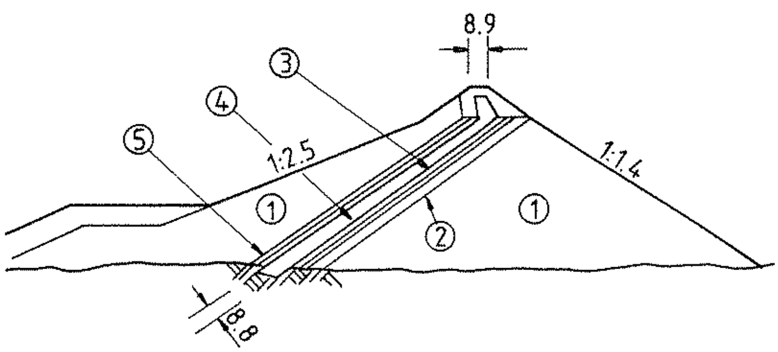


Figure 7.22. Nantahala rockfill dam with internal earth sealing (dimensions in m, adapted from Crowden 1960).

1 Quarry-run rockfill material
2 Transition zone
3 Double filter below internal sealing
4 Impervious soil
5 Transition above sand layer

faces being 1V:0.25H, 1V:0.5H and 1V:0.75H. The simplifications are: elastic deformation of the materials, no transition zone between the core and the shell, dam construction in one step instead of several layers.

The main results are shown in Figure 7.23. As expected, the degree of arching d_a increases with increasing stiffness ratio $K = E_A/E_B$ of the shell and the core (Fig. 7.23a). The position of the maxima is between 0.5 and 0.7 times the dam height. For a selected stiffness ratio of $K = 4$ the degree of arching decreases with an increasing Poisson ratio because the horizontal stresses increase in the same sense (Fig. 7.23b). The figure also demonstrates the increase of d_a when the core face is steepened.

The values of d_a according to Figure 7.23 discover the tendency towards arching due to core inclination and material properties. Because of the simplifications made, the values cannot be taken directly for practical use. For plastic deformation and dam construction in several layers the degree of arching will be reduced. It was found with the Kinda dam that a deficit in vertical stresses of 20% did not lead to harmful arching (Section 6.1.1). Maksimovic points to the advantage of 'flat' core inclination over the vertical and symmetrical core. He recommends 1V:0.5H to 1V:0.6H as the best selection.

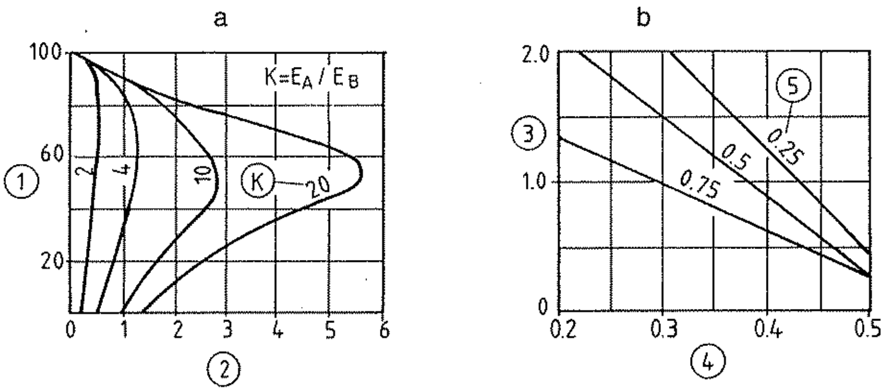


Figure 7.23. Effect of material properties and core inclination on the degree of arching (simplified computation, adapted from Maksimovic 1973).

a Degree of arching as a function of the stiffness ratio K
b Degree of arching as a function of the core inclination and the Poisson ratio, $K = 4$
E_A Young's modulus of the shell material
E_B Young's modulus of the core material
1 Dam height (%)
2 Degree of arching d_a
3 Maximum values of d_a
4 Poisson's ratio of the dam materials
5 Parameter m of the upstream core inclination 1V:mH

A reduced width may be attributed to the inclined core, because of its favourable deformation conditions. Reduced width results in increased hydraulic gradients which are typically $i = 3$ to 4, in contrast to $i = 2$ with vertical cores. The 'narrow' core may cause seepage from the core to the foundation. This can be ruled out by steepening the downstream face of the lowest portion of the core to make it wider there. It is recommended to exclude such seepage by shotcreting the rock surface. The increase in gradient may result in reduced volume of the core. The construction progress of the core, if reduced in volume, will be less sensitive to inclement weather conditions.

Examples of dams with inclined core are given in Table 7.4 (center part). Again, the core material is placed mainly with water contents slightly greater than optimum. Measurements confirm the good performance of such dams, including the wet core dams (nos 8 and 9). For reference see also ICOLD 1985a. There are other examples where lack of material is reported to be the reason for the selection of an inclined core, e.g. the Svartevann dam, Sweden (Kjærnsli et al. 1982) and the Hans Strijdom dam, South Africa (Hollingworth & Druyts 1982).

A particular type of core is a wet core where the material's fill water content is considerably above the optimum, e.g. optimum +20% with the Monasavu dam (Table 7.4). The high water content must be accepted due to the climatic conditions of the region and related water contents of available materials. According to Knight et al. (1985) the following applies to the Monasavu dam: maximum settlements of the core – at approximately half the dam height – were 1.7 m at end of construction and 1.8 m two years later after about one year of full supply level, i.e. around 2% of the dam height. Such settlement is not excessive. The pore-water pressure in the lower portion of the core was about 50% of the dead weight after the full supply level was reached. This pressure is higher than is usual with other than wet cores. Settlements and pore-water pressures and related tendencies are tolerable.

7.3.2.2 *Trench diaphragm of soil-cement*

After 1970 a method of core improvement was developed in Germany by excavating a trench in the center of a dam core and filling it with soil-cement. This material is also called plastic concrete or earth-concrete. The material is here considered as a 'natural' material since the main component is soil. The trench method was applied where the permeability and the erosion stability of a given soil were not considered satisfactory. It is based on the world-wide known method of establishing a trench diaphragm of conventional or plastic concrete in the foundation of dams as a positive cut off wall.

Soil-cement or plastic concrete consists of soil, cement, bentonite or powdered clay and water. It must guarantee erosion stability by the content of cement, and flexibility by the content of bentonite or powdered clay. It must be able to follow the deformations of adjacent zones without cracking. The composition of soil-cement is discussed in Section 9.6.

Three different procedures are in use (the letters N and T refer to the German language):

1. The wet method N1. The trench is excavated from the crest of the dam. During excavation it is filled with the final sealing material. The material serves to stabilize the trench being excavated and to act as final sealing. An example is the Brombach dam, Germany (Beier & List 1982, Strobl 1989). In English literature the final structure is also called the 'grout diaphragm'.

2. The wet method N2. Again the trench is excavated from the crest of the dam. During excavation the trench is supported by a bentonite suspension. After excavation to the final depth, the suspension is replaced by the final sealing material. An example is the test diaphragm at Frauenau (Lorenz 1976).

3. The dry method T. The trench is excavated in sections down to depths of 2 to 4 m from the temporary surface of the dam being constructed, and then filled with the final sealing material. The trench does not need a stabilizing fluid, which is why the procedure is called the dry method. The excavation depth is compatible with the stability of the open trench. Examples are the Förmitz dam (Lorenz & List 1976, List 1980) and the Frauenau dam (List 1987).

With the wet methods N1 and N2 the trench is excavated at the time when the major parts of dam consolidation and settlements are finished. The stresses on the diaphragm during and after the hardening of the plastic concrete are mainly caused by the deformations of the embankment during reservoir impounding and operation. The enlarged flexibility of the plastic concrete, in comparison to conventional concrete, takes account of these circumstances. The flexibility must fit with the strength required to achieve erosion stability.

The method N1 offers the advantage of suspending the arrangement of joints between neighbouring panels. Because of this and because of the need for only one material the method N1 is cost-effective. The processing of the material needs high standards, since the content of solids is low and, therefore, irregularities will noticeably affect the quality. The upper limit of the material's unit weight is in the range of 12 to 12.5 kN/m^3. The method was applied to produce a diaphragm for temporary purposes down to 45 m. Fell et al. (1992) infer that such a diaphragm may be used to at least 50 m depth. The depth will depend on the climate, which affects the hardening time of the filling material.

The wet method N2 equals the slurry trench system known from urban foundation and dam foundation engineering. The trench is divided into panels, where primary panels are excavated first (odd numbers), then secondary panels (even numbers). The panels are 2 to 4 m in length, being stabilized by bentonite slurry with a unit weight of 10.2 to 10.5 kN/m³. After the final depth of the panel is reached the slurry is replaced by plastic concrete poured from bottom to top, the bottom end of the tremie pipe placed well below the concrete surface. Primary panels are filled first, then secondary panels.

The hardening of the plastic concrete and the construction progress must be compatible with each other. This means that secondary panels are concreted at the time when the concrete of the primary panels is strong enough to exclude collapse at the end sections, but weak enough to be scraped away by the excavator to create a tight bond between the panels. Alternatively, the end sections may be supported by a steel pipe which is withdrawn after concreting, as is the common technique to produce a trench diaphragm of conventional concrete. Many experts consider the remaining half round key as a guide to excavate the adjacent panel with a reduced risk of misalignment. It may be a matter of work performance which type of joint is more prone to leakage.

The plastic concrete used with method N2 is stronger than that used with N1. It must be flexible enough to follow the deformations of the adjacent zones without cracking. The weight is in the range of 20 kN/m³. The weight and related viscosity help to avoid the confinement of lumps of bentonite suspension in the diaphragm. Common depths are up to about 60 m, an existing exceptional depth is 131 m (conventional concrete, foundation of the Manicouagan dam, Canada, Fig. 7.64).

The diaphragms produced according to the dry method T must be able to follow the deformations of the dam during its consolidation and during reservoir operation. These deformations are greater than those which affect diaphragms produced according to the wet methods. The diaphragm is raised almost simultaneously with the embankment. The vertical joints to the neighbouring panels and the horizontal joint to the previously concreted section can be cleaned by hand, the workers being lowered into the trench with a protective cage.

Experiences in Germany have shown that plastic concrete at a weight of 20 kN/m³ was able to adjust to the deformations of the dam (Förmitz and Frauenau, Germany, Figs 7.24 and 7.25). The cores of the two dams consist of low quality core material and a 60 cm wide diaphragm of plastic concrete in the center. The initial permeabilities of the two materials are around 10^{-8} and 10^{-9} m/s, respectively. Four months after the end of construction the total settlements of the Förmitz dam reached 20 to 25 cm, i.e. about 1% of the dam height (Lorenz 1976). Displacements of the crest of the Frauenau dam, 75 m in height, were measured as 18 to 26 cm vertical and 28 to 36 cm horizontal, 3½ years after reaching the full supply level (List 1987). Such displacements refer to the conditions of reservoir impounding and operation

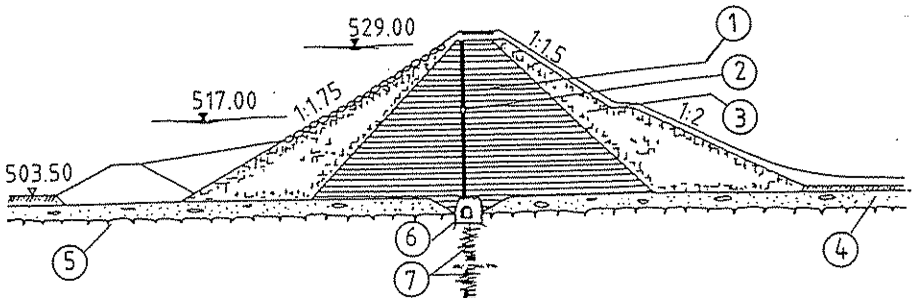


Figure 7.24. Typical section of Förmitz earth and rockfill dam with earth core and trench diaphragm (adapted from List 1980).

1 Core, silty sand
2 Diaphragm, plastic concrete with geotextile
3 Shell, clayey gravel and cobbles
4 River deposit
5 Rock foundation
6 Inspection gallery
7 Grout curtain

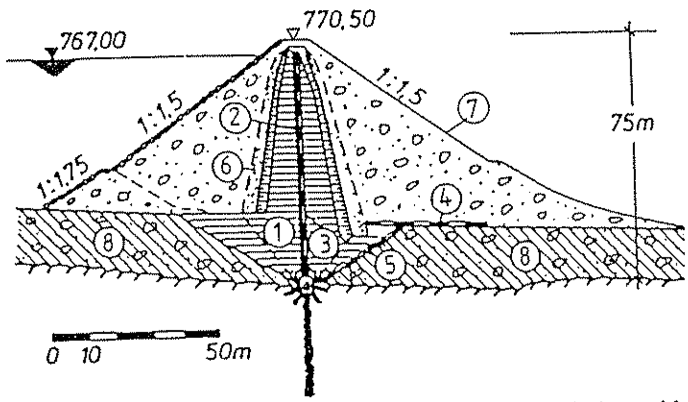


Figure 7.25. Right abutment section of Frauenau rockfill dam with earth core and trench diaphragm (adapted from List 1987).

1 Core, gravelly clay
2 Diaphragm, plastic concrete
3 Geotextile in the core
4 Geotextile in the downstream shell
5 Geotextile in the core trench
6 Filter and transition zones
7 Shell, rockfill material
8 Weathered gneiss
(Elevations m a.s.l.)

with about 10 m water level fluctuation. The vertical settlement is estimated to be about 80 cm at the location of the maximum settlement within the core, again being around 1% of the dam height. Such values will be well within the deformation limits of plastic concrete (see Table 9.6).

The dams at Förmitz and Frauenau are equipped with geotextiles located at the downstream face of the membranes of plastic concrete. The geotextile is a synthetic felt (fleece) which has two functions: drainage of seepage and erosion protection with a self-sealing effect. With the Förmitz dam the fleece acts mainly as drainage, with the Frauenau dam mainly as erosion protection. All seeping water at Förmitz is captured by the fleece and conducted to the inspection gallery. It was measured as 0.4 l/s with reference to a sealing area of 5700 m^2. In contrast, the seeping water at Frauenau is not conducted to the inspection gallery. It penetrates the fleece and the downstream portion of the core and is then conducted to a collector at the downstream toe of the core enabling separate registration of seepage through and seepage below the dam. The two have been measured as 0.4 l/s through the diaphragm of 30,000 m^2 and 2 l/s through the foundation, both after 3½ years of reservoir operation at full supply level (List 1987).

7.3.2.3 *Slot diaphragm of soil-cement*

A similar diaphragm is used to form an impervious barrier in soil by producing a slot and filling it with plastic concrete or similar material. Originally the method was applied to seal the foundation of small dams. Since about the second half of the 1970s it is also used within dams. A heavy H-girder is driven into the soil. When being withdrawn a slot is left open which is then filled with plastic concrete or similar using a grout pipe attached to the H-girder. The material is pumped into the slot under pressure to penetrate the soil in the vicinity of the slot. Usually, a single girder 500 to 1000 mm in width is brought down by vibration in a way to overlap neighbouring slots and to form a continuous diaphragm (Figs 7.26 and 7.27).

Due to the high daily production rate the slot diaphragm is a low-cost system. The depth is limited to a maximum of about 20 to 25 m, according to the density of the soil and to obstacles, such as boulders and cobbles. Daily production rates are in the range of 200 to 300 m^2 which exceeds the production rate of trench diaphragms. In Germany the procedure was applied to a great extent to seal existing dikes and their foundation at the river Lech (Fig. 7.28). In dam construction boulders and cobbles can be omitted. In the foundation, such obstacles and the natural density of the material must be identified to predict that depth to which the diaphragm can be fairly established. That means, comprehensive soil investigations are required, including penetration tests. Where unfavourable conditions prevail, drilling to a large diameter ahead of driving the H-girder may facilitate the procedure.

Due to the limited depth the diaphragm is subjected to minor deformations only. Therefore, the thickness of 8 to 20 cm is sufficient, provided the fill material is impervious and strong enough to resist erosion. The thickness results from the web plate of the girder which is 6 to 8 cm thick and a surplus of grouted soil. The excavated slot diaphragm in Figure 7.27 exposes such a surplus on both sides. Its thickness depends on the soil's gradation

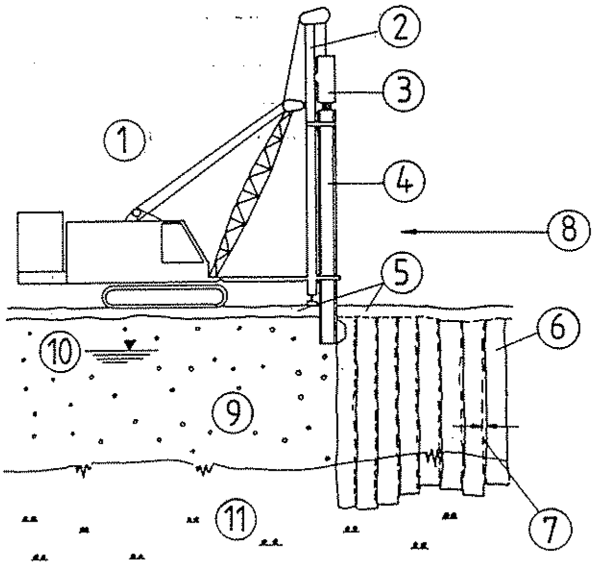


Figure 7.26. Slot diaphragm procedure (adapted from BAWAG 1986).

1 Carrier of driving and vibrating equipment
2 Guiding beam
3 Vibrator
4 H-girder
5 Trench for plastic concrete
6 Slot diaphragm
7 Overlapping
8 Working progress
9 Soil to be sealed (e.g. gravel)
10 Ground water level
11 Impervious soil (e.g. tertiary clay)

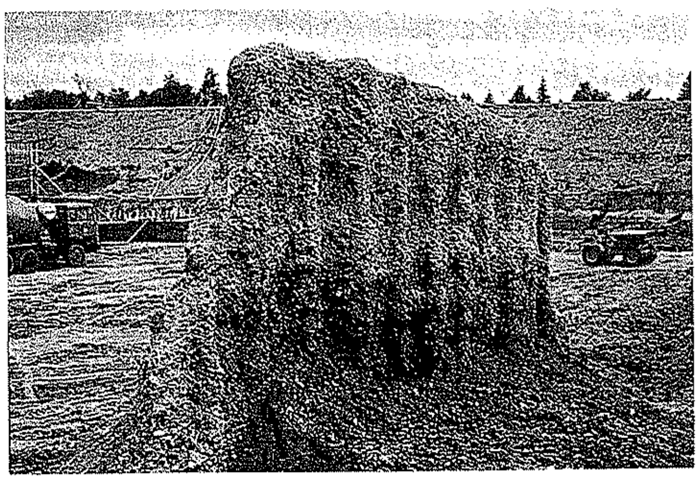


Figure 7.27. Excavated slot diaphragm (courtesy of BAWAG).

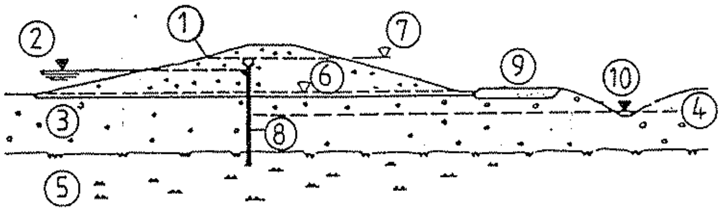


Figure 7.28. Slot diaphragm as an impervious barrier in river dike and foundation (adapted from BAWAG 1986).

1 River dike
2 Flood water level
3 Permeable soil (e.g. gravel)
4 Ground water level
5 Impervious soil (e.g. tertiary clay)
6 Ground surface
7 Working level
8 Slot diaphragm
9 Road
10 Drainage trench

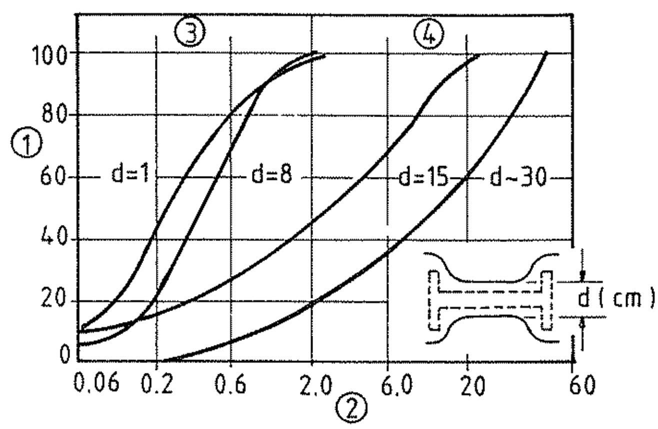


Figure 7.29. Effect of soil gradation on the thickness of the slot diaphragm (adapted from Strobl 1989).

1 Percent finer by weight
2 Grain size (mm)
3 Sand
4 Gravel
d Thickness of the slot diaphragm (cm)

and density and on the driving and withdrawing velocity of the girder. Figure 7.29 gives an approach to the diaphragm thickness in relation to the grain size distribution of the soil.

The permeability of the hardened fill material of the Lech projects was 10^{-8} to 10^{-7} m/s at the age of 28 days (laboratory). Samples of the diaphragm (field) showed values $\geq 10^{-9}$ m/s, due to the effect of adhering cemented soil. The required erosion stability is usually controlled by permeability tests at very high gradients. After BAWAG (1986) it is related to the uniaxial compressive strength determined at a controlled feeder speed of 0.2% of the initial sample height per minute (i.e. according to DIN 18 136). Erosion stability was found to be satisfactory with samples of a strength ≥ 200 kPa. The relation of erosion stability and strength must be verified for each individual project if the strength is taken as the controlling parameter.

A new development involves producing a similar slot diaphragm by double or triple auger drill rigs (Fig. 7.68). With the rotation of the auger, a cement suspension is pressed through the drill rods and mixed with the loosened soil. It is a mix-in-place method like jet grouting but without the application of very high pressures.

7.3.2.4 *Face sealing of soil*

Mineral face sealings are in use with dikes where there is little wave action and little water fluctuation. According to USBR (1973) soil types GM, GC and GW-GC are appropriate if the swelling and shrinking rates are small. Such sealing is also used with higher dams of limited function, for instance with cofferdams as in Figures 7.17 and 7.18. Usually the face needs a protective layer.

The sealing consists of well compacted cohesive soil. A transition layer is required between the sealing and a rockfill embankment to prevent the sealing from being washed into the voids of the rockfill. The stability conditions of the sealing are the same as those for homogeneous dams. The slope must be compatible with the shear strength of the sealing material. Frequently the slope is made 1V:2.25H. Under the conditions shown in Figure 7.17 the gradient at the toe of the sealing is $i = 5$ to 6.

Prior to impounding the sealing should be removed from the cofferdam slope to achieve steady saturation of the dam body from bottom to top. It may be sufficient to excavate trenches along the lines of the steepest gradient.

7.3.3 *Filters and transition zones*

Both filters and transition zones are discussed together since their functions cannot clearly be separated from each other. In literature the use of the two terms is not fully defined. A third term, namely drainage zone, is used as well. Here, the terms shall be used as follows, according to the main functions:

– Filters act mainly to protect fines from being washed into the voids of an adjacent coarser material. The function is effective only if water flows across the zones. In accordance to other references such filter zones are nominated as 'protective filters', 'critical filters' and 'perfect filters'. These terms apply to fine filters. In addition, there are coarse filters to protect the fine filters.

– Transition zones act mainly to establish acceptable stress conditions and tolerable differential deformations between adjacent materials having noticeably different properties, such as core and rockfill materials. The function is independent of the flow of water.

– Drainage zones act mainly to conduct water safely through the dam to the downstream area. This function is attributed also to filter and transition zones. The term 'drainage zone' will be restricted here to cases where the zone could be replaced by a pipe or an open channel to conduct water.

As a consequence of the overlapping functions filter and transition zones may sometimes substitute for each other. There is no need always to have them both, with a clear separation of filters and transition zones.

7.3.3.1 *Filters*

The considerations on filters are generally directed to the requirement that the pore system of the filter geometrically does not allow fines of the adjacent base material to pass the filter, by this way preventing the base material from being eroded. The pore system is assumed to consist of differently sized pores, connected by pore channels of varying size and shape. Larger and smaller pores and pore channels are homogeneously distributed in the soil mass. Only at the interface of base material and filter the pore distribution is not homogeneous, thus to some extent permitting migration of fines into the filter (Fig. 7.30).

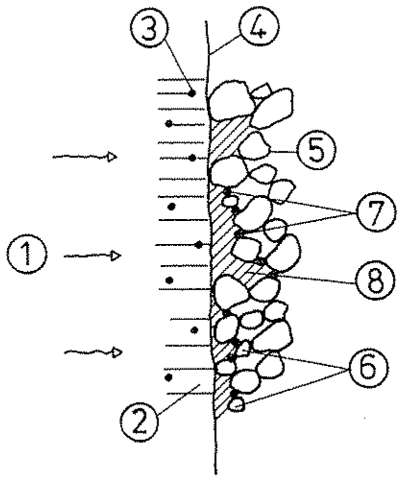


Figure 7.30. Illustration of the interface of base soil and filter.

1 Seepage
2 Base soil
3 Particle d_{85} of base soil
4 Interface filter/base soil as designed
5 Filter
6 Particles D_{15} of filter
7 Particles d_{85} captured by the filter
8 Base soil that migrated into filter

According to experience the particles of the base material $\geq d_{85}$ are captured by the filter, as shown, making further penetration of fines impossible. The 'characteristic grain size' d_{85} is that grain size of the base material at which 85% of the particles are smaller. This phenomenon of limited migration affects the required minimum filter thickness to make sure that fine particles cannot pass across.

It is very complex to verify and to illustrate the distribution of the pores and channels of different size within the soil mass. It is, however, justified to assume that a defined grain size distribution exists as an analogue to the pore and channel distribution. The grain size distribution can easily be verified and illustrated. Therefore, the common considerations on filters are based on the grain size distribution substituting the pore and channel distribution as a criterion. Alternatively, but not generally agreed, the permeability of a filter is used as a criterion (see below at *Perfect filters*).

Protective filters
Terzaghi & Peck (1948) have empirically developed the basic concept of the filter gradation required to protect fine soils from erosion. Their findings are essentially applied as filter rules up to now, for fine filters as well as for coarse filters:

$$D_{15} \geq 4\ d_{15} \tag{7.4a}$$

$$D_{15} \leq 4\ d_{85} \tag{7.4b}$$

where: D_{15} = Grain size of filter material at which 15% of the particles are smaller
$d_{15(85)}$ = Grain size of base material at which 15 (85)% of the particles are smaller.

Filters composed according to Equations (7.4a) and (7.4b) are frequently called 'Terzaghi-filters'. Modifications are discussed below.

The following should not be forgotten: the above rules are essentially based on tests on poorly graded sands. The coefficient of uniformity d_{60}/d_{10} of the tested soils is in the range of 2 to 4. In addition, the soils are non-cohesive.

Other tests resulted in modified filter criteria (USBR 1973):

$$D_{15} = 5\ d_{15} \text{ to } 40\ d_{15} \tag{7.5a}$$

provided that the filter does not contain more than 5% of material finer than 0.074 mm, and

$$D_{15} \leq 5\ d_{85} \tag{7.5b}$$

The grain size curve of the filter should be roughly parallel to that of the base material. These criteria are satisfactory for filters of natural sand and gravel and of crushed rock, and for uniform and well graded materials. The maximum grain size should not exceed 75 mm in order to minimize segregation and bridging of large particles during filter placement.

The filter rules according to Equations (7.4) and (7.5) have in numerous cases proven to be satisfactory. If applied as a filter to protect clay the rules are deemed conservative. We will see that Equations (7.4a) and (7.5a) cannot always be satisfied, and do not always need to be satisfied.

There are mainly two reasons to restrict the content of fines < 0.074 mm to 5%:

– The filter must be non-cohesive. It must be able to follow all deformations without cracking. Such a filter maintains its erosion protective potential after deformation. The permeability might be slightly decreased after settlements and shear deformations. In contrast, this is not the case with cohesive material that develops cracks when being deformed by static and dynamic loads.

– Fine particles of the filter might be displaced or washed out, which would unfavourably affect the retaining potential of the pore system and the permeability of the filter.

A non-cohesive filter of this kind will have a minimum D_{15} of about 0.1 mm (no 8 in Fig. 7.31). Applying Equation (7.5a) with $D_{15} = 40\ d_{15}$ yields the result that cohesive materials up to about 15% clay content < 0.002 mm will be protected by the non-cohesive, finest possible filter no 8. In practice, soils of more than 15% clay content are used for dam construction, as is demonstrated in Figure 4.31. According to experience, such base soils are protected by the filter no 8. This is due to the effect of cohesion which allows flocs to develop during erosion. The size of the flocs is in the order of 0.02 mm = $D_{15}/5$. Such flocs block the entrance to the pore system of the filter, as is illustrated in Figure 7.30, making the protective function of the filter effective.

Critical filters

The critical interface where erosion might start is the downstream face of a sealing element. Therefore, the neighbouring fine filter is called the 'critical filter'. Comprehensive tests made by Sherard and his staff have confirmed that the Terzaghi-Filter is conservative, and that coarser materials are equally able to protect cohesive soils from erosion. Sherard et al. (1984b) have investigated 36 types of silt and clay, the clay content < 0.002 mm being more than 20% for 23 of these soils, and 7 of them being dispersive. Selected examples of these soils are shown in Figure 7.31, nos 12 to 15.

The tests resulted in the coarsest filter bands for critical filters shown in Figure 7.32. The limiting parameter D_{15} is 0.2 to 0.8 mm (sand) and 0.3 to 1.0 mm (sand-gravel), instead of D_{15} = 0.1 mm of the Terzaghi-filter. Sher-

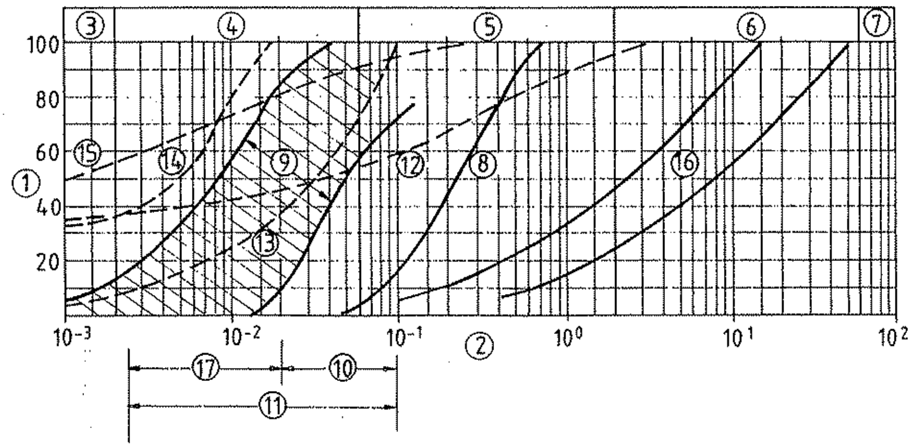


Figure 7.31. Illustration of filter criteria.

1 Percent finer by weight
2 Grain size (mm)
3 Clay
4 Silt
5 Sand
6 Gravel
7 Cobbles
8 Finest non-cohesive filter
9 Range of base soils which are protected by filter no 8, applying Equations (7.4) and (7.5)
10 $D_{15} = 5 \times d_{85}$
11 $D_{15} = 40 \times d_{15}$
12 to 15 Base soils proven to be protected by filter no 8
12, 13 Soils of group 2 (Sherard et al. 1984b, 1989)
14 Soil group 1 (Sherard et al. 1984b, 1989)
15 Dispersive soil of group 1
16 Gradation range of 'critical' filter after Sherard et al. 1984b (Fig. 7.32)
17 Grain size δ of clay flocs

ard et al. recommend the criteria according to Figure 7.32 for non-dispersive and dispersive silts and clays of the group symbols CH and CL (Table 4.7) and min. $d_{85} \approx 0.03$ mm, the min. $D_{15} = 0.3$ mm being attributed to the min. $d_{85} \approx 0.03$ mm of fine-grained clays and fine-grained silts of low cohesion. The criteria should not be applied for extremely graded gravelly clays which are internally unstable. The permeabilities of the two filter bands are equivalent, namely $\approx 5 \times 10^{-4}$ m/s for the two min. D_{15} and $\approx 7 \times 10^{-3}$ m/s for the two max. D_{15}.

In a further test programme Sherard & Dunnigan (1989) have investigated the phenomenon of erosion of 4 groups of soil. These are:

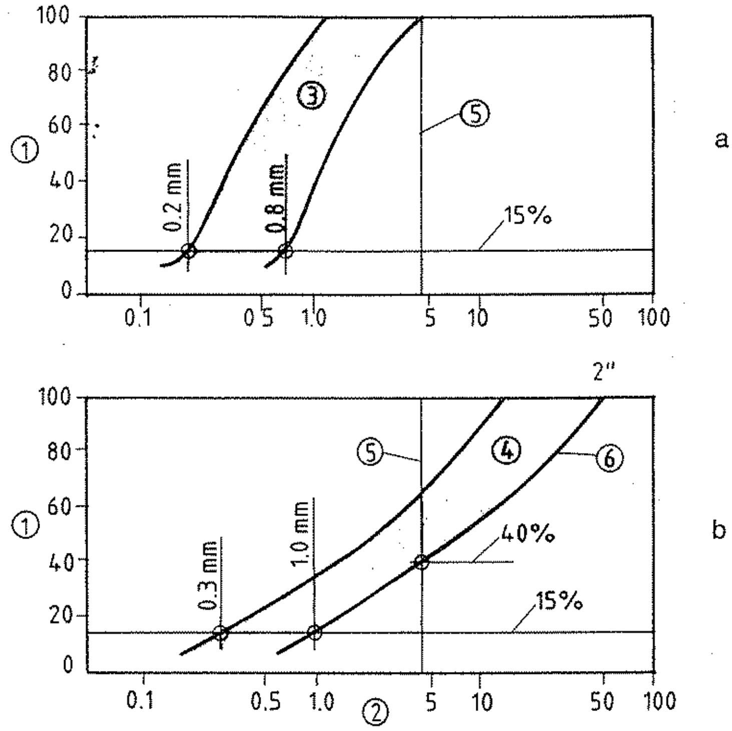


Figure 7.32. Coarsest (largest D_{15}) filter bands considered desirable for 'critical' filters for very fine clays (CL and CH) (Sherard et al. 1984b).

1 Percent finer by weight
2 Grain size (mm)
3 Sand filter
4 Sandy gravel filter
5 Grain size d = 4.76 mm (US-sieve no 4)
6 Suggested coarse boundary to limit segregation

– Group 1: Fine silts and clays with more than 85% below 0.074 mm.

– Group 2: Silty and clayey sands and sandy silts and clays with 40 to 85% below 0.074 mm.

– Group 3: Silty and clayey sands and gravelly sands with 15% or less below 0.074 mm.

– Group 4: Soils intermediate between groups 2 and 3.

They developed a 'No erosion filter test' (NEF-test) where the base soil has a preformed hole to allow erosion, as shown in Figure 7.33. Details of the test are listed in Table 7.5. The authors note that the test results are reliable and reproducible, so the test can be used quickly to evaluate the effectiveness of critical filters. For each of the soils of group 1 to 4 there exists a well defined limiting grain size D_{15} of the critical filter which can be taken as a property of the base soil in the same sense as e.g. the Atterberg limits and the shear strength parameters. The respective data D_{15} are given in Table

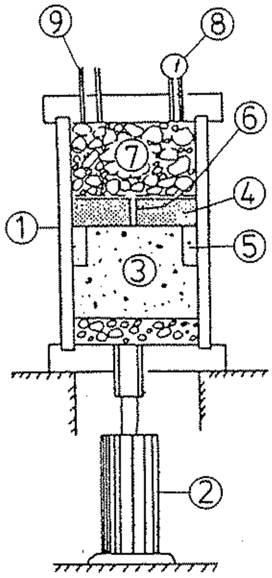


Figure 7.33. No erosion filter test details, dimensions see Table 7.5 (adapted from Sherard & Dunnigan 1989 and Fell et al. 1992).

1 Plastic cylinder
2 Graduated cylinder for measuring rate of flow
3 Filter
4 Compacted impervious base specimen
5 Side material, sand finer than filter
6 Preformed hole in base specimen
7 Gravels fill space
8 Pressure gauge
9 Water source (high pressure)

Table 7.5. NEF-test according to Figure 7.33, details (produced after Sherard & Dunnigan 1989 and Fell et al. 1992).

Type of soil	d_{85} (mm)	Sample diameter (mm)	Sample thickness (mm)	Hole diameter (mm)
Fine soils	≤ 3.0	100	25	1.0
Coarse soils	> 3.0	280	100	5 to 10

7.6. The parameter D_{15} of the filter, found by the NEF-test, classifies the filter as effective or non-effective to protect a given base soil. According to Sherard & Dunnigan, D_{15} can be determined within an accuracy of the order of 0.1 mm. It is not influenced by the base soil's plasticity index, the relative erosion resistance and the potential for dispersive erosion.

A comparison of the filter criteria discussed before, according to Table 7.6, to Figure 7.32 and to Equations (7.4b) and (7.5b) discovers very similar conditions:

– The filters of Figure 7.32 are deemed appropriate for base soils with min. $d_{85} \approx 0.03$ to 0.1 mm. The recommended data $D_{15} = 0.2$ to 1.0 mm make 7 to 10 times d_{85} as is suggested in Table 7.6 for soils of groups 1 and 3 and partially group 4.

– Understandably, coarser base soils allow greater D_{15}. For group 2 it is max. $D_{15} = 1.5$ mm and min. $D_{15} = 0.7$ mm (Table 7.6) which fit with Equations (7.4b) and (7.5b) down to max. $d_{85} \approx 0.3$ mm and min. $d_{85} \approx 0.15$ mm, respectively.

Table 7.6. Filter boundaries (D_{15}) as determined by fine content for four soil groups (adapted from Sherard & Dunnigan 1989).

Soil group	Fine content < 0.074 mm[1] (% by weight)	Grain size max. D_{15} of filter (mm)
1	85 to 100	7 d_{85} to 12 d_{85} (Mean value 9 d_{85})
2	40 to 85	0.7 to 1.5
3	0 to 15	7 d_{85} to 8 d_{85} (rounded grains) 9 d_{85} to 10 d_{85} (crushed grains)
4	15 to 40	Intermediate between groups 2 and 3, depending on fine content

[1]US-sieve no 200

– Still coarser base soils with floating gravel are protected by the filters of Figure 7.32 within the limits of Equations (7.4b) and (7.5b).

– Extremely fine base soils with $d_{85} < 0.03$ mm are protected by the Terzaghi-Filter with $D_{15} \approx 0.1$ mm. This filter meets the criterion of Table 7.6 for soil group 1 down to the extreme value of $d_{85} \approx 0.008$ mm.

The following can be concluded: for the design of successful filters there exist reasonable criteria for almost all types of base soils. The reliability is proven in practice. A remaining uncertainty exists for cohesive soils with $d_{85} > 0.5$ mm which may be internally unstable. This has led to investigations on a perfect filter, initiated by a given case of erosion.

Perfect filters

In 1967 defects in the core of the Balderhead dam had been observed which resulted from hydraulic fracturing and subsequent internal erosion. The filter downstream of the core was not able to retain the migration of material. Investigations ended in the definition of a perfect filter and a design criterion for such a filter on the base of the filter permeability (Vaughan & Soares 1982).

The grain size distributions of the core material and of the existing filter are shown in Figure 7.34. The core material is a broadly graded clayey silt-sand with gravel (till with a clay matrix), belonging to groups 2 and 4 of Table 7.6. The clay is reported to be of high erosion resistance. The filter consists of crushed hard limestone.

The damage was initiated by local hydraulic fracturing of the core, followed by erosion which started at the uneven crack walls. Erosion was accompanied by segregation and deposition of the coarser particles within the crack. It is assumed that mainly the eroded fine particles after segregation reached the filter which could not stop the migration. It can be traced back, from Figure 7.34, that the criteria of Table 7.6 are not fulfilled for the coarser part of the filter gradation and under the conditions of internal ero-

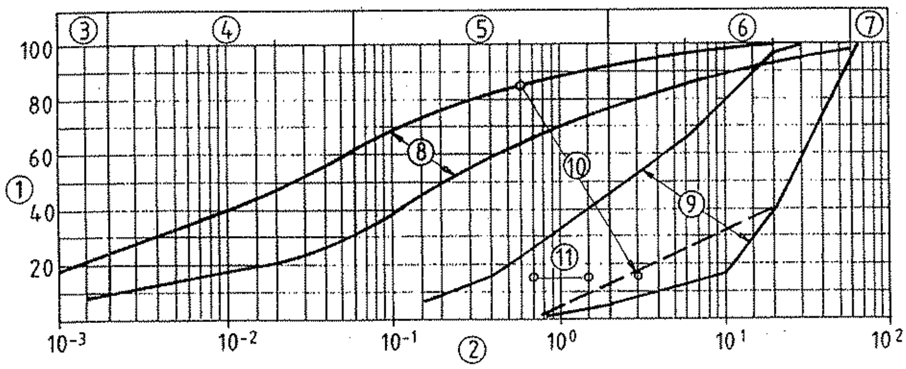


Figure 7.34. Balderhead dam. Core and filter gradings (adapted from Vaughan & Soares 1982).

1 Percent finer by weight
2 Grain size (mm)
3 Clay
4 Silt
5 Sand
6 Gravel
7 Cobbles
8 Core
9 Actually existing filter
10 $D_{15} = 5 \times d_{85}$
11 $D_{15} = 0.7$ to 1.5 mm

sion, where only fine particles reach the filter. However, this explanation could not be given at the time of the investigations (1982) since the findings were known only after 1989.

The Balderhead tests were focused on a perfect filter, able to retain minute particles of clay being washed into the filter after erosion and reaching the filter unaccompanied by coarser particles of the core material. The problem appeared to have no solution because D_{15} of the filter should be in the order of $40 \times 0.001 = 0.04$ mm. Such a filter would be cohesive, with the risk of cracking and being ineffective.

The key to a solution was the discovery that clay flocs are developed during erosion which the filter is able to retain, in contrast to individual clay particles. The development and the size of flocs depend on the chemistry of the seeping water and of the clay. As a rule, clay flocs will develop. According to tests the size of flocs of illitic clay is in the range of 0.006 to 0.012 mm (see Fig. 7.31). Other clay minerals may develop other flocs.

A relation was found between the permeability of a perfect filter and the size of clay flocs which separates effective and ineffective filters. It is

$$k = 6.7 \cdot 10^{-6} \cdot \delta^{1.52} \tag{7.6}$$

where: k = permeability (m/s)

δ = diameter of eroded flocs and particles (μm).

A number of filters was tested, specified according to Equation (7.6), and used for the construction of smaller dams.

The criterion for the perfect filter was not generally agreed by the profession. According to USCOLD (1993) the criterion is severe and the permeability is not a practicable parameter for field control. Irrespective of this, the case of Balderhead is discussed here in detail because it resulted in the definition of the floc-like character of eroded particles. Balderhead is an unusual case because the damage resulted from the interaction of hydraulic fracturing and internal erosion with segregation of fines and coarser particles. In addition, the filter installed was – at its coarse boundary – unlikely to protect the base material. A slight modification, as shown by the dotted line in Figure 7.34, would have met the criteria

max. $D_{15} = 5 \times d_{85}$, and
$D_{15} = 0.7$ to 1.5 mm for soils of group 2

in a practical sense. It was suggested repeatedly always to check the internal stability of well graded cohesive soils with respect to the filter design (see below at *Internal stability*).

Effect of uniformity

The above discussion on filters was concentrated on non-cohesive protective filters for cohesive base soils. The typical property of such filters is the low content of fines, with $D_5 \approx 0.06$ to 0.074 mm. Most filters are to be found in the range between the poorly graded Terzaghi-filter no 8 in Figure 7.31 and the lower boundary of the well graded Sherard-filter no 16. The effect of uniformity was not examined though this parameter affects the permeability of the filter and hence its retaining potential. The number of smaller particles around D_{10} or D_{15} increases with the coefficient of uniformity. Therefore, the permeability decreases and the ability to retain fines increases with the grading.

Different approaches can be found in the references to take account of the grading. USBR (1974) suggests that for poorly graded filters the gradation ratio $D_{50}/d_{50} = 5$ to 10 should be used, and for well graded filters the ratio $D_{50}/d_{50} = 12$ to 58 and $D_{15}/d_{15} = 12$ to 40 (letters D always applying for the filter and d for the base soil). It can be seen from Figure 7.31 (not shown) that the ratios $D_{50}/d_{50} = 10$ for the Terzaghi-Filter and $D_{50}/d_{50} = 58$ for the Sherard-filter do not cover all potential base materials. That means the ratio D_{50}/d_{50} in the sense of USBR should not be applied as a reasonable criterion for the design of effective filters. This is repeatedly stated in the references.

A more elaborated approach for the use of the ratio D_{50}/d_{50} can be seen from the criterion of Cistin & Ziems (1968), Figure 7.35, which is frequently applied in Eastern Europe. The ratio D_{50}/d_{50} varies between a minimum of about 8 for a well graded base material and a poorly graded filter ($U_D \approx 2$)

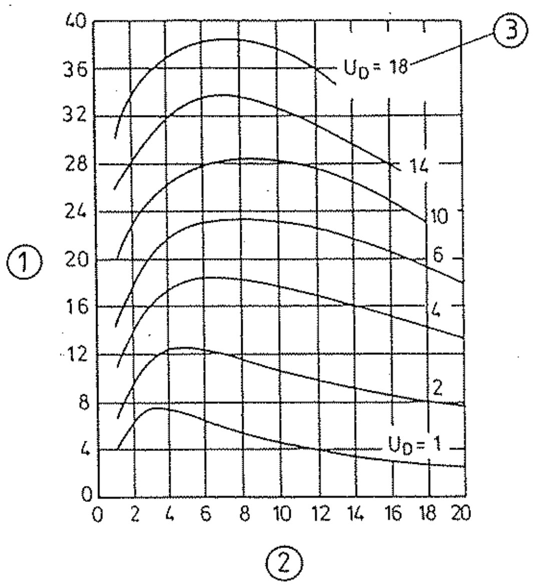


Figure 7.35. Filter criteria after Cistin & Ziems (1968).

1 Ratio D_{50}/d_{50}
2 Coefficient of uniformity $U_d = d_{60}/d_{10}$ of base soil
3 Coefficient of uniformity $U_D = D_{60}/D_{10}$ of filter

and a maximum of about 38 for a moderately graded base material and a well graded filter ($U_D \approx 18$), respective minimum and maximum ratios according to USBR being about 5 and 58. The comparison confirms again that the ratio D_{50}/d_{50} should not be taken as a design criterion.

The low value of all criteria based on a fixed ratio D_n/d_n results from the following consideration: the coefficient of uniformity and the permeability increase in the opposite sense, and the coefficient of uniformity and the retaining potential increase in the same sense. That means, given an increasing coefficient of uniformity and constant ratio D_n/d_n, that the filter becomes more conservative, but its discharge capacity decreases. This leads to increased costs because the low capacity filter must be made wider.

For similar reasons it is not regarded as essential, and it is not always reasonable, to make the filter and the base gradations parallel to each other. This criterion applies only – and is limited – to the range around D_{10} and d_{10} which control the permeability. The filter's permeability should at least be 2 powers of ten greater than the base material's permeability.

Coarse filters

Filters made of gravel-sand or gravel are needed as a transition between the fine filters, described above, and very coarse construction materials such as rockfill. In principle, the classical filter rules according to Equations (7.4b) and (7.5b) can be applied to the design.

Similarity of the gradings of fine and coarse filter is not unreasonable, but it is not a must. It eases the design that fine filters as the base material will not be extremely graded and will not develop flocs. All considerations on the effect of cohesion do not apply. Figure 7.36 illustrates the development of coarse filters as a protective filter for the finest non-cohesive filter no 8 in Figures 7.31 and 7.36.

Gradation no 9 in Figure 7.36 follows from min. $D_{50}/d_{50} = 5$ according to USBR. The permeability is only about 25 times that of the fine filter, thus demonstrating that the criterion does not meet the requirements.

Gradations no 10 and no 11 in Figure 7.36 do meet, approximately, a criterion of Witt (1986), the criteria of Cistin & Ziems (1968) and Equation (7.4b). Gradation no 12 fits with another criterion of Sherard et al. (1984a)

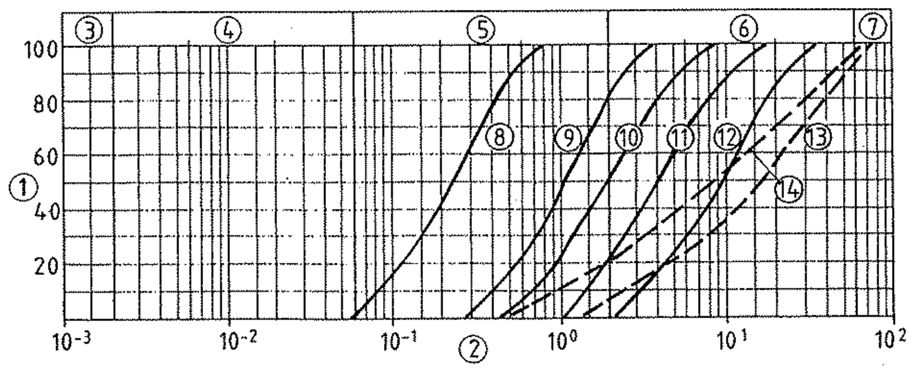


Figure 7.36. Illustration of the development of coarse filters.

1 Percent finer by weight
2 Grain size (mm)
3 Clay
4 Silt
5 Sand
6 Gravel
7 Cobbles
8 Non-cohesive fine filter, $k = k_F$
9 Medium filter, $D_{50}/d_{50} \approx 5$
10 Coarse filter, $D_{30} = 2.5\ d_{85}$, $U_D \approx 3$ after Witt (1986)
11 Coarse filter after Terzaghi & Peck (1948) and Cistin & Ziems (1968)
12 Coarse filter after Sherard et al. (1984a)
13 Upper boundary of well graded coarse filter to protect the fine filter no 8
14 Lower boundary of well graded coarse filter to protect the fine filter no 8, $k \geq 100\ k_F$

which was derived from tests on poorly graded sands and poorly to well graded gravel-sands:

$$D_{15} = 9\ d_{85} \tag{7.7a}$$

$$d_{85} = 0.11\ D_{15} \tag{7.7b}$$

The boundary between effective and ineffective filters proved to be sharply defined. As a rule, sand particles were able to pass the filter at $d_{85} < 0.10\ D_{15}$, and were retained by the filter at $d_{85} > 0.12\ D_{15}$. Taken $D_{15} = n\ d_{15}$ as a geometrical safety criterion, coarse filter no 11 is about twice as safe as filter no 12 with respect to the protected material no 8.

It can be concluded that the coefficient of uniformity of a coarse filter is of minor importance provided the filter is poorly or well graded, but not gap-graded. The essential function of erosion protection is controlled by the ratio D_{15}/d_{85}. The essential function of drainage capacity is controlled by the permeability, i.e. mainly by D_{10}. The desired ratio $k_{filter}/k_{base} = 100$ is approximately arrived at gradation no 10 and completely arrived at gradations nos 11 to 13.

The design concept of the upper boundary of well graded filters (gradation no 13 in Figure 7.36) to protect the fine filter no 8 is as follows:

– D_{15} must not exceed $9 \times d_{85}$, according to Equation (7.7a). Geometrical safety is assumed as equal to unit.

– The maximum grain size must not exceed 75 mm to minimize segregation of fine and coarse particles when the filter is placed.

The lower boundary gradation no 14 follows from the minimum desired permeability ratio of $k_{filter}/k_{base} = 100$.

The shape of the grains may be of importance for coarse, gravelly filters since the shape may affect the compactability and hence the permeability. Tests made by Sherard et al. (1984a) on sand and gravel of crushed rock show only little differences in comparison to alluvial, rounded sand and gravel of the same gradation. It is justified to conclude that filters made of rounded and of angular material can be designed according to the same criteria.

Internal stability

The design of filters must take account of the risk of segregation when the filter is placed. The risk increases with an increasing coefficient of uniformity. After segregation, fine particles may be washed into clusters of coarse particles. This would correspond to the start of internal erosion.

Soils may be internally unstable without segregation. This applies mainly to gap-graded soils where intermediate sizes between fine and coarse particles are missing. According to Sherard's suggestion (1979, also cited in Brauns 1990), internal stability can be checked as follows: the grain size curve is separated at any size into the finer and the coarser part. New grain size curves are plotted for both parts. For all pairs of curves it is $D_{15} \leq 5\ d_{85}$ if the original soil is internally stable. The problem is addressed in more detail by Lowe III (1988) and USCOLD (1993).

Internal stability of all base materials should be checked, as for instance that of fine filters as the base of coarse filters. As an example: it can be proven for the filter shown in Figure 7.32b that this material is internally stable. It can be verified by such an exercise that the risk of instability of graded materials increases with increasing D_{max} and with missing intermediate grain sizes. For this reason it is recommended to limit the grain size of filters to about 75 mm.

Dimensioning of filters
Darcy's law is used for dimensioning filters. It is

$$v = k \cdot i = Q/F \tag{7.8a}$$

$$Q = k \cdot i \cdot F \tag{7.8b}$$

where: Q = seepage quantity
k = permeability
i = hydraulic gradient
F = discharge area of the filter.

The development of filter dimensions shall be shown for the filters of an embankment dam with earth core (Fig. 7.37a). The inclined filter at the downstream face of the core has to drain the seepage quantity Q_1 penetrating the core. The quantity can be estimated after Equation (7.8b). With the denominations of Figure 7.37a it is

$$Q_1 \approx k_k \cdot H \cdot H/0.67\ L$$

The result of the estimation will be on the safe side in comparison to a more accurate computation using a flow net. For $k_k = 10^{-8}$ m/s, $H = 100$ m and $L = 50$ m it is $Q_1 \approx 3 \times 10^{-6}$ m³/s·m.

For the dimensioning of the inclined filter it is – simplifying – assumed that the whole quantity of seepage must be drained by the inclined filter with permeability k_1. There is no other draining layer, and no water flows into the downstream shell. This assumption is on the safe side in respect of the dimensions of the filter. The thickness of the filter B_1 at the toe, according to Figure 7.37b and respective denominations, is

$$B_1 = \frac{Q_1}{k_1 \cdot (\sin 90 - \alpha)} \cdot \frac{H_1}{H_1} \tag{7.9a}$$

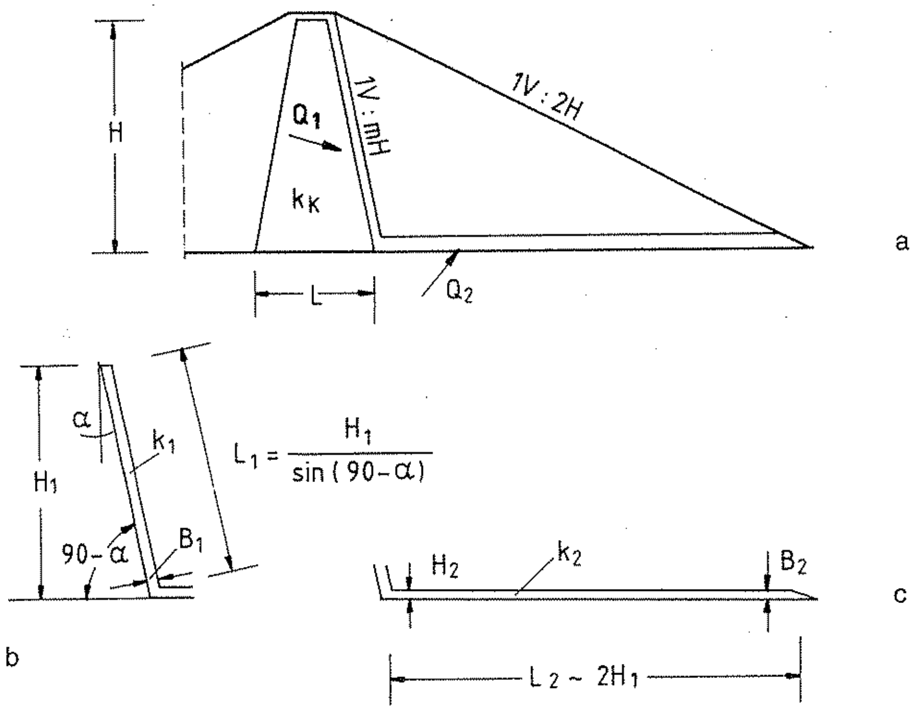


Figure 7.37. Illustration to estimate the dimensions of filters (see Fig. 7.38).

a Model of embankment dam with earth core
b Inclined filter
c Horizontal filter
Q_1 = Quantity of seepage through the dam
Q_2 = Quantity of seepage through the foundation

For a given thickness B_1 the required filter permeability will be

$$k_1 = \frac{Q_1}{B_1 \cdot (\sin 90 - \alpha)} \tag{7.9b}$$

In dimensioning the horizontal blanket filter it must be recalled that this filter has to drain the quantity Q_1 and, in addition, a quantity Q_2 which seeps through the foundation. No water is assumed to flow through the shell above the blanket filter. The head H_2 of seepage along the blanket filter is equal to the filter thickness B_2. With the denominations of Figure 7.37 it is

$$B_2 = \sqrt{\frac{(Q_1 + Q_2) \cdot 2H_1}{k_2}} \tag{7.10a}$$

and

$$k_2 = \frac{(Q_1 + Q_2) \cdot 2H_1}{B_2^2} \tag{7.10b}$$

The nomographical demonstration of Darcy's law in Figure 7.38 helps to estimate the dimensions of vertical, horizontal and inclined filters: place a triangle or ruler on the nomograph between the known parameters Q_1 and k_1 of the filter (example 1 for a vertical filter). Rotate the ruler in the pivot point on the center line D to the correct hydraulic gradient i and read the required filter thickness B_1 from the B-line. Example 2 for a horizontal filter demonstrates the use of the nomograph for given parameters B_2, i and Q_2 to find the required filter permeability k_2. Each of the parameters Q, k, i and F of Darcy's law can be determined with the aid of the nomograph.

In practice the filter thickness is affected by the availability of appropriate material and the costs and – for a small drainage capacity – by constructional restrictions. It is usual to design for high safety factors in the order of 10 to 100, that means to design for a seepage quantity 10 to 100 times the calculated quantity. High safety can easily be satisfied by magnifying the permeability of the filter within the limits of the filter rules, provided respective material is available at reasonable costs. The use of the Terzaghi-filter with $k \approx 5 \times 10^{-5}$ m/s in the example 1 of Figure 7.38 allows drainage of more than 100×10^{-6} m³/s·m at 2.5 m filter thickness, resulting in a factor of safety $Q_{\text{effective}}/Q_{\text{required}}$ of more than 50, according to the magnified permeability $k_{\text{effective}} = 5 \times 10^{-5}$ m/s instead of $k_{\text{required}} = 8 \times 10^{-7}$ m/s. The modification of the horizontal filter to a permeability of 10^{-3} m/s would also allow drainage of 100×10^{-6} m³/s·m. Magnified permeability will cover the imponderables of permeability tests. It is noted that flow in very coarse filters is turbulent, thus reducing the drain capacity.

On site, the inclined and vertical filters are placed without the use of formwork. Horizontal filters are levelled with tractor blades and graders. For construction reasons it is recommended to select the following thicknesses as a minimum:

- Vertical and inclined filters: 1.5 m,
- horizontal and near horizontal filters: 1.0 m.

These dimensions result in a drain capacity of the Terzaghi-filter of about 75×10^{-6} m³/s·m. With the assumption that the minimum horizontal filter has to drain twice this quantity – 150×10^{-6} m³/s·m under the same safety aspects and over a length of 100 m – the required permeability is about 10^{-2} m/s or 200 times greater than the permeability of the Terzaghi-filter.

The above mentioned minimum thicknesses do not account for a loss of discharge area at the outer boundaries by migrating particles (Fig. 7.30). The amount of such loss depends on the gradations of the base soil and the filter. The tendency is correctly reflected by Figure 7.35 where the ratio D_{50}/d_{50} decreases with decreasing U_D of the filter and increasing U_d of the base soil.

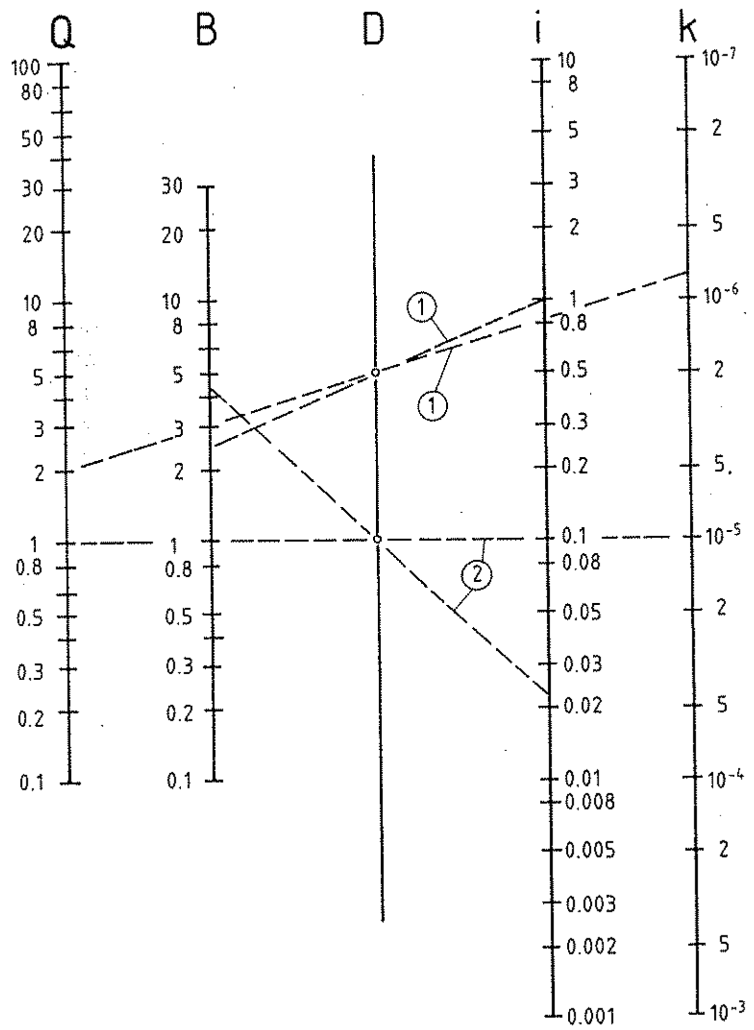


Figure 7.38. Nomograph to estimate the dimensions of filters (developed from Darcy's law, denominations see Fig. 7.37).

Q Seepage quantity (10^{-6} m³/s·m)
B Filter thickness (m)
i Hydraulic gradient
k Filter permeability (m/s)
D Pivot line

Example 1:
$Q_1 = 2 \times 10^{-6}$ m³/s·m
$k_1 = 8 \times 10^{-7}$ m/s
$i = 1$ (vertical filter)
Required filter thickness $B_1 = 2.5$ m

Example 2:
$B_2 = 4.5$ m
$i = 0.0225$ (horizontal filter, length 200 m)
$Q_2 = 1 \times 10^{-6}$ m³/s·m
Required filter permeability $k_2 = 10^{-5}$ m/s

The reasons for this are the increasing number of large pores with decreasing U_D and the increasing number of erodible particles with increasing U_d. In the extreme cases of $U_D < 3$ and $U_d > 15$, penetration may reach a depth of $25 \times D_{50}$ (Wittmannn 1982). The resulting losses of discharge area would be in the order of 10 cm with coarse filter no 11 of Figure 7.36, and almost 50 cm with filter no 13. It is, therefore, advisable to adjust the coefficients of uniformity of adjacent fine and coarse filters to each other, or to select small ratios D_{50}/d_{50}.

Some American references recommend much greater minimum thicknesses for inclined and horizontal filters, such as 4.0 m. This is probably due to the idea of compacting the filters to the most compact state. Dump trucks and compaction equipment can maneuver on a 4 m wide inclined or vertical filter.

It is now – almost generally – recommended to compact vertical and inclined filters to not more than a relative density of 70 or 80%. Then the filters offer sufficient deformation potential to compensate for differential deformations of the adjacent zones. In contrast, a well graded and heavily compacted sand-gravel might develop the greatest deformation moduli in the dam. Its behaviour would then be the contrary of that what is expected from a transition zone – a function which is attributed also to the filters.

Horizontal or near horizontal filters do not have this function of a transition zone. Thus, they must be well compacted to minimize settlements.

Filters and drain pipes

Frequently, filters end up in drain pipes which conduct the water to a greater distance. Such pipes must not allow fines of the filter to enter. Respective criteria for the filters and the pipes are based on D_{85}. For slotted pipes and for perforated pipes with circular holes the following is recommended:

USCE (1955):	D_{85}/slot width > 1.2
	D_{85}/perforation > 1.0
USBR (1973):	D_{85}/slot width > 2.0
	D_{85}/perforation > 2.0
DIN 4095:	D_{85}/perforation ≈ 3.0.

Surface erosion at interfaces

The sections above were concentrated on flow conditions normal or subnormal to the interface of the base material and the filter. Also, flow conditions parallel to the interface exist where surface erosion of the fine material may occur. Such conditions exist, for example, at the interface of two neighbouring filters, at the interface of a blanket filter and the foundation, at the interface of the dam material and a protective surface layer, or at the interface of layers of different permeability in the foundation, where seepage develops with the first reservoir impounding.

According to investigations surface erosion will not occur if the neighbouring layers are stable in the sense of conventional filter criteria. Equations (7.4a) and (7.4b) have proven to be conservative (USCOLD 1993).

In the case of parallel flow through existing layers in the foundation, the stability must be evidenced. An approach of Brauns (1985) allows us to determine critical hydraulic gradients for very poorly graded sands and gravels. The critical gradient is very small ($i < 1$) if D_{15} is greater than $5 \times d_{85}$ or if the ratio D_{50}/d_{50} exceeds about 10. This result is not surprising. It coincides with the geometrical approach to filter criteria like e.g. Equation (7.4b) and Figure 7.35. Nevertheless, the approach of Brauns is valuable in the sense that sealing measures in the foundation can be designed in a way which does not exceed critical gradients.

7.3.3.2 *Transition zones*

Transition zones are designed to adjust stresses of adjacent zones to each other. The need for such transition zones is repeatedly mentioned in the literature, e.g. by Seed (1979), as a precautionary measure against problems due to earthquakes. The effect of transition zones shall be demonstrated in the case of a rockfill dam with central earth core. As a rule the shells and the core are the zones with the most different deformation properties. Given optimum conditions the properties of the transition zones are between those of the shells and the core. There are two controversial cases:

1. The core is the most rigid element; its modulus of deformation is greater than that of the other construction materials. An example: the core consists of low plastic soil, and the shells of weak rock or of rock with a significant tendency towards breakage.

2. The core is the weakest element; the deformation moduli of filters, transitions and shells are progressively greater. An example: the core consists of plastic soil, and the shells of strong, well graded rockfill material without a tendency towards breakage.

With respect to the desired function of transition zones the material selection should result in progressively coarser grain size distributions from the core towards both outer slopes. A typical and almost ideal example is the Kinda dam shown in Figure 7.39 and Tables 7.7 and 7.8. Similar conditions of the material gradations exist with the majority of rockfill dams with an earth core.

Frequently, the filters are made of alluvial river deposits, i.e. of rounded particles, the graded material being well compactable. The material for the transition zones originates commonly from the same quarry as the shell material, or from similar rock of necessary excavations. The transition material does not contain the large blocks of the typical rockfill material. That means

Table 7.7. Examples of rockfill dams with central earth core. Gradation of materials and compaction method.

Dam zone	Width (m)	Material	D_{max} (mm)	d_{min} (mm)	C_U[5]	Fill water content –/+ of opt. (%)	Layer thickness (m)	Weight of compactor (kN)	Number of passes
Srinagarind dam[1]									
Core		Clayey sand, SC low compressible	150		> 100	0/+2	0.4	> 100	4
Filter	u/s 3 to 5 d/s 3 to 5	Crushed limestone, alluvial gravel-sand	150	0.05	10 to 100	d m	0.4	> 100	3
Transition	u/s 5 to 20 d/s 5 to 20	Crushed limestone	300	0.05	15 to 100	d	0.4	> 100	4
Shell: inner part		Strong limestone	700	1.0	8 to 15	d	1.0	135	4
outer part		Strong limestone	1500	1.0	10 to 20	d	2.0	135	4
Kinda dam[2]									
Core		Sandy silt, clayey, CL	30	10^{-3}	> 100	0/+2	0.25	200	5
Filter	u/s 2 d/s 2	Alluvial gravel-sand	50	0.06	60	m	0.4	–	–
Transition	u/s 6 d/s 3	Crushed diorite	150	1.0	5 to 10	d	0.6	150	2
Shell		Crushed andesite	1000	2.0	10 to 15	d	1.0	150	5
Monasavu dam[3]									
Core		Clay (halloysite), CH	100	92% < 0.074		+ 20	0.1	175 (LGP)	> 6
Filter	u/s 3 to 4 d/s 4 to 5	Gravelly sand (crushed monzonite)		3 to 7% < 0.074		d			
Transition	u/s 4 d/s 5[4]	Sandy gravel (crushed monzonite)				d			
Shell		Crushed monzonite				d			

[1]Champa & Mahatharadol (1982)
[2]Kutzner et al (1988)
[3]Knight et al. (1982, 1985)
[4]Widened at the toe
[5]Coefficient of uniformity
d = Quarry-run, no addition of water
m = Moist, as supplied from deposit or washing plant
LGP = Low ground pressure tracks

Table 7.8. Examples of rockfill dams with central earth core. Properties of compacted materials and assumptions for computations. References as in Table 7.7.

Dam zone	Material (abbrev)	Dry unit weight (kN/m^3)	State of compaction	Friction (°)	Cohesion (kPa)	Modulus of deformation (MPa)	Failure strain (%)	Poisson's ratio
Srinagarind dam								
Core	Clayey sand	18.5 (18.0)	100% Pr	(17)	(40)			
Filter	Gravel-sand	20 to 22 (20)	≥ 65% D_r	(35)	(0)			
Transition	Limestone	(20)		(35)	(0)			
Shell: inner part	Limestone	(18)	22% n	(33)	(0)			
outer part		(17.5)		(39)	(0)			
Kinda dam								
Core	Silt	20	100% Pr	(25) CD (17) CU	(50)	(13.5)[1]	6 to 10	(0.33)
Filter	Gravel-sand	21	70% D_r	(35)	(0)	(17.5)[1]		(0.23)
Transition	Diorite	21	80% D_r	(35)	(0)	(17.5)[1]	9 to 12	(0.23)
Shell	Andesite	22	95% D_r	(42)	(0)	(29)[1]		(0.15)
Monasavu dam								
Core	Clay	8.6		30	15 34 (UCS)	(2 to 5)	10 to 20	(0.4)
Filter	Crushed sand					(60)		(0.3)
Transition	Crushed gravel					(60)		(0.3)
Shell	Monzonite					(60)		(0.3)

[1]Tangent modulus
Pr = Standard Proctor density
D_r = Relative density
n = Pore volume
() = Assumed values

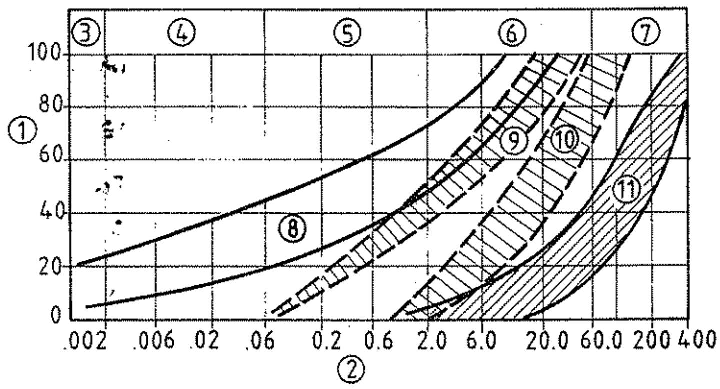


Figure 7.39. Kinda rockfill dam as in Figure 5.4. Range of material gradations.

1 Percent finer by weight
2 Grain size (mm)
3 Clay
4 Silt
5 Sand
6 Gravel
7 Cobbles
8 Core material
9 Fine filter
10 Coarse filter downstream and transition zone upstream
11 Rockfill material for the shells

the tendency towards edge breaking is reduced, making the transition material less deformable than the rockfill material. These conditions – rounded filter material and strong transition material – bear the risk that the transition zone is going to be the stiffest element in the dam. This is in contrast to its desired function.

The following typical examples will show how the problem is approached in practice. Table 7.7 gives typical gradation data of the materials of the Srinagarind, Kinda and Monasavu dams. The development of the gradations from the core to the outer slopes is as shown in Figure 7.39.

At Srinagarind the core is the stiffest element. This follows from the low compressibility of the SC-material. A further indication is the low compaction effort of the shell material, namely 4 passes only at 1.0 and 2.0 m layer thicknesses. The deformability of the filter at 65 to 90% relative density and of the transition zone is probably between that of the core and of the shells (Table 7.8). Measured settlements (Table 7.9) of the shell at 100 m dam height are the same as of the core at 140 m dam height, namely 0.8 m. The settlement of the shell as a percentage of dam height is more than that of the core.

At Kinda the core is weaker than the shells. The desired function of the transition zone was supported by the availability of materials on site. Diorite 1 to 150 mm was available as tunnel muck from the diversion tunnel, and was used as downstream coarse filter and as upstream transition. The

Table 7.9. Examples of rockfill dams with central earth core. Measured settlements. References as in Table 7.7.

Dam zone		Settlements inside the dam until end of construction	Total settlements inside the dam	Total settlements at the crest	Years after end of construction
Srinagarind (H = 140 m)					
Core	(m)			0.8 (H_{140})[1]	2.5
	(% of H)			0.57	
Shell d/s	(m)			0.8 (H_{100})	2.5
	(% of H)			0.8	
Kinda	(H = 76 m)				
Core	(m)	0.95 (H_{35})	1.42 (H_{35})	0.54 (H_{76})	3.3
	(% of H)	1.25	1.87	0.71	
Shell d/s	(m)	0.58 (H_{26})	0.65 (H_{26})		3.3
	(% of H)	0.76	0.86		
Monasavu	(H = 85 m)				
Core	(m)	1.68 (H_{35})	1.80 (H_{35})	0.33 (H_{85})	1.5
	(% of H)	1.98	2.1	0.39	
Shell d/s	(m)	0.32 (H_{35})	0.45 (H_{35})		1.5
	(% of H)	0.38	0.53		

[1](H_{index}) = Height of the measuring instrument above foundation (m)

strength of the diorite was – on average – 10% less than that of the andesite which was used for the shells. The andesite strength is 80 to 100 MPa. In addition, the builders attempted to support the transitional effect by different compaction of the materials (Tables 7.7 and 7.8). The well graded fine filter was not separately compacted. The specified relative density of about 70% was achieved by the compaction of the neighbouring zones. Measured settlements reveal well adjusted conditions. Plausible tendencies can be seen (Table 7.9):

– Settlements of the core in m and as a percentage of the dam height are greater than those of the shell.

– Settlements inside the dam from the end of construction to the end of the measuring time (3.3 years) increase, plausibly by around 50% in the core and by around 12% in the less deformable shell.

Particular conditions prevail for the Monasavu dam with wet core and stiff shells (Table 7.9). The settlements of the core are greater than those of the Kinda dam core, after a considerably shorter time of observation. But the core of the Kinda dam was consolidated to a higher degree after 3.3 years than that of the Monasavu dam after 1.5 years. According to Knight et al. (1985) there is a difference of internal settlements between the core and the downstream shell of 1.4 m. This was dissipated in the outer 12 m wide portion of the core, not in the filter and the drain layers. Obviously, this was tolerable due to the large failure strain of 10 to 20% of the core material. And obviously, the filter and the drain layers of about 9 m width constitute the most rigid element of the structure. The measured settlement of these layers was slightly less than that of the shell material at a distance of 16 m from the center line of the dam, at the same elevation H_{35}.

Table 7.8 shows material properties and data for computations of the three dams. For Srinagarind these data are assumptions for slip circle computations. For Kinda and Monasavu, the data served for back-analyses using finite element methods. Monasavu was analyzed with a linear-elastic approach of the stress-strain relationship. Kinda was analyzed using a non-linear, stepwise linear adjusted approach. Core, filter, transition and shell had individual material parameters. The results show acceptable agreement with measured deformations, in spite of the different approaches. The core of Monasavu is considerably more deformable than the core of Kinda. The shells of both dams are stiffer than the cores. The difference in stiffness is significantly greater with Monasavu than with Kinda.

The width of the transition zone should be adjusted to the expected differential settlements of core and shells. As a rule, for dams with an inclined core, the maximum difference will occur at the downstream face of the core. The movement of the core at the interface of core and downstream transition causes shear deformation there which exceeds the deformation at the upstream interface. Therefore, the transition downstream should be made wider. Example: the Monasavu dam with inclined core and a downstream face sloping of 1V:0.15H (Table 7.7).

For dams with a vertical core and equal shell materials on both sides, the differential settlements due to dead load will be approximately identical. Horizontal displacement and rotational movement of the core due to impounding will cause increased movements downstream. Therefore, the downstream transition zone is frequently made wider. Example: the Srinagarind dam with vertical core.

Differential settlement upstream is affected by the reservoir operation. First impounding will cause heaving of the upstream shell due to uplift and post-construction settlement if the strength of the shell material is affected by saturation. The interaction of heaving and settlement due to the operation of the reservoir will continue over a certain period. It is up to the designer to value these different conditions upstream and downstream and to size the transition zones accordingly. Example: the Kinda dam with vertical core and wider transition zone upstream (Table 7.7). For this example it is justified to neglect the shear movement at the interface of core and downstream filter during impounding. This follows from the very small horizontal core displacement at the crest, which was measured as a few centimeter only in the main section, with the reservoir at 95% of its full supply level.

7.3.4 *Dam foundation*

7.3.4.1 *Foundation of the sealing element*

The sealing element rests on rock or soil which is capable of bearing the loads and which is of low compressibility. The rock or soil should be impermeable in a practical sense, otherwise it must be made impervious by proven means.

Competent rock foundation

The majority of existing dam cores rest on strong, fresh or slightly weathered rock. The approval of the foundation area is the criterion for starting embankment works. Competent rock means rock of weathering grade WI or WII according to the classification system of Table 3.2. Given these preconditions the core will be compatible with the settlements and differential settlements of the foundation. The core will remain intact. The effect of rock discontinuities is of minor importance due to the width of the core. The exceptions are karstic and soluble rock where rock improvement measures may be required.

The foundation of natural face sealings is less critical since such sealings are applied mainly in the case of small dams or with dams of temporary function, such as cofferdams.

The foundation area must be cleaned of soil and weathered rock. After this, detailed treatment is required (dental concrete etc). This treatment serves to create a smooth foundation area and hence favourable stress conditions without irregularities in the lower portion of the core. This is discussed in Section 9.8.

Due to the rock conditions consolidation grouting of the upper 5 to 10 m may be advisable. This grouting serves to minimize settlements and seepage. It may cover the whole foundation area of the core, or it may be limited to a center portion, for instance 2 rows upstream and downstream of the grout curtain. Commonly, consolidation grouting is performed through vertical boreholes at a spacing of 2 to 4 m in both directions.

An inspection gallery, if appropriate, is located below the core (Fig. 7.40). Excavation of the trench is assisted by smooth rock blasting. Slight disturbance of the adjacent rock cannot be excluded. For this reason a number of experts does not favour installation of a gallery. The value of galleries is controversial worldwide. The matter has been discussed in detail by the author (1982b, 1996). Necessarily, consolidation grouting must repair any rock disturbances.

The intention of placing the core on strong and impermeable rock will affect the shape and location of the core which must be adjusted to the local conditions. An instructive example is the Bolgenach dam where such adjustment necessitated great effort in design and construction (Innerhofer

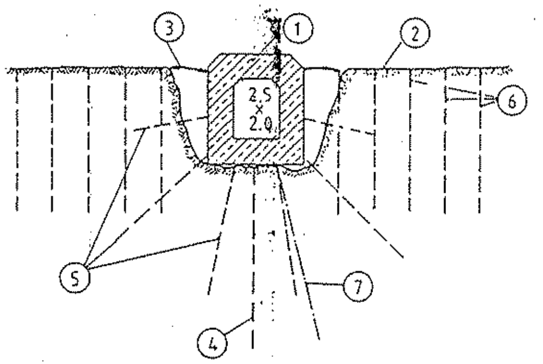


Figure 7.40. Arrangement of inspection gallery and foundation grouting.

1 Inspection gallery (minimum inner dimensions height × width in m)
2 Rock foundation
3 Lean concrete
4 Grout curtain
5 Contact grouting
6 Consolidation grouting
7 Piezometer (measuring sections only)

1980, Innerhofer & Loacker 1982). The upstream lower section of the core contacts sandstone up to a vertical height of 30 m (Fig. 7.41). The contour shows overhangs of the sandstone of up to 8 m. The center portion of the core rests on marl and sandstone, after excavation of disintegrated materials. The contour shows a considerable difference in the elevations of the marl and the sandstone surfaces.

The core consists of moraine material of low plasticity. The shells are made of low permeable gravels. The materials are reported to have quite reasonable deformation properties with respect to the boundary conditions. The shape of the core, being inclined upstream in the lower part and vertically in the upper part, fits well with the foundation conditions. However, FE-computations showed low vertical stresses in the core/foundation interface, resulting in the risk of arching in longitudinal and transverse directions and hydraulic fracturing. It was minimized by the following measures:

– Sealing of open joints in the sandstone at the upstream interface of core and sandstone.

– Placing of plastic moraine at the bottom of the core. It was placed like shotcrete, resulting in a very dense state and giving a good bond to the marl and sandstone underneath.

– Grouting of the marl including crack grouting to achieve locally increased stress conditions.

Careful control of seepage reveals impermeability of the core and its foundation. The seepage quantity is reported to be 0.25 l/s.

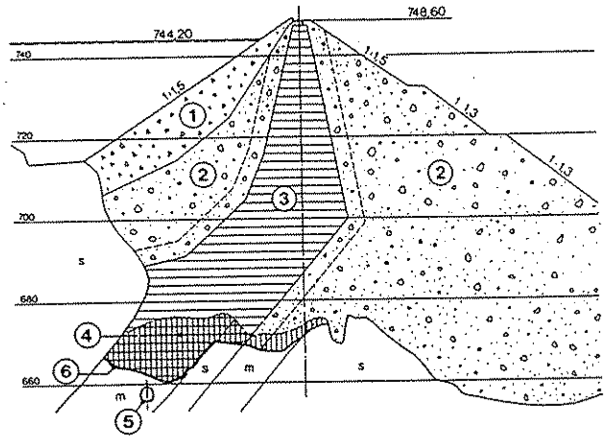


Figure 7.41. Bolgenach dam, typical section (elevations m a.s.l., adapted from Innerhofer & Loacker 1982).

1 Rockfill material, max. grain size 800 mm
2 Gravel, max. grain size 600 mm
3 Moraine, max. grain size 90 mm
4 Excavation of disintegrated marl and sandstone
5 Concrete lined inspection gallery, diameter 2.6 m
6 Moraine, 0 to 30 mm + 3% bentonite
m Marl
s Sandstone

Weathered rock and soil foundation

Reasons of economy may result in placing the sealing element on weathered rock or soil. In such cases it is a must to reduce seepage and settlements, by respective measures, to tolerable rates. Sealing measures may considerably affect the design.

An example is the Agus IV dam (Fig. 5.3). The investigations of the design phase were not fully successful in identifying the degree of weathering of the basalt in the foundation. But it was verified that excavation down to fresh or slightly weathered basalt was not possible at justifiable cost. Moderately to highly weathered basalt was selected as the entire foundation rock. During the excavation works, visual inspection determined at which elevation the excavation was to be stopped and the dam placed. From percolation tests in boreholes the horizontal permeability there was estimated as 10^{-4} to 10^{-3} m/s. The compressibility of the foundation was regarded as of minor importance. This was confirmed by related settlements of only 30 mm, several months after impounding.

A field grout test with ordinary cement resulted in only partial success. A reasonable take of grout was limited to isolated zones. Other zones proved not to be groutable since the weathered rock there was rather a soil. In terms of soil mechanics it was a silty sand-gravel, which is not groutable.

In principle it was accepted to live with a semi-pervious foundation. It was decided to perform ordinary cement grouting below the center portion of the dam to produce the best possible type of grout curtain. In addition, an impervious blanket of clayey silt was placed upstream to reduce the seepage flow and the risk of erosion. The blanket covers the valley bottom over a length of 180 m measured from the center line of the dam.

The concept proved to be appropriate. The piezometers and pore pressure gauges in the downstream part of the dam and in the foundation indicate free flow of seepage in the horizontal downstream filter. After first impounding the quantity of seepage was 8 to 10 l/s, with respect to the center portion of the dam, 150 m in length. A considerable part of this quantity was seen to be attributed to a local defect of the grout curtain/grout cap interface. Probably, the grout cap had been lifted during grouting.

A prominent example of a dam on soil, and hence on a permeable foundation, is the Tarbela dam. There the substrata of the Indus valley consist of alluvions down to a maximum of 230 m depth, with the characteristic grain size composition of cobbles, gravel and fine sand. The grain sizes of fine gravel, coarse and medium sand are missing. Occasionally the fine and coarse portions are separated, resulting in pure sand layers and highly permeable layers of cobbles and gravel (similar to Fig. 4.7). In the sense of filter rules the coarse and fine layers are not stable at the interface. The risk of internal erosion exists at respective hydraulic gradients. As is known, all investigatory works in such soil conditions – and under water – are extremely problematic. It is almost impossible to collect sufficiently intact samples.

As an alternative to a vertical sealing element, an upstream blanket was placed in the valley. The length is in the order of 2000 m. It is designed for a hydraulic gradient not exceeding $i = 0.07$. The blanket consists of a processed, well graded mixture of sandy silt and crushed gravel. At the first impounding, almost to the full supply level, a seepage flow of 6.4 m^3/s was measured, in contrast to a predicted rate of 0.15 m^3/s (Izar-Ul-Haq 1991). After emptying the reservoir 400 sinkholes at a diameter of up to 4.5 m were visible in the blanket. The bottom of all the sinkholes showed gravel with poor or no sand content (open-work gravel).

The sinkholes were filled with filter material and the thickness of the blanket was generally extended to 5 m, resulting in a gradient of $i \approx 0.03$ across the blanket. In addition, grouting and drainage measures were executed at both the abutments. According to Izar-Ul-Haq all these repair measures were successful. The quantity of seepage has progressively been reduced since 1977 to negligible rates after 1985. The almost linear reduction between 1977 and 1985 indicates a considerable effect of reservoir sedimentation, which frequently produces a sedimentary layer of silt in the vicinity of the dam.

7.3.4.2 *Foundation of the shells*

The shear strength of the material in the foundation should be equal to or higher than the shear strength of the shell material. If the shear strength is lower the slope of the embankment must be flattened, or stabilizing berms must be dumped.

Rock and coarse-grained soil foundation

Satisfactory shear strength – approximately equal to that of rockfill shells – is usually attributed to moderately weathered rock grade WIII of Table 3.2 and to dense or medium dense coarse soil, like slightly sandy gravel, gravel and cobbles. Foundation and shell must be compatible in the sense of filter rules. This applies mainly to the downstream part of the dam, where seepage from the foundation into the shell tends to wash fines of the foundation into the voids. Such fines originate from joint fillings of rock and from silt as a component of coarse soil. Areas from which fines might be eroded must be covered with filter material. The cover may be restricted to local areas.

The rockfill material of the shells is free-draining. It provides sufficient drain capacity for any quantity of seepage. Other materials, which do not drain freely, require a coarse drain layer to provide sufficient drain capacity. This layer must be compatible with the materials underneath and above to exclude migration of fines. Therefore, a filter and drain system of three layers is required, namely – from bottom to top – a fine filter to exclude erosion of the foundation, a coarse layer for drainage, and a fine filter to prevent migration of fines from the shell to the drain layer, if the shell material contains fines.

Similar conditions apply to the upstream shell of an embankment dam on jointed rock with partially open joints. A filter must exclude the erosion of fines from the shell into the foundation. Such erosion might lead to unacceptable settlements of the shell and eventually to piping.

The Finstertal dam provides an instructive example of how to improve the shear strength of the foundation, as described by Schwab & Pircher (1979). The dam rests in part on an inclined rock surface which is polished by glacial abrasion (Fig. 7.42). It has an internal sealing of asphaltic concrete. The case is described in this chapter because it reflects the conditions of a rock foundation of the dam shell. The same problem would arise for a dam with a natural sealing.

At the dam site the bedrock consists mainly of unweathered schistous gneiss. The rockfill material of the shell (zone 3a in Fig. 7.42) is a well graded material of granodiorite. The friction angle is reported to be 45° at 300 kPa normal stress and 42° at 900 kPa normal stress. The shear resistance

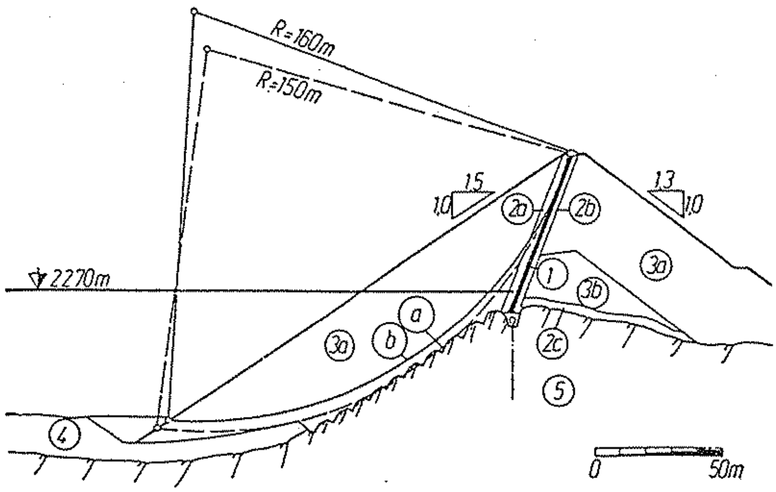


Figure 7.42. Finstertal rockfill dam. Critical slip circles with and without artificial roughening of the rock surface (adapted from Schwab & Pircher 1979).

a Critical slip circle without roughening, factor of safety $f = 1.265$
b Critical slip circle with roughening, factor of safety $f = 1.415$
1 Asphaltic concrete
2a Transition zone, moraine 0 to 100 mm
2b Transition zone, rockfill 0 to 100 mm
2c Drainage zone, rockfill 0 to 700 mm
3a Shell, rockfill 0 to 700 mm
3b Inner shell, moraine 0 to 700 mm
4 Moraine
5 Bedrock, schistous gneiss

at the interface of the rockfill material and the foundation rock was tested by comprehensive large-scale tests with a shear area of 1 m^2, similar to direct shear tests (Schober & Rostek 1979). Two big slabs of schistous gneiss were removed from the site to be used as quasi undisturbed rock samples for the tests. The slabs were artificially roughened by chiselling grooves of 3.5, 7.3 and 10 cm depth into the polished slabs. The tests revealed partially unexpected but plausible results. The tendencies will most probably be valid also for other types of rock. The main results are:

– The shear resistance at the interface of rockfill and rock is stress-dependent, as is the internal shear resistance of granular materials,

– the shear resistance increases with the roughness of the rock surface to rates of 94 to 98% of the internal shear resistance of the rockfill material,

– the shear resistance at the interface is a function of the grain size distribution of the rockfill material. It is slightly greater with a material 0 to 200 mm than 0 to 600 mm. Interlocking is more effective with the finer material.

On site the V-shaped grooves of approximately 70 cm depth have been produced by blasting. The depth is equal to the maximum grain size of the rockfill material. The nominal alignment of the grooves is parallel to the center line of the dam. It was optimized in respect of a desired minimum of blasting work. 'Thus, by comparatively economical measures a substantial improvement in the stability of the dam is achieved' (Schwab & Pircher).

Fine-grained soil foundation

The effect of the shear strength of the foundation on the stability of the dam is enlarged in the case of dams on fine-grained soil. The reason is the time-dependence of the shear strength due to the development of pore-water pressure and its dissipation. This is explained by an example.

The foundation of two 40 and 54 m high dams of the Nipawin project consists of glacial till (Fig. 7.43). Two thick beds of middle and lower till are to be found below an alluvial surface layer. Underneath is clay shale with a residual strength corresponding to a friction angle of 9°. It is the critical potential failure surface for the main dam.

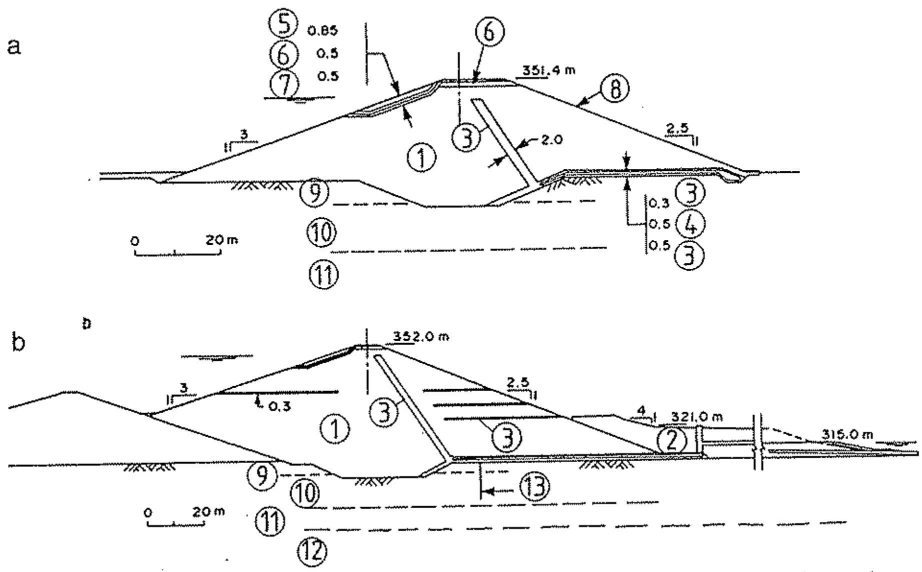


Figure 7.43. Examples of embankment dams on soil foundation, Nipawin project (adapted from Morgenstern et al. 1991).

a West dam
b Main dam
1 Compacted till
2 Random fill
3 Sand filter
4 Gravel drain
5 Riprap
6 Riprap bedding
7 Sand-gravel filter
8 Grass
9 Alluvium
10 Middle till
11 Lower till
12 Clay shale
13 Relief holes at 10 m spacing

Both dams are constructed of compacted till from excavation works. They are protected and drained by granular chimney drains and blanket drains under the downstream shell. In addition, horizontal drains are placed in the downstream shell of the main dam as a precautionary measure against unexpected high pore-water pressure. The dam design accounts as follows for the relation of pore-water pressure, shear strength and dam stability:

– The development of pore pressure in the foundation and in the dam during the period of construction requires flat outer slopes of 1V:3H and 1V:2.5H, irrespective of the great shear strength of the till in situ in the foundation and compacted in the dam, of about 35 to 40°.

– The existence of the clay shale close to the bottom of the main dam necessitates a downstream stabilizing berm of random fill.

– The maximum horizontal displacement was measured in the area of the berm as 33 cm. The movement was stopped by placing an additional 4 m of fill on the downstream berm.

– The pore-water pressure increased in all fine-grained zones until the end of construction to parameters of 0.12 to 0.43 for middle till and of 0.7 for clay shale.

– The calculated dam stability corresponds with a factor of safety of 1.4 at maximum pore-water pressures. It increases to 1.5 with the long-term dissipation of the pressures. The actual safety factor will be slightly higher since the measured pore pressure parameters are below those assumed for the computations.

– Pressure relief holes accelerate the dissipation of pore pressure. Logically, they are located below the inner part of the downstream shell, where the stresses and the pore pressure will be maximum and where a maximum of shear strength is desirable.

A comment on such relief holes is useful. The main purpose is to support the dissipation of pore pressures which develop in the foundation due to the dam load. In this way the relief holes are helpful in maintaining or redeveloping shear strength. In addition, they are effective in reducing uplift. However, this effect is not required to control the stability of embankment dams, in contrast to concrete structures. In the foundation of embankment dams uplift is automatically reduced to the head of the tail water at the place where seepage enters the downstream system of filters and drains.

Particular attention must be paid to dams on a sand or silty sand foundation, which is prone to liquefaction under dynamic loads. After some failures of such dams, engineers tried to avoid building dams at such locations. In recent years, corresponding to increasing knowledge about the relation of sand behaviour and dynamic load, dams on sand foundation have been constructed. An example is the Nagara dam, described by Takahi et al. (1991). The two main problems of this project are the permeability of the foundation and related seepage flow and the risk of liquefaction of the foundation. Here, only the second problem is addressed.

The dam consists essentially of medium and fine sand with about 10% silt. The permeability is reported to be 10^{-6} m/s. The main part of the dam rests on sand of the same type, reaching to a depth of 7 to 12 m. Below the sand is a bank of siltstone which is again followed by sand. The siltstone bank is 2 to 5 m thick. The face sealing of the dam, consisting of cohesive soil, penetrates into this bank. The upstream dam slope is 1V:2H.

The grain size distribution of the sand is deemed to be in the range of the soils that are highly susceptible to liquefaction (no 7 in Fig. 4.27). The sand of the foundation is saturated from the tail water. The peak ground acceleration of the design earthquake is 0.2 g. These conditions typically indicate the risk of liquefaction.

According to the approach of Seed & Idriss (1971), liquefaction relates to the ratio of existing normal and shear stresses to the dynamic stresses. The existing stresses are a function of the density of the sand. Therefore, the liquefaction potential can be approached by: a) cyclic triaxial tests to determine critical dynamic stresses at different states of density, and b) determination of the sand density in situ.

The following was found for the actual case: the relative density of the sand in the dam should be 95% (moist) and 97% (saturated) to exclude liquefaction safely. The state of the sand in the foundation should be 'very dense', corresponding to a number of blows $N \geq 30$ for 30 cm penetration with the standard penetration test. Respective tests revealed values of $N = 30$ to 50 in the upper zone of the foundation and $N > 50$ in the lower zone. Accordingly, satisfactory safety against liquefaction can be assumed.

7.4 DAMS WITH ARTIFICIAL SEALING

7.4.1 *Dam zoning*

Artificial sealings of asphaltic concrete and of conventional concrete have been constructed for very large dams, since comprehensive knowledge and experience about these materials have been available in theory and practice, and since the compaction methods of the embankment materials result in very high moduli of deformation. Asphaltic concrete is used for internal and for face sealings, while concrete is used mainly for face sealings.

The dam body may consist of different zones, according to the stress conditions. The outer, downstream part which is not affected by the water load, is usually made of material of lesser quality, or the compaction effort for this part of the dam is slightly reduced. Examples are given in Figures 5.6 and 5.8. Figure 5.6 demonstrates also the zoning of the upstream part of a dam with internal artificial sealing. For earthquake resistance, the upstream shell must consist of free-draining material.

The zoning of one of the largest concrete face rockfill dams, Foz do Areia, is shown in Figure 7.44. A comment on the zoning was given by Wilson & Marsal (1979). According to this, the transition zone under the concrete face is a crusher-run rockfill with a maximum grain size of 150 mm and a permeability of 10^{-6} m/s. The layer is 4 m wide (horizontal) at the crest and 10 m at the toe. The zoning of the dam enables safe storage of water without the concrete face in order to control construction floods. For this purpose the permeabilities of the different zones increase continuously from upstream to downstream (Fig. 7.44): a semi-pervious layer under the face (IIB) – smaller sized rockfill in the upstream portion (IB, ID) – free-draining rockfill at the downstream base (IC) – large rocks at the downstream toe (IA).

7.4.2 *Sealing elements*

7.4.2.1 *Internal sealing of asphaltic concrete*

Asphaltic concrete is a mixture of mineral aggregates and bitumen as the binding agent. It is an elasto-plastic material. In the dam asphaltic concrete is subjected to axial and transverse strain. The question arose up to which

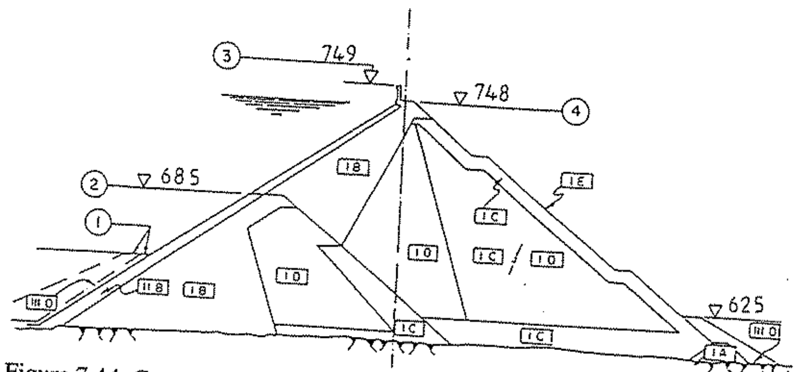


Figure 7.44. Concrete face rockfill dam. Zoning of Foz do Areia dam (elevations m a.s.l., adapted from Pinto et al. 1982).

1 Clay protection
2 Elevation of 1st stage
3 Top elevation of parapet
4 Crest elevation
IA Dumped basalt
IB Compacted basalt, layers 0.8 m
IC Compacted basalt, layers 1.6 m
ID Compacted basalt and basaltic breccia, layers 0.8 m
IE Selected placed basalt > 0.8 m
IIB Compacted crushed basalt < 150 mm, layers 0.4 m
IIID Impervious soil < 20 mm, layers 0.3 m

rate of strain the material would remain sufficiently impervious. Tests made by Breth & Schwab (1973) resulted in permissible strains of 14% axial and 6% transverse. The elasto-plastic behaviour enables asphaltic concrete to follow the deformations of the adjacent dam materials and the foundation up to these rates without cracking, remaining intact, i.e. impervious. Filter and transition zones prevent the asphaltic concrete from migrating into the large pores of rockfill shells.

Under these preconditions vertical and inclined membranes of asphaltic concrete are constructed as sealing elements of dams. The width of existing membranes is in the range of 0.4 to 1.2 m. It was recently recommended by ICOLD (1992) to keep the width constant from bottom to top, other than with the examples of Table 7.10. The persistent tightness and satisfactory long-term performance of the membrane must be guaranteed by the support given by the dam shells, by the composition of the asphaltic concrete and by the placing method. One disadvantage is that repair measures are very difficult. From the ecological point of view an internal sealing is preferable to a face sealing.

Apart from this, the selection of an internal sealing of asphaltic concrete will be related to the particularities of the project. This is demonstrated by the following example. According to Arnevik et al. (1988) an artificial sealing was selected for the Storvatn dam because there was no cohesive soil available at reasonable costs and because there was no prototype in Norway with an earth core of processed moraine. In addition, construction delays due to weather conditions could not be excluded. An internal sealing was preferred to a face sealing because impounding could start prior to the end of construction. This offered an economic benefit to the owner.

The position of an internal sealing within the dam is selected according to the deformation conditions of the dam and of the sealing. Because of its small weight and volume the thin membrane must follow the movements of the dam shells. The membrane does not affect the dam deformations. They depend mainly on the geometry of the structure, on the properties of the materials and on the shape of the valley. Computations and studies using different parameters serve to find appropriate design criteria for each project.

Criteria for the location of the internal sealing are explained by the examples Grosse Dhünn with vertical membrane, distorted in the upper section, and Finstertal with inclined membrane. These dams have been investigated in detail by Breth & Arslan (1990). The results allow us to evaluate the two different systems, namely vertical and inclined membranes. The typical sections of the dams are shown in Figures 5.6 and 7.42. The Megget dam is taken as a third example which has a vertical membrane from bottom to top. With all three dams, the membrane rests on a concrete structure with an inspection gallery, being placed on strong rock. The center lines of Megget and Finstertal are slightly curved. According to measurements it can be as-

Table 7.10. Examples of embankment dams with asphaltic concrete core. Characteristic data of construction materials (see also Table 7.11).

Project		Grosse Dhünn[1]		Finstertal[2]		Megget[3]
Shell u/s		Inner part	Outer part	Inner part	Outer part	
Material		Greywacke and siltstone		Granodiorite		Sandy gravel
d_{max}	(mm)	300	600	700	700	400
< 2 mm	(% by weight)				0 to 20	10 to 30
< 0.06 mm	(% by weight)				0 to 5	0 to 10
Layer thickness	(cm)			75	100	35 to 40
Vibratory roller	(kN)			150	150	55
Number of passes				6	6	4
Dry unit weight	(kN/m³)	23	23			
Pore volume	(%)			19 to 24	19 to 24	
Angle of friction	(°)			42 to 45	42 to 45	
Shell d/s						
Material		Greywacke and siltstone		Moraine	Granodiorite	Sandy gravel
d_{max}	(mm)	300	600	700	700	400
< 2 mm	(% by weight)			20 to 30	0 to 20	10 to 30
< 0.06 mm	(% by weight)			5 to 10	0 to 5	0 to 10
Layer thickness	(cm)			100	100	35 to 40
Vibratory roller	(kN)			150	150	55
Number of passes				6	6	4
Dry unit weight	(kN/m³)	23	23			
Pore volume	(%)			19 to 24	19 to 24	
Transition u/s						
Material		Greywacke		Moraine		
d_{max}	(mm)	56		100		100
< 2 mm	(% by weight)			25 to 45		5 to 10
Width	(m)	2.0		3.0		1.5
Transition d/s						
Material		Greywacke		Granodiorite		
d_{max}	(mm)	56		150		100
< 2 mm	(% by weight)			0 to 20		5 to 10
Width	(m)	3.0		2.0		1.5

[1]Cords (1982, 1989), Breth & Arslan (1990), Kuhlmann (1989)
[2]Pircher & Schwab (1988), Breth & Arslan (1990), Schwab & Pircher (1979)
[3]Penman & Charles (1985), Gallacher (1988, 1989), Charles & Penamn (1988)

sumed that the different shell materials have been well compacted to achieve proper strength and deformation properties. The parameters of the materials and of the compaction method are listed in Table 7.10. The properties of the asphaltic concrete, deformation measurements and results of computations are listed in Table 7.11. The measurements and the investigations of Breth & Arslan lead to the following conclusions:

Table 7.11. Examples of embankment dams with asphaltic concrete core. Characteristic data of asphaltic concrete. References as in Table 7.10.

Project		Grosse Dhünn H = 58 m	Finstertal H = 96 m	Megget H = 56 m
Dimensions of asphaltic concrete				
Width of the toe	(m)		0.7	0.9
Standard width	(m)	0.6	0.7/0.6	0.7
Width of the crest	(m)		0.5	0.6
Area	(m^2)	12,900	37,000	20,000
Volume	(m^3)	7,800	31,000	13,500
Layer thickness	(cm)	20	25	20
Properties of asphaltic concrete				
Aggregates			Granodiorite	Basalt
d_{max}	(mm)		16	20
Crushed gravel/sand	(% by weight)		81.7	62.4
Natural sand	(% by weight)		4	20.5
Lime filler	(% by weight)		8	10.3
Bitumen	(% by weight)		6.3	6.8
Fill temperature	(°C)		160 to 200	≥ 160
Pore volume	(%)		1.0 to 2.0	0.4 to 2.9
Dry unit weight	(kN/m^3)			24.9
Deformations of asphaltic concrete				
Maximum settlement	(cm)	45 (30) 0.008 H (at 0.5 H)	30 (30) 0.003 H (at 0.5 H)	9.5 (10.9) 0.002 H (at 0.33 H)
Max. horizontal displacement:				
– End of construction	(cm)		–5 (6) (lower part)	–7 (center part)
– Full supply level	(cm)	+5 (4 to 5) (top)	+14 (10 to 11) (top)	+2.3 (4.6) (top)
Stress and strain of asphaltic concrete				
Axial stress	(MPa)	(–0.8)	(–0.9)	(–0.53)
Transverse stress	(MPa)	(–0.4)	(–0.44)	(–0.2)
Shear stress	(MPa)	(+0.04)	(+0.22)	
Axial strain		(18.4×10^{-3})	(8.6×10^{-3})	
Transverse strain		(4×10^{-3})	(5.2×10^{-3})	
Shear strain		(1.5×10^{-3})	(12.8×10^{-3})	
Seepage	(l/s)	0.1	3	1.6

– = Displacement towards u/s, pressure
+ = Displacement towards d/s, tension
() = Result of computations

– The vertical and the inclined membranes hang between the neighbouring shells. This follows from the computed normal stresses (Fig. 7.45). The deformation moduli of the shells and the transition zones may be up to ten times those of the membranes. Assumed properties of the natural materials are given in Table 4.10.

– Due to arching, the normal stress on top of the gallery of Grosse Dhünn was reduced to 70% of the calculated stress due to the weight of the membrane. The reduced stress is sufficient to prevent hydraulic fracturing. Finstertal revealed an even greater, but still acceptable, deficit of vertical stresses (see below).

– The lines of computed equal horizontal displacements in Figure 7.46 demonstrate that the vertical membrane can be located in a zone of minimum displacements. The shear deformations of the vertical membrane will then undergo those of an inclined membrane.

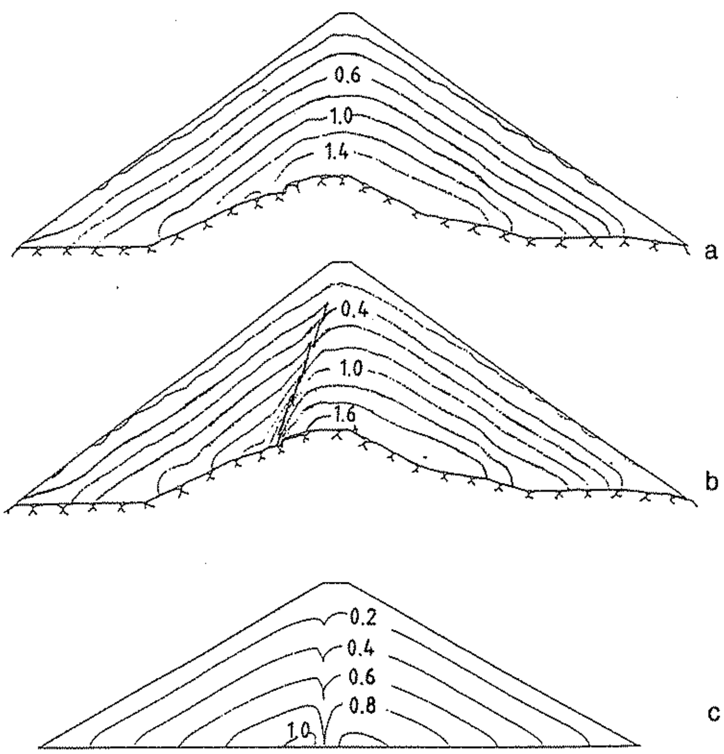


Figure 7.45. Rockfill dams with artificial internal sealing. Lines of computed equal normal stresses (MPa) at end of construction (Breth & Arslan 1990).

a Finstertal without internal sealing
b Finstertal with internal sealing
c Grosse Dhünn with internal sealing

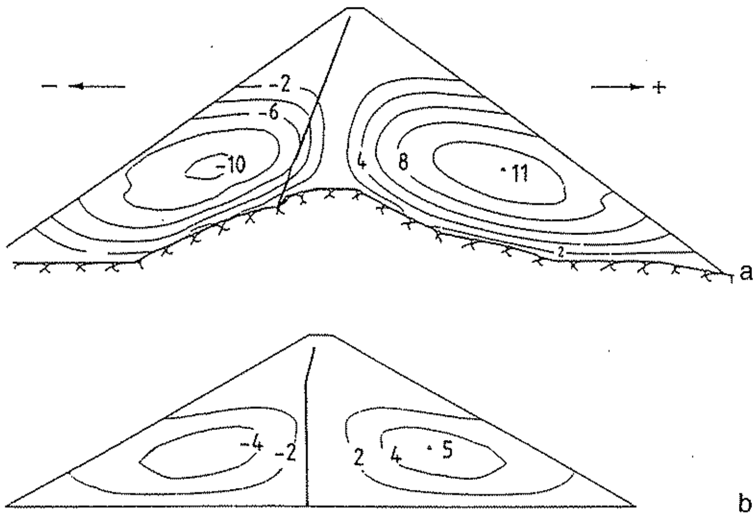


Figure 7.46. Rockfill dams with artificial internal sealing. Lines of computed equal horizontal displacements (cm) at end of construction (Breth & Arslan 1990).

a Finstertal
b Grosse Dhünn

– Figure 7.47 shows the measured horizontal displacements of the Finstertal membrane, after the end of construction and after reservoir impounding and drawdown. The maximum is about 14 cm or 0.15% of the vertical height of the membrane. The measured maximum horizontal displacement of the Grosse Dhünn membrane was only 4 cm or 0.07% of the height. Table 7.11 shows the computed stresses and strains of the two membranes. It is evident that the shear deformation and related strain of the inclined membrane is greater than that of the vertical membrane.

– Figure 7.47 shows the approximate parallel locations of the upper section of the Finstertal membrane given different load conditions. It appears that the inclination of the top section fits well with the deformation conditions. This confirms the common practice of distorting the top section as shown in Figure 7.46b. Usually, the inclination is about 1V:0.25H. This shape helps to prevent cracks due to settlement between the membrane and the dam shell. Such cracks would run parallel to the center line of the dam.

The axial and transverse strains of the three membranes are well within permissible limits. The quantities of seepage confirm the excellent performance of the membranes.

The topography of the dam location may favour an inclined membrane though the stresses and strains are less favourable than with a vertical one. In the case of Finstertal, Pircher & Schwab (1988) comment as follows:

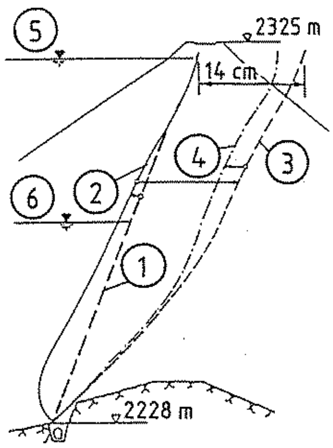


Figure 7.47. Finstertal rockfill dam. Measured horizontal displacements of the internal sealing (adapted from Breth & Arslan 1990).

1 Location of the sealing as designed
2 Location of the sealing at end of construction
3 Location of the sealing after impounding to full supply level
4 Location of the sealing after drawdown
5 Full supply level
6 Drawdown level

'...The peculiar topography of the site causes the dam to straddle the rock sill which bars the glacial trough of the reservoir basin... An optimum position in respect of embankment volume and stability was achieved with a slight upstream curvature of the main body and a short reverse curve at the right wing... With the location of the gallery slightly upstream of the former lake's outlet sill, the surface area of the 96 m high core is kept to a minimum of 37,000 m^2... The inclination, which in the upper reach coincides favourably with the direction of the deformation vectors, allocates a larger portion of the dam volume to the downstream shoulder, gives the resultant hydrostatic load a favourable downward direction, and thus makes a very steep downstream slope possible, with inherent savings in embankment volume. Furthermore, the core membrane rests with a part of its weight on the downstream shoulder, and likewise the upstream shoulder rests on the core, which is kept effectively constrained and thus has only a very limited possibility of developing any undesirable deformation behaviour...'.

The different deformations, stresses and strains of the three examples of dams follow from the particular circumstances (Tables 7.10 and 7.11):

– The maximum settlements as a percentage of the dam height correspond to the deformability of the shells. According to experience, the well compacted sandy gravel of Megget will be less deformable than the well compacted rockfill material of Finstertal. The minimum modulus of deformation – and hence the maximum settlement – will be attributed to the greywacke and siltstone of Grosse Dhünn which is classified as 'friable'.

– The axial and transverse strains reflect the different dam heights and the different degree of arching. Arching means stress transfer from the membrane to the shells. The axial stress of the Grosse Dhünn membrane corresponds to about 70% of the dead weight of the membrane. The stresses of the Finstertal and Megget membranes correspond to only about 50% of the respective dead weights. This reflects – as expected – the tendency towards arching: it is more pronounced with the stiff shells of Finstertal and Megget than with the shell of 'friable' rock of Grosse Dhünn.

The deformation behaviour of asphaltic concrete is controlled by its composition and the selected type of bitumen. The two must be adjusted to the individual conditions of each project. The composition is discussed in Section 9.2.6, examples are given in Table 7.11.

The following relations have been found by Breth & Schwab (1979) as the result of triaxial tests: friction and cohesion of asphaltic concrete decrease with increasing quantity of bitumen. The same tendency is to be expected from the ratio filler/bitumen. The viscosity of the bitumen does not significantly affect the friction angle, provided the bitumen content does not exceed 6% by weight. The cohesion decreases with the viscosity of the bitumen.

Other tests made by Breth & Schwab (1973) allow us to decide on the suitability of asphaltic concrete under the conditions of narrow V-shaped valleys and of earthquakes. The tests served to investigate the behaviour of asphaltic concrete under shear deformations. At the abutments of a dam in a V-shaped valley great differential settlements must be expected. Asphaltic concrete is deemed suitable since permissible strains are about 14% axial and 6% transverse. An example of great deformations is the Eberlaste dam which is discussed in Section 7.4.4.1.

The shear deformation related to earthquakes was simulated in cyclic triaxial tests. Samples of asphaltic concrete have been confined and consolidated by stresses $\sigma_3/\sigma_1 = 0.25$. This state of stress was superimposed by a cyclic shear stress of 0.2 MPa at a frequency of 3.5 Hertz. This cyclic load corresponds approximately to the load of the El Centro earthquake of 1940 in California, acting on a 100 m high rockfill dam with internal sealing of asphaltic concrete. A total of 200 load cycles did not change the structure and the strength properties of the samples. Breth & Schwab comment: 'The deformation remained linear with regard to the applied shear forces and decreased to zero with each release of the shear force. The asphaltic concrete behaved like an elastic body even with the conditions applied being very unfavourable.' Further data on long-term load tests are given in Section 7.4.2.2e.

7.4.2.2 *Face sealing of asphaltic concrete*

Asphaltic concrete as a sealing element for embankment dams has first been applied as face sealing. The thin membrane consists of one or more layers which must be flexible enough to follow all deformations of its base. The support conditions are different on the slope, at the dam toe and at the abutments. Therefore, the sealing must be flexible in the long term. This flexibility may conflict with stability against creeping on the slope. A reasonable compromise must be found between the deformation properties of the asphaltic concrete as an elasto-plastic material and the slope angle of the base.

Typical systems of asphaltic concrete faces are shown in Figure 7.48. We distinguish controlled and uncontrolled faces. The controlled face is a sandwich-like structure of two units of asphaltic concrete with an intermediate drain layer which allows control of the seepage quantity, e.g. in an inspection gallery. The uncontrolled face is only one unit of asphaltic concrete which consists of one or more individual layers with staggered joints. Both types of face rest on a bituminous binder course, which is the transition to the dam body. The surface of the asphaltic faces is seal coated. Data for both types of faces are listed in Table 7.12.

The uncontrolled face offers the advantages of more rapid and less costly construction. There is a tendency towards this type of face, which benefits from modern equipment capable of constructing thick layers.

The forces acting on a surface membrane are different from those acting on an internal membrane. Triaxial stress conditions and related deformations are marginal. In the direction down the slope the face sealing is subjected mainly to bending. Shear stresses occur along the line of contact with a rigid structure, such as a gallery. Tensile stresses may occur at the abutments and

Table 7.12. Typical composition of asphaltic concrete for face sealings (adapted from ICOLD 1982).

Layer of the face		Asphaltic concrete	Drainage layer	Binder course	Seal coat Mastic	Sandmastic
Bitumen	(% by weight)	7 to 9	3 to 5	3 to 6	≤ 50	≤ 50
Filler < 0.074	(% by weight)	10 to 15	2 to 5	2 to 10	40 to 60	
Sand < 4.57 mm	(% by weight)	50 to 80	18 to 30	30 to 60	–	17 to 20
Max. grain size	(mm)	10 to 20	20 to 30	12 to 25	≈ 0.1	≈ 0.5
Dry unit weight	(kN/m^3)	21.4 to 25.5	17 to 25	22 to 27		
Pore volume	(%)	1 to 3	8 to 30 20 to 30[1]	5 to 10 8 to 12[1]		
Permeability	(m/s)	$\leq 10^{-9}$	$\leq 10^{-4}$	10^{-4} to 10^{-7}		

[1]German standard

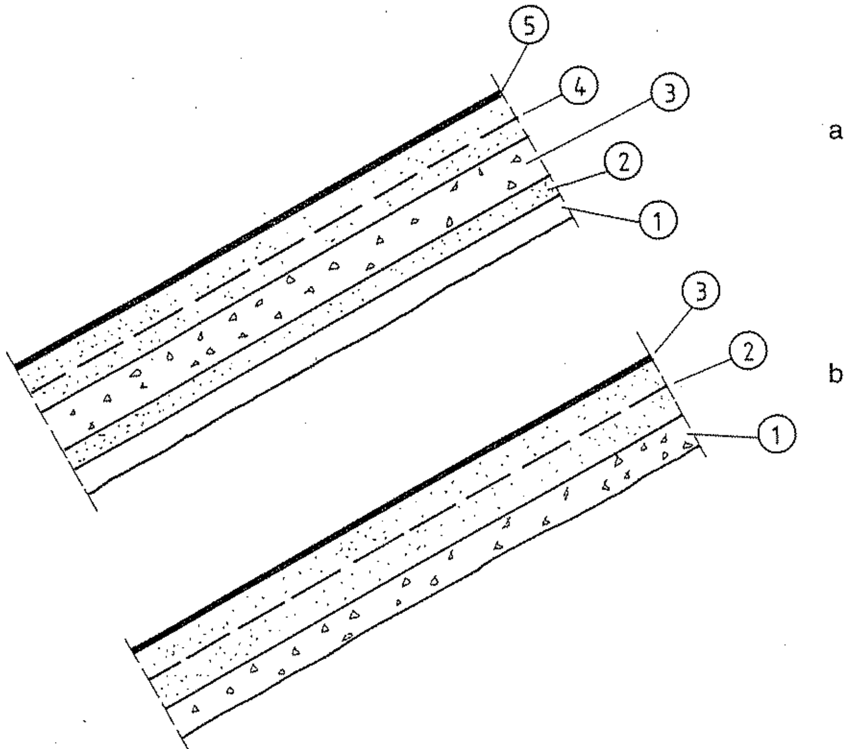


Figure 7.48. Face sealings of asphaltic concrete, typical design.

a Controlled face sealing
(with drainage, ICOLD 1982: Type A)
1 Bituminous binder course, 5 to 15 cm
2 Asphaltic concrete, 3 to 8 cm
3 Bituminous drainage layer, 5 to 15 cm
4 Asphaltic concrete, one or more layers, 5 to 12 cm
5 Bituminous seal coat, 0.2 to 0.4 cm

b Uncontrolled face sealing
(without drainage, ICOLD 1982: Type B)
1 Bituminous binder course, 5 to 15 cm
2 Asphaltic concrete, one or more layers, 6 to 12 cm
3 Bituminous seal coat, 0.2 to 0.4 cm

in the outer sections as shown in Figure 7.49. The properties of the face must fit with these circumstances. The properties are:

a) Impermeability at hydraulic gradients up to $i = 1000$ (height 100 m, length 0.1 m).

b) Resistance to ice, waves, solid matters (stones etc.).

c) Flexibility without the development of cracks due to deformations of the base.

d) Stability against creeping.

e) Durability under all weather conditions.

These requirements can be fulfilled provided the materials of the asphaltic concrete and of the dam, and the method of placing, are properly selected.This is confirmed by more than 100 existing dams up to a height of 100 m and 75 smaller reservoirs which have been listed by ICOLD (1982).

a) The permeability should be less than that of internal sealings, because of the high gradients. Permeabilities $\leq 10^{-9}$ m/s have been realized. Related quantities of seepage are insignificant with respect to the dam's stability.

The following is noted: the quantity of seepage is related to the tightness of the whole sealing system, which depends on joints at the contact with other structures, on cracks and leaks. Asphaltic concrete as a material should be classified 'impervious' or 'pervious'. Irrespective of this the nominal permeability of the material is mentioned in literature and is checked on site as a quality control. Leaks open and close due to temperature and water head. Open joints up to 5 mm width have been observed (e.g. Hasegawa & Kikusawa 1988). In this extreme case the seepage quantity was reduced to acceptable rates by self-sealing after a few years of operation.

In the case of concentrated leaks and large quantities of seepage, the dam body must be stable. Seepage must be drained without any risk of instability. Using a model dam, 3.2 m in height, Brauns et al. (1988) have investigated the relation of seepage and the size and location of leaks in terms of a 'degree of damage'. The results are given in Table 7.13. The results do not directly reflect the conditions of a large dam, but the tests are useful to confirm the well known fact that very small leaks in a membrane will cause great quantities of seepage. This is the background of the German standard to consider the dam's stability under the assumption 'no sealing effective'.

b) The required resistance to mechanical attack usually does not create problems. Ice-floes constitute the most aggressive attack. Experiences are available from the mountainous regions of Austria and Switzerland. The seal coat helps to protect asphaltic concrete faces. Tschernutter (1988) reports on striation produced by the combined action of stones and blocks of ice originating from a glacier drifting to the face, the stones being deposited by avalanches. Stones and ice slide down and carve grooves up to several centime-

Table 7.13. Seepage quantity Q/Q_0 through dam with face sealing as a function of 'degree of damage' (produced after Brauns et al. 1988).

'Degree of damage' F/F_0	0.001	0.01
Open leaks at high level	0.7	0.9
Open leaks at low level	0.4	0.8
Open leaks at high and low levels	0.7	0.9

F_0 = Area of face sealing
F = Area of open leaks
Q_0 = Seepage quantity through dam without sealing
Q = Seepage quantity through dam with sealing and open leaks

ters in depth into the face. Damage has been repaired by placing a new 7 to 8 cm thick asphaltic concrete layer, after 1 or 2 cm of the existing face had been milled off. A good bond between the old and the new materials was produced by cleaning the milled surface with compressed air and spraying it with bitumen.

Earthquake action may be seen as a particular mechanical attack. Such action is not a risk for the asphaltic concrete face provided the dam is stable and no extraordinary deformations develop on the slope. Measurements at the 75 m high Miyama dam indicate that the face does not develop its own motion (Hasegawa & Kikusawa 1988). The following is noted: for this type of dam – 'homogeneous' rockfill dam with face sealing – the amplitude magnification from the ground to the crest is in the order of 3 to 4, while it is 1.5 to 2 for rockfill dams with an earth core. The water load on the face has a stabilizing effect on the upstream slope. This follows from the measured responses of the upstream and downstream slopes.

c) Usually, the required flexibility is achieved provided the deformation properties of the asphaltic concrete are within the limits discussed in Section 7.4.2.1. Interfaces of asphaltic concrete and other structures must be designed so as to exclude unacceptable differential settlements. The permeability of deformed asphaltic concrete is related to its flexibility. It can be checked by a test using a disc of asphaltic concrete which is fixed at the periphery. The center portion is loaded with water. The deformation depends on the speed of loading, the temperature, the viscosity and the thickness of the disc. The deformed disc will attain a shape similar to a soup-plate. According to DGEG (1983) the material will remain sufficiently tight if the depth of the plate does not exceed 1/10 of the diameter. The strain is approximately 2%.

A similar test is described by Ishii & Kamijo (1988). A disc, 50 cm in diameter and 5 cm in thickness, was deformed to a soup-plate. The initial permeability was not changed when the depth of the plate was almost 1/6 of the diameter. The related strains are approximately 20% at the periphery and 10% in the center. The tests were made at temperatures of 5° and 20°, and the water load was almost 0.9 MPa.

Hasegawa & Kikusawa (1988) and Daicho (1988) report on deformations of asphaltic concrete with the Miyama and Tataragi dams, respectively. In both cases the critical interface of the asphaltic concrete face and the gallery was studied. In both cases no damage to the face was observed. Bending of the Miyama face corresponds to the depth of a plate of about 1/130 of the diameter. A depth of 1/30 is permissible. At Tataragi the observed maximum strain was about 1%. It is known from bending and tension tests that 3% strain is a lower limit to develop cracks in the asphaltic concrete.

d) Creeping of asphaltic concrete along the base must be prevented by the shear strength of the materials and by the shear resistance in the contact area. Tests result in a friction angle of asphaltic concrete of 35° and a cohesion of

250 kPa (Breth & Schwab 1973, 1979). Le Coroller et al. (1988) report on a shear strength of 400 to 600 kPa of readily consolidated asphaltic concrete. That means the consolidated material does not creep along slopes that are relevant for embankment dams.

However, creeping of asphaltic concrete may be critical when placed at temperatures up to 200°. The slopes of existing dams with an asphaltic face are about 1V:1.5H or flatter. After ICOLD (1982) 'it could be said that the maximum slope limit compatible with the construction of this type of facing is 1V:1.5H, which is the limit for the stability of the hot mix on the inclined plane, before and after compaction, and for a safe foothold for the workers, without provision of special devices.' According to Haas et al. (1988) engineers are now able to construct asphaltic concrete faces on slopes of 1V:1.2H. This is because of the capacity of modern placing and compaction equipment. The material is compacted to extreme density. The number and size of point-to-point contacts between neighbouring particles of the aggregates and hence the shear strength of the material are increased by such extreme compaction.

e) The durability of asphaltic concrete has been evaluated since about 1955. Since then the placing technique and knowledge about the material have considerably improved. Therefore, examples are rare for asphaltic concrete faces produced according to the present state of the art.

The layer thickness was increased initially from 3 cm to 12 cm now and more. This favours the tendency towards uncontrolled faces of only one impervious bituminous layer above the binder course. Units of more than one layer include the risk that bubbles and blisters remain between the layers. Such bubbles and blisters develop from trapped air and water. An insufficient bond between the layers is also a typical defect. Tschernutter (1988) and a Group of the Swiss National Committee on Large Dams (1988) reported on defects in asphaltic concrete faces constructed in mountainous regions of Austria and Switzerland where the structures are subjected to great differences in temperature and to daily fluctuations of the water level. Bubbles and blisters lead to damage of the upper layer and subsequent increased attack on the lower layer by weathering. Cracks may develop in the upper layer if it cools down too quickly due to the weather conditions. One case is reported where the upper layer slipped down due to inadequate placing methods and due to contamination of the contact area by oil.

As to modern systems of one thick layer, these authors report on repair work. The examples refer to the effect of temperature changes, to cracks at the contact with rigid structures and to inadequate placing methods. They do not reflect the process of ageing. The Swiss engineers conclude 'that single-layer linings with greater layer thickness, in combination with improved equipment, have been very successful.'

No preference can be attributed to one of the systems, either one layer or more than one, since the boundary conditions of the projects in question are different. Too many parameters have to be taken into account. With the multi-layer system the joints are scattered. There are no joints cutting the face from top to bottom, as it is the case with the single-layer system. For this reason many engineers prefer the multi-layer system in earthquake endangered regions. The risk of open joints after movements is minimized. In contrast, systems of more than one layer bear the risk of blisters from trapped water where the face has to be constructed in rainy weather conditions.

The question of durability was carefully studied by Fabian & Ditter (1988), by investigating the asphaltic concrete faces of the Geesthacht and the Eggberg reservoirs in Germany, which had been operated since 1957 and 1967, i.e. 30 and 20 years, respectively. Both faces consist of two layers, 3.5 + 3.5 cm (Geesthacht) and 6 + 4 cm (Eggberg), in thickness. In Geesthacht the following was found with reference to different zones:

– The zone of the face which is permanently under water. No change in the quality of the asphaltic concrete was observed, and no repair and maintenance work had to be executed.

– The zone between full supply and minimum operating water levels. After a few years of operation the joints of the thin upper layer (3.5 cm) began to open. This is caused by extended bending of the face with each filling. Open joints have been welded or filled with a special joint filler. Bubbles and blisters appeared between the two layers due to trapped air and water. The affected asphaltic concrete was removed and replaced by fresh material.

– The zone above full supply level. This zone is subjected to maximum temperature changes. Open joints have been detected, after a longer period of operation, starting in places most affected by the sun's rays. Cracks developed in the asphaltic concrete, which indicate a process of fatigue. The upper layer was removed and replaced by new material 8 cm thick.

At Eggberg the investigations were concentrated on the zone above full supply level and the places of maximum effect of sun rays. 'The mastic asphalt seal coat in that area had already completely disappeared because of weathering....In order to prevent the face from becoming brittle, the seal coat has to be renewed from time to time'. In principle, the seal coat should be seen as a wear and tear membrane suffering from solar radiation and oxygen in the air. It protects the asphaltic concrete from the direct effects of these agents as the two main factors of ageing.

Two more aspects of durability have been approached by freezing and thawing tests and by cyclic bending tests (Ishii & Kamijo 1988). The tests were initiated in the design phase of a 90 m high rockfill dam with asphaltic concrete face in an earthquake endangered region at 1000 m above sea level.

The face is designed as a controlled system with a seal coat, the upper unit consisting of 3 layers, 5 cm each in thickness.

The freezing and thawing tests did not indicate any change in the physical properties of the asphaltic concrete. The density, the permeability and the failure stresses at tension, compression and shear remained constant after at least 100 freezing and thawing actions in the range of +20° and –20°.

The cyclic bending tests indicate that the selected asphaltic concrete face will be stable over more than 200 years of operation with an anticipated fluctuation once a day. It will resist the expected maximum amplitude of earthquake motion for more than 1500 events. The expected motion corresponds to the El Centro earthquake of 1940. The 'yield times' of 200 years and 1500 events are related to the test result, showing that the deformation modulus of the asphaltic concrete decreases rapidly after a certain, but large, number of load cycles.

7.4.2.3 *Face sealing of conventional concrete*

The use of conventional concrete as a surface sealing leads to the well known concrete face rockfill dam (CFRD) as shown in Figure 5.8. The concrete face consists of parallel slabs, 6 to 14 m in width. The slabs are placed within slipforms. The width is related to practical considerations of how to maneuver the slipform and the reinforcement, and to the conditions of concrete supply and distribution.

The concrete is subjected mainly to compression (Fig. 7.49). At the outer zones the joints between the slabs are supposed to open due to tensile stresses. Tension may also occur at the perimetric joint. With regard to tension and to temperature and shrinkage stresses the concrete is reinforced. The reinforcement is about 0.5% of the concrete section in both directions, or 0.5% vertical and 0.4% horizontal. The concrete section refers to the slab thickness at the toe. Related reinforcement is kept constant for the whole slab, irrespective of its thickness at the crest. A surplus of reinforcement is recommended at the perimetric joint to compensate for differential settlements of the base due to irregularities of the foundation.

Cracks in the slabs were observed to be horizontal, probably due to thermal shrinkage (Fitzpatrick et al. 1985). They have been found mainly in long slabs. The width usually does not exceed 0.2 mm. Cracks of more than 0.5 mm in width have been sealed by painting with a rubber-based compound. Smaller cracks are supposed to be insignificant or to be self-sealed with time.

The evolution of the face thickness t leads to the usual design of

$$t = 300 + 0.001h \text{ (mm)} \tag{7.11}$$

This leads to a thickness of 420 mm for a dam 120 m in height. In Australia

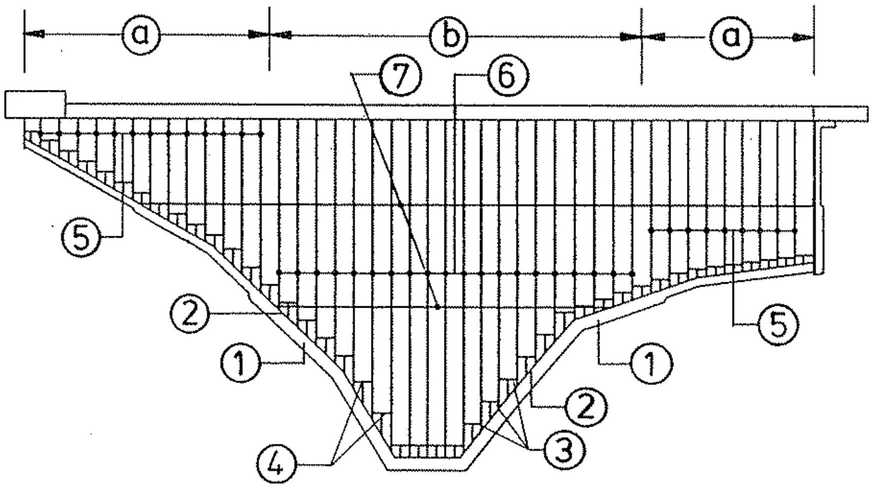


Figure 7.49. Concrete face sealing. Joint layout (example Bakun 1996, courtesy of LI).

1 Plinth
2 Perimetric joint
3 Starter slabs
4 Horizontal contraction joint, type 1
5 Vertical contraction joint, type 1
6 Vertical compression joint, type 2
7 Horizontal construction joint, location according to the work progress
a Zones of tension
b Zone of compression

slabs have been constructed 250 mm thick for dams up to 75 m in height (instead of 375 mm as per Eq. 7.11).

The nature of conventional concrete and construction methods results in the permanent existence of joints (Fig. 7.49):

– Vertical contraction joints between neighbouring slabs,
– vertical compression joints between neighbouring slabs,
– horizontal contraction joints between starter and standard slabs,
– horizontal construction joints between the lower and the upper parts of each slab, and
– the perimetric joint between the starter slabs and the plinth.

The typical design of vertical and horizontal joints is shown in Figure 7.50. The design of the contraction joints (Fig. 7.50a) follows a triple line of defense, namely mastic filler (2), PWC waterstop (4) and copper waterstop (5). It is noted that no tear protection of the PVC waterstop is used since differential settlements are not expected to occur, as is the case with the perimetric joint (Fig. 7.62). This critical joint is discussed in Section 7.5.3. A similar design of contraction joints was developed for the Salvajina dam (Sierra et al. 1985) and for several other dams, where it worked successfully. The vertical compression joint (Fig. 7.50b) shows a rather simple design with a copper waterstop at the bottom. According to Sierra et al. 'it has been

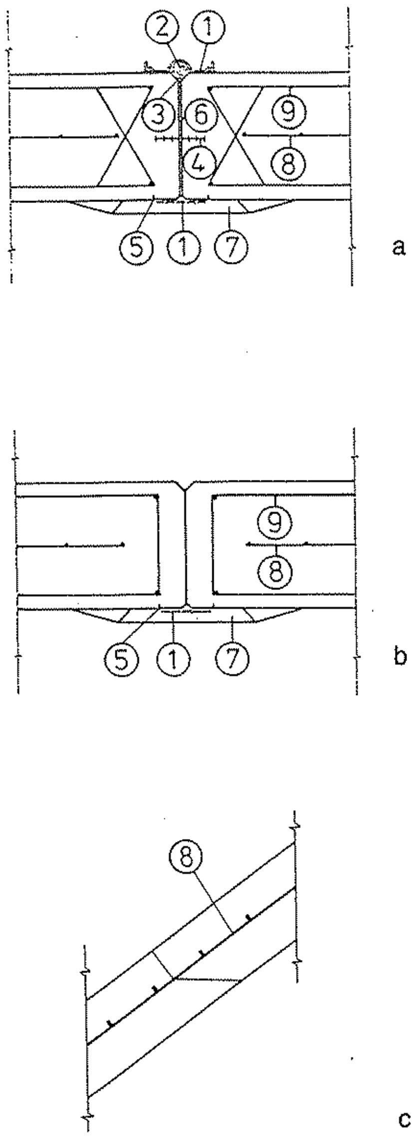


Figure 7.50. Concrete face sealing. Joint details (example Bakun 1996, courtesy of LI).

a Horizontal and vertical contraction joints type 1
b Vertical compression joint type 2
c Horizontal construction joint
1 PVC band
2 Mastic filler
3 Neoprene tube
4 PVC waterstop
5 Copper waterstop
6 Asphalt paint
7 Mortar pad
8 Reinforcement, 0.3% each direction
9 Reinforcement to protect concrete from spalling

repeatedly proved by experience that the joints tend to close under a moderate compression'.

The design of construction joints with the reinforcement going through is shown in Figure 7.50c. No waterstop is used. The joint in the concrete is normal to the surface in the upper part and horizontal in the lower part. This is favourable with respect to concrete cleaning prior to continuation of the work.

In principle, conventional concrete appears to be less flexible than asphaltic concrete. The existence of different types of joints and the cracking behaviour of concrete may cause concern about the tightness of concrete faces as a system. Information on that can be taken from measurements of existing dams. Reports have been edited by Cooke & Sherard (1985). Some data are listed in Table 7.14. From this and from the reports the following is concluded:

– The displacements of the slabs are not excessive. Given excellent workmanship, such displacements do not lead to critical cracks. This follows from the great displacement of Foz do Areia and the non-critical seepage there.

– Joint openings or defects appear to be the main reason for concern. For Foz do Areia openings are reported to be up to 30 mm due to tension. Opening of the perimetric joint was measured up to 25 mm. In this case such conditions did not lead to excessive seepage or need repair.

– With a number of dams the quantity of seepage is higher than is usual with dams with earth core or asphaltic sealing. This is seen as insignificant by those who favour concrete faces, since the stability of the dams is not endangered. Only the reservoir operation might be affected, but this is not critical for large reservoirs.

– The number of necessary repairs after first impounding is noted. It can be seen from the reports that such repairs originate in part from mistakes in construction. Some of them are such mistakes which have to be made before being noticed and verified as mistakes. Supposedly the number of such mistakes will decrease as experience of concrete face rockfill dams grows.

The durability of concrete faces and the stability of concrete face dams given earthquake conditions are valuable properties of such structures. These properties are confirmed by the report of Arrau et al. (1985) on the dam Cogoti. This 85 m high dam was constructed in Chile in 1938. It experienced four major earthquakes with ground accelerations up to 0.19 g. 'No earthquake damage to the face slab has occurred. Substantial settlement did occur, but the dam is considered effective in withstanding seismic loadings'.

7.4.3 *Filters and transition zones*

The function of transition zones in dams with artificial sealing is somewhat

Table 7.14. Concrete face rockfill dams. Examples of face displacements and seepage (produced after Cooke & Sherard 1985).

Project	Country	Year	Height (m)	Maximum face displacement normal to face (mm)	(Time)	Seepage (l/s)	Remarks
Fades	France	1968	68	150	After impounding	60	Including abutments and precipitation
						20	Probable seepage through dam with tendency towards reduction
Kangaroo Creek	Australia	1968	59	65	6 years after end of construction	1	After 13 years of operation
Little Para	Australia	1977	53	64	As above	19	Right abutment
Alto Anchicaya	Columbia	1974	140	160	2.5 years after 1st filling	1800	After 1st impounding
						180	After 2 repairs at the perimetric joint
Golillas	Columbia	1978	125	160	During 1st impounding, estimation	1080	After 1st impounding
						650	After repairs, mainly perimeter wall
Winnecke	Australia	1980	85	< 200	After impounding	32	After cement dosing treatment
Bailey	Virginia/ USA	1979	95	150	Failure, mainly of the perimetric joint	370	Water head ≈ 47 m
						60	After repair, water head ≈ 34 m
Foz do Areia	Brazil	1980	160	775	4 years after impounding	240	1st impounding
						70	4 years water head ≈ 150 m
Mangrove Creek	Australia	1981	80	12	During impounding	4	Water head ≈ 33 m
Shiroro	Nigeria	1983	125	> 50	Perimetric joint	1500	After impounding
						100	After pumping of silty sand into cracks
Fortuna	Panama	1982	60	40	After impounding	1	
Khao Laem	Thailand	1984	130	125	During impounding		
Terror Lake	Alaska/ USA	1984	59			14	Water head ≈ 33 m
Batang Ai	Sarawak/ Malaysia	1985	85	–	No displacement recorded	30	Water head ≈ 60 m

different from their function in dams with natural sealing. Concrete and asphaltic concrete are not erodible, so the transition zones are not a protection against erosion. The zones must support the equipment constructing the artificial sealing. Therefore, they must be well compacted to achieve sufficient bearing capacity.

7.4.3.1 *Internal sealing of asphaltic concrete*

The width of transition zones upstream and downstream of internal sealings is related to the dam height. The zones are up to 3.0 m wide on both sides. They consist of natural or crushed sand and gravel. The particles must be strong and durable. The material must be well graded, and not gap-graded. This results in satisfactory compactability and bearing capacity. The compact state contrasts with the desired transitional function. This is confirmed by the deficit of normal stresses due to arching as was measured in some dams (compare Section 7.4.2.1).

ICOLD (1992) recommends at least 10% (by weight) of the transition material to be smaller than the maximum grain size of the aggregates of the asphaltic concrete, and the maximum grain size of the transition material to be at least 1/4 of the maximum grain size of the shell material. Fines may be added to the transition material upstream to create self-sealing of leakages. The fines will also help to reduce seepage through a concentrated leak.

In the downstream transition zone, seeping water is conducted vertically to the bottom of the sealing and further into a gallery or to the downstream dam toe. It is recommended to construct little separating walls to enable registration of the seepage quantities by section (e.g. Fig. 7.58). It is normal practice to have horizontal conduits of asphaltic concrete (half shells) at different elevations which enable the location of concentrated leaks.

7.4.3.2 *Face sealing of asphaltic concrete*

The bituminous binder course acts as the transition between asphaltic face sealings and the dam body. The transitional function refers to permeability. It is 3 to 4 powers of ten greater than the permeability of the face and 3 to 4 powers of ten lower than the permeability of the drain or base layer underneath. The binder course is a component of controlled and of uncontrolled systems (Fig. 7.48). The course is about 5 to 15 cm thick. The mineral skeleton consists of natural or crushed material. The great permeability – in comparison to the asphaltic face – is caused by the low content of bitumen and fines.

Underneath the binder course is a drain or base layer. The drainage function is effective only if the shell material is less permeable. An example is the Prims dam in Germany, consisting of clay slate (Section 4.3.3). Water seeping across the face is in this layer conducted to the gallery at the toe. If the shell material is a free-draining rockfill the transitional function of the base layer refers only to the permeability and the gradation. The grain sizes are about 1 to 100 mm. The layer is usually 10 to 50 cm thick.

7.4.3.3 *Face sealing of conventional concrete*

The transition zone underneath faces of conventional concrete consists of two individual layers which are graded according to common filter rules. The outer layer is a well graded sand-gravel of low permeability with a maximum grain size of about 75 mm and 35 to 55% below 5 mm. The gradation guarantees good compactability and hence high deformation modulus. The low permeability is believed to reduce seepage through leaks. The gradation shall not allow silt particles to pass. Silt may be used to clog leaks and defective joints of the facing. According to Kulesza (1988) there are not many examples to show that the desired low permeability in the order of 10^{-6} to 10^{-5} m/s is achieved on site. A silt content of about 10% and regular distribution of the fines are preconditions for low permeability.

The layer is placed in about 0.5 m lifts. The horizontal width is in the order of 1.0 m. After compaction the surface of the outer layer is stabilized by spraying a bituminous emulsion or by shotcrete (Fig. 9.39). This is designed to protect the layer from any damage by further works and from erosion by rain.

The inner part of the transition zone prevents the outer layer from being washed into the rockfill material of the dam body. The grading must be done accordingly. The layer is up to 5.0 m wide (measured horizontal). It is placed in 0.5 to 0.8 m lifts and conventionally compacted.

7.4.4 *Foundation of artificial sealings*

All artificial sealings rest on or end in a concrete structure at the toe. It is a reinforced concrete slab or a strong beam with or without a gallery. It is the element which connects the sealing and the foundation. The forces of the sealing acting on this structure are of minor importance. The structure is affected more by the weight of the dam body, which is located above or beneath, and by the full hydrostatic pressure of the stored water. These forces may cause deformations of the foundation. The toe structure must be able to follow these deformations without separating from the sealing element.

7.4.4.1 *Foundation of internal sealings*

Rock foundation

The toe structure rests on practically incompressible rock. An example is shown in Figure 7.58. Existing defects of the foundation rock and defects caused by excavation must be repaired. This is usually made by contact grouting as shown in Figure 7.40. This grouting is also required to block or to lengthen the critical short seepage path from upstream to downstream around the toe structure.

It is an advantage of internal sealings over face sealings that the toe structure follows the shortest line in plan from one end of the dam to the other. Discontinuities in the foundation and related deformations must be carefully considered. The location of the toe structure of vertical and – within certain limits – of steeply inclined sealing membranes can be adjusted to great differences in elevation of the rock surface, without changing the geometry of the membrane. Depressions are backfilled with concrete. This results in a regular and smooth alignment of the interface of the toe structure and the membrane.

Compressible foundation
Reasons of economics may cause the designer to accept a compressible foundation. An example is the Eberlaste dam. The internal membrane of asphaltic concrete rests on a toe structure which links the membrane with the diaphragm of plastic concrete in the foundation. The foundation conditions and the typical section of the dam are shown in Figures 7.51 and 7.52. At the abutments the rock surface is steeply inclined. In the valley, bedrock was not encountered after drilling 124 m deep. The valley is filled up by very heterogeneous sandy and gravelly river sediments with a 20 m thick layer of sand and silt on top. The steep slopes are covered by slope debris with boulders. The permeability of the foundation is reported to be in the order of 10^{-7} to 10^{-4} m/s (Breth & Günther 1967).

The internal membrane of asphaltic concrete was selected for economic reasons. With respect to the necessary flexibility, the material is composed as follows:

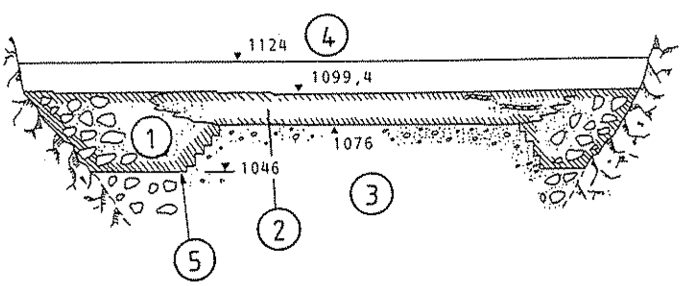


Figure 7.51. Eberlaste earth dam. Foundation conditions (adapted from Müller-Salzburg 1992).

1 Slope debris with boulders
2 Sand and silt
3 Sandy, gravelly river sediments
4 Dam crest
5 Bottom of the diaphragm
(Elevations m a.s.l.)

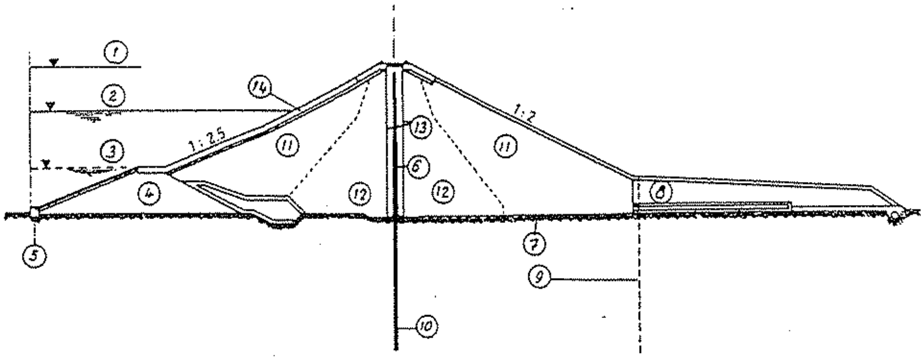


Figure 7.52. Eberlaste earth dam. Typical section (adapted from Kropatschek & Rienössl 1970).

1 Crest elevation 1124 m a.s.l.
2 Full supply level 1116 m a.s.l.
3 Min. operating level 1106 m a.s.l.
4 Cofferdam
5 Sheet piling
6 Asphaltic concrete sealing, width 40 and 50 cm
7 Bituminous gravel layer, 8 cm
8 Stabilizing berm
9 Relief holes, depth 60 m
10 Plastic concrete diaphragm, width 60 cm, depth 21 to 52 m
11 Slope debris
12 Slope debris, selected, ≤ 200 mm
13 Transition zones
14 Wave protective layer

Aggregates by weight:
– 30% gravel 10 to 25 mm,
– 30% gravel 3 to 10 mm,
– 32% sand 0 to 3 mm,
– 8% limestone filler, where 20% is below 0.002 mm.

Bitumen B 300 by weight: 7.5% to above mix of aggregates. (B 300 refers to DIN 1995).

This mix proved to be flexible enough to follow great differential settlements without cracking which were predicted as 50 cm over a short distance.

The impermeabilization of the foundation was limited due to the great depth of pervious sediments. A diaphragm of plastic concrete was selected. It was to be non-erodible and sufficiently flexible to withstand expected settlements of 2.4 m. The deformation properties are adjusted to those of the neighbouring materials. Two materials have been developed: a plastic concrete of low strength for the upper section with adjoining sand and silt, and a slightly stronger material for the lower section with sand and gravel. Breth & Günther report on the composition and on the properties of the low strength material as follows:

– Base material slope debris of gravel and sand with about 10% below 0.06 mm (by weight),

– d_{max} = 40 mm,

– contents per m^3 of plastic concrete: 1385 kg slope debris (dry), 60.5 kg Portland cement, 16 kg bentonite, 0.3 kg delaying agent, 465 kg water.

– Strength after 14 days storage under water and triaxial pre-consolidation at 0.25 MPa: Modulus of deformation 15 MPa at $\sigma_1 = 0.25$ MPa, 4 MPa at $\sigma_1 = 0.9$ MPa. No failure at $\sigma_1 = 0.9$ MPa and 8% axial strain.

It is worth mentioning that the layer of bituminous gravel on the dam bottom downstream of the membrane serves to prevent migration of silt from the foundation into the slope debris of the dam body. Under the prevailing circumstances the bituminous gravel was cheaper than a mineral filter.

Eberlaste belongs to the smaller examples of ICOLD's 'large dams'. Our knowledge of asphaltic concrete has considerably been improved since 1968 when Eberlaste was completed. There is, therefore, no doubt that larger dams can be successfully constructed with internal membranes of asphaltic concrete on a compressible foundation.

7.4.4.2 *Foundation of face sealings*

Rock foundation

Typical examples of asphaltic concrete faces and related toe structures are shown in Figures 7.60 and 7.61. The toe structure penetrates into the foundation rock. Excavation must be made by smooth blasting. The rock and the interface of the rock and the structure must be improved by consolidation grouting after concreting.

The great length of the toe structure and the perimetric joint are two disadvantages of face sealings. It is very difficult to adjust the design should modifications of the alignment come up due to an unexpected location of the bedrock. In such case large amounts of additional excavation of weathered rock and backfilling of lean concrete have to be accepted. Alternatively, the geometry of the membrane would have to be changed.

An approach to the problem was exercised at the Mornos project. In the Pyrnos area the left bank of the reservoir had to be sealed. This was done using a membrane of asphaltic concrete and related toe structure with a grouting and inspection gallery (Fig. 7.61). A rock foundation of the gallery was standard. Over a section of about 50 m in length the gallery crossed colluvium which reached a depth of 12 m below the nominal foundation line. Excavation of this colluvium would cause a change of the alignment and of the membrane's geometry. It was decided to place the structure on the colluvium and to construct a diaphragm of bored piles on the upstream side to block the seepage across the foundation. Resulting settlements of the gallery were deemed tolerable and not affecting the contact of the membrane with the gallery structure. This approach offered significant cost savings. The bottom and the roof of the structure – with the interface of membrane and structure – followed a regular line in plan and in elevation.

Membranes of conventional concrete are usually connected to a concrete slab at the toe, called a toe slab or plinth. A typical plinth design is shown in Figure 7.53. Such or similar plinths have been incorporated in all typical concrete face rockfill dams. The width of the plinth is selected with respect to the hydraulic gradient, which should not exceed a maximum of $i \approx 20$ for non-erodible rock. Accordingly, the width increases from the abutments down to the valley. The minimum width should be 3.0 m. It is also selected according to the need to maneuver equipment for grouting and slipforms. The reinforcement is at least 0.5% of the concrete section.

The seepage path across the foundation underneath the plinth can be lengthened by placing concrete or shotcrete on the foundation immediately downstream of the plinth. Such lengthening may be necessary for slightly erodible foundations. It is fully effective provided the joint between the plinth and the shotcrete is watertight. Tightness is provided by waterstops. However, such a design creates an additional joint like the critical perimetric joint, with the difficulties of construction and potential problems caused by deformations. Therefore, many designers prefer to suspend waterstops and to place a carefully selected filter on top of the shotcrete. The filter must prevent migration of fines from the foundation across the joint into the material between the face slab and the shotcrete.

Usually, the plinth is used as a grout cap to execute curtain and consolidation grouting. Problems which might arise from this procedure are noted: the grout pressure acts to lift the plinth, thus creating uncontrolled seepage. Therefore, the plinth must be anchored to the ground, and the reinforcement must be designed to exclude bending. Irrespective of this, any movement of the plinth during grouting operations must be controlled. The surface of the plinth should be horizontal in the direction normal to the plinth line, as

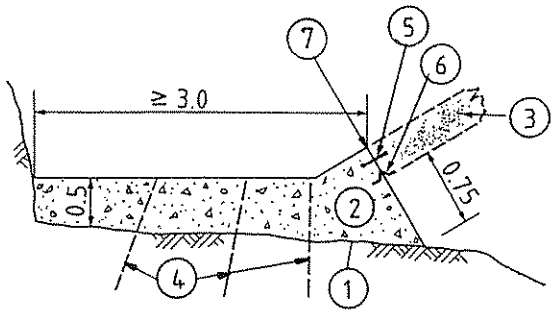


Figure 7.53. Concrete face sealing. Typical plinth design.

1 Rock surface
2 Reinforced concrete
3 Face sealing
4 Grouting holes
5 PVC waterstop
6 Copper waterstop
7 Plinth line
(Joint details see Fig. 7.62, dimensions in m)

shown in Figure 7.53. It is inclined only along the alignment of the perimetric joint (= plinth line). This considerably facilitates all maneuvers of the drilling and grouting equipment.

Compressible foundation
Vertical and horizontal displacements have to be considered at the toe of a dam with a compressible foundation. Vertical settlements follow from the dam's dead weight and from the water load. In critical cases of unacceptable settlements the foundation must be improved. Proven measures are vibro compaction and grouting (Section 7.6).

Horizontal movements originate from sliding along deep slip circles and from the spreading forces mentioned in Section 6.1.3. Safety against ground failure must be ensured by construction measures. There are a number of approaches to the problem of spreading forces and related necessary shear resistance. The finding is that required friction angles to resist spreading forces in a horizontal and cohesionless sliding plane are always smaller than the friction angle of the dam material. Therefore, spreading forces do not lead to significant horizontal movements.

There is a number of dams with face sealing on compressible foundation which show promising performance with respect to toe deformations. Experiences from Italy are reported by a Working Group of the Italian National Committee on Large Dams (1988). Figure 7.54 shows the upstream toe of a 70 m high rockfill dam with asphaltic concrete face and its compressible foundation of alluvium. This stratum is made impervious by a reinforced concrete diaphragm. The toe structure with a gallery rests, watertight and flexible, on top of the diaphragm. The design of the joint permits vertical and rotational deflections. The diaphragm does not contribute to the stability of the gallery structure. Strong, incompressible clay was found about 12 m underneath.

It is a proven measure to locate the gallery structure at a suitable distance from the toe of the dam, about 10 m in the case of Figure 7.54. The selection, grading and compaction of the materials (2) and (3) supporting the asphaltic concrete must exclude settlements and hence tensile stresses of the membrane. The materials must be well graded with some fines to minimize deformations due to the water load. The authors see the permeability as an indicator of the material's suitability. It should be in the order of 10^{-6} to 10^{-4} m/s.

For some dams built on 'relatively compressible soils' defects at the toe have been reported after first impounding, with leakage quantities of some hundred liters per second, however, without generally affecting stability. The defects have been quickly detected and repaired. The design philosophy for dams on compressible foundation and in seismic areas – like Italy – is 'still to rely on an impervious upstream facing. However, on account of the fact

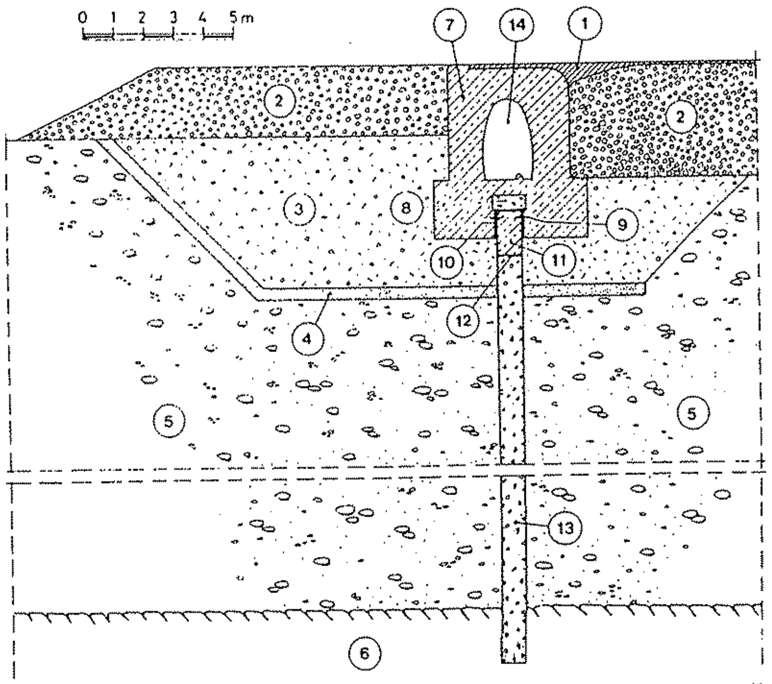


Figure 7.54. Face sealing of asphaltic concrete. Example of compressible foundation (adapted from Working Group of the Italian Committee on Large Dams 1988).

1 Sealing of asphaltic concrete
2 Rockfill
3 Rolled sandy clay
4 Drainage blanket
5 Alluvium
6 Clay
7 Reinforced concrete plinth
8 Bituminous concrete
9 PVC waterstop
10 Bituminous mastic, 20 mm
11 Reinforced concrete connection beam
12 Epoxy resin
13 Reinforced concrete diaphragm wall
14 Drainage and inspection gallery

that a fully reliable waterproofing system is still to be invented, a second line of safety is tentatively considered'. The second line of safety is to permit excessive leakage but to ensure the stability of the dam at the conditions of 'transient flow consequent to an exceptional event leading to an ultimate structural failure of the impervious facing.' The reservoir is taken out of service until the sealing system is repaired.

7.5 INTERFACES

Structures within embankment dams are foreign bodies constituting irregularities of stresses and strains in the dam in the vicinity of the interface of the structure and the embankment. It is recommended to keep the number and the size of interfaces as small as possible. It is, therefore, preferable to place the spillway at one of the abutments – if possible – instead of placing it in the middle of the dam, as is usual with concrete dams. It is preferable to locate the structures for river diversion and power as tunnels in the abutments, instead of close to the surface across the foundation or as culverts across the embankment.

The connections of the shells to structures do not usually create problems. Soil or rockfill material are dumped directly beneath the structure and compacted as usual. It is common practice to limit the maximum size of rockfill to less than the standard, for instance 1000 mm standard maximum size in the dam and 600 mm at the interface.

The connections of structures to the sealing elements need particular attention. The problems of connecting structures with natural sealing elements and structures with artificial sealing elements are, in part, different. They will be discussed in the following sections.

7.5.1 *Interfaces of natural internal sealings and structures*

The main purpose of the design is to achieve permanent tightness of the interface to minimize seepage. In case of earthquakes it must be expected that the embankment and the structure follow different individual motions. A reasonable and proven design of the interface of an earth core and a concrete structure – typically the spillway – is shown in Figure 7.55. In vertical and longitudinal sections the surface of the structure is inclined in such a way as to establish the following conditions:

– The core rests on the concrete structure (Fig. 7.55a). The vertical component of the weight of the core contributes to the contact pressure against the interface.

– The core is pressed towards the concrete structure by the hydrostatic pressure of the stored water (Fig. 7.55b). The hydrostatic pressure contributes to the contact pressure against the interface.

In contrast, the contact pressure against non-inclined vertical surfaces of the structure is a minimum. It is equal to the earth pressure at rest. Dead weight and water load do not increase the pressure against the interface.

The inclination is related to the size of the interface and to the costs of the additional concrete. It should be at least 1V:0.1H. The core material in the contact area is placed in a plastic state, i.e. with an increased water content. The desired high pore-water pressure is achieved by proper compaction of the plastic material. Usually, the concrete surface is not particularly prepared or treated.

In earthquake regions the downstream filters are widened (Fig. 7.55b). This measure is believed to control excessive seepage after different motions

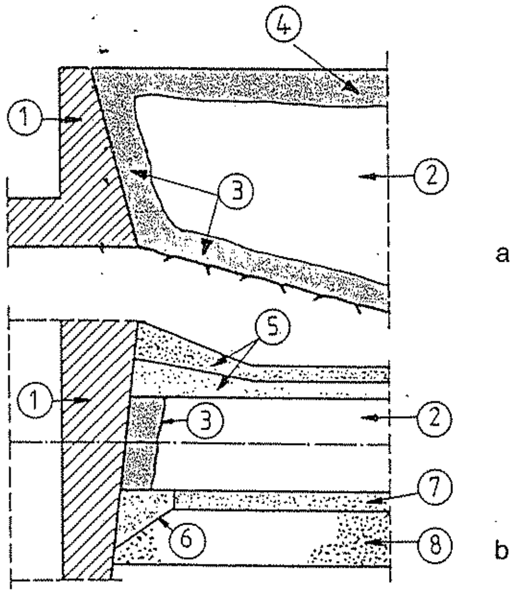


Figure 7.55. Rockfill dam with earth core. Interface of the core and a concrete structure.

a Vertical section
b Horizontal section
1 Concrete structure, e.g. spillway retaining wall
2 Earth core of the dam
3 Plastic soil, water content well above the optimum, e.g. 10%
4 Plastic soil, water content moderately above the optimum, e.g. 6%
5 Filter widening downstream
6 Non-cohesive fine filter causing self-sealing of cracks
7 Filter
8 Transition zone upstream

of the structure and embankment resulting in defective junction of the two media. The fine filter upstream (6) is designed to be washed into voids with a self-sealing effect.

In this context it is useful to point to obsolete design details (Fig. 7.56). Such details are still found in recent designs, but they should be rejected. The wart-like small keys were thought to lengthen the seepage path along the interface, hence reducing the quantity of seepage. However, compaction of the core material at the irregular surface is difficult and unreliable leading to worse conditions than without the keys. Additionally, the contact pressure might be reduced due to arching.

Figure 7.57 demonstrates correct and incorrect designs of the junction of an earth core and a gallery structure: the working space beneath the structure must be re-filled with lean concrete instead of core material. Core material

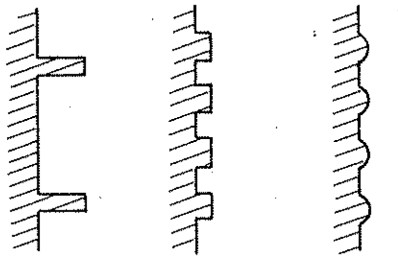


Figure 7.56. Rockfill dam with earth core. Unreasonable design of the interface core/structure.

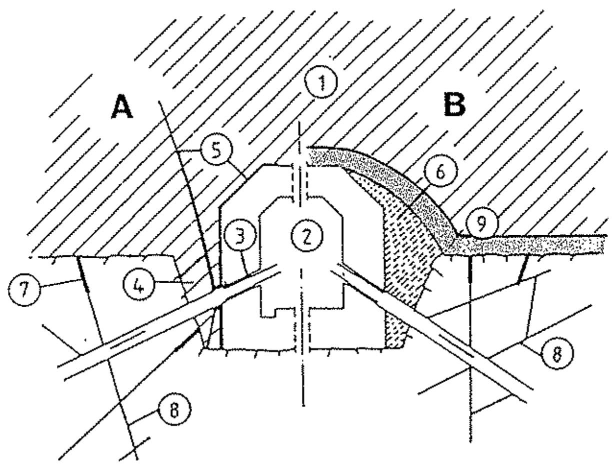


Figure 7.57. Rockfill dam with earth core. Junction of the core and a gallery structure.

A Incorrect design
B Correct design
1 Earth core of the dam
2 Gallery
3 Borehole and packer for contact grouting
4 Backfilling of core material (incorrect)
5 Hydraulic fracturing of the core and the interface core/gallery
6 Backfilling of lean concrete (correct)
7 Sealed superficial cracks
8 Grouted cracks of the foundation rock
9 Layer of plastic material

creates the risk of hydraulic fracture and core damage during contact grouting. Seepage paths might develop at the interface of core and structure.

More critical are structures passing through the sealing element, e.g. a bottom outlet or a diversion culvert. Because of its stiffness the structure attracts compressive stresses to the crown with reduced stresses aside. Such conditions bear the risk of hydraulic fracture. The shape of the culvert affects the stress distribution. Most reasonable is an arch with firm foundation and an inclination of the lateral walls of 1V:1.5H. The best bond between the core and the concrete is again achieved by well compacted plastic soil. Some examples from ICOLD (1979) show 'seepage collars' around structures passing through earth cores, in the sense of Figure 7.56 left. Such collars are known from pipes embedded in concrete. In the opinion of the author the collars should be suspended since they are more detrimental than useful.

7.5.2 *Interfaces of artificial internal sealings and structures*

Artificial internal sealings are mainly made of asphaltic concrete. The width of the membrane is usually a few decimeters to 1.0 m. At the interface with a concrete structure, the width of the membrane is approximately doubled. As an example, Figure 7.58 shows the junction of a membrane and a gallery structure. The interface is slightly curved and coated with mastic. The location of the waterstops is an important detail. Note that the waterstop in the joint between structural blocks must penetrate into the foundation rock and must be tightly connected with the mastic in the interface.

A similar design is shown in Figure 7.59 for the junction of a membrane

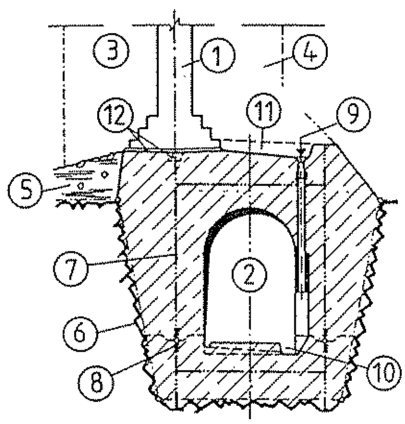


Figure 7.58. Embankment dam with internal sealing of asphaltic concrete. Junction of the membrane and a gallery structure (adapted from Haug & Rothacker 1989).

1 Membrane of asphaltic concrete
2 Gallery
3 Transition zone upstream
4 Transition zone downstream
5 Shell material
6 Rock surface after excavation
7 Waterstop in the joint between structural blocks
8 Waterstop in the construction joint
9 Seepage controlling pipe
10 Drainage
11 Barrier for sectional control of seepage
12 Mastic

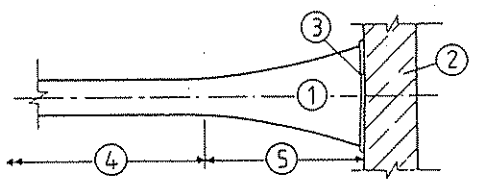


Figure 7.59. Embankment dam with internal sealing of asphaltic concrete. Junction of the membrane and a lateral structure, horizontal section (adapted from Haug & Rothacker 1989).

1 Membrane of asphaltic concrete
2 Concrete structure
3 Mastic
4 Mechanical placing of asphaltic concrete
5 Hand-placing of asphaltic concrete

and a lateral structure. Again the membrane is widened. Usually there is no waterstop. Tightness of the interface is achieved by the mastic coat.

7.5.3 *Interfaces of artificial face sealings and structures*

7.5.3.1 *Asphaltic concrete*

The face membranes of asphaltic concrete usually end at a gallery structure at the toe. The foundation of the structure must prevent any movements. Two contrasting aspects have to be considered at the interface of structure and membrane: the hydrostatic pressure at the dam toe is maximum, hence contributing most favourably to the contact pressure of the membrane. Simultaneously, the maximum hydrostatic pressure may deform the dam material directly beneath the structure. This material is the base of the membrane. In conditions of deformation, differential settlements and shear and tensile stresses will detrimentally affect the membrane and its junction with the structure. Therefore, the dam material must be practically incompressible in this critical zone.

Figure 7.60 shows the junction of a controlled sealing and a gallery structure. The membrane contacts the structure in or near the apex. This configuration eases mechanical construction of the membrane in the valley where the alignment is near horizontal. At inclined sections, starter membranes with hand-placing are required. The structure and the membrane are tightly and firmly connected by the mastic coat (20). The mastic is also connected with the waterstops (11). The wedge of asphaltic concrete (7) helps to compensate for differential settlements. The extension of the waterstop to the wedge is a second line of defense against seepage across the joint between structural blocks. Again, penetration of the waterstop (11) into the foundation rock is noted.

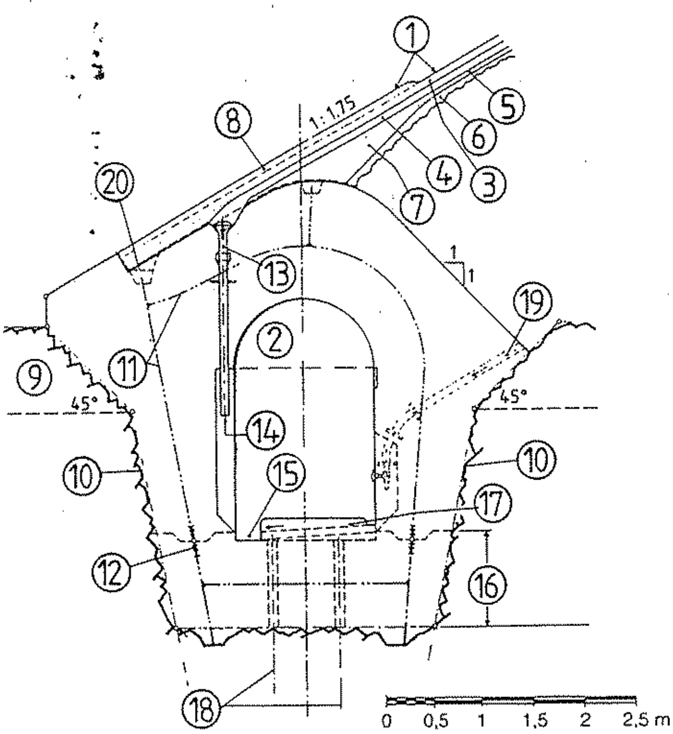


Figure 7.60. Embankment dam with controlled face sealing of asphaltic concrete. Junction of the membrane and a gallery structure (adapted from Haug & Rothacker 1989).

1 Face membrane of asphaltic concrete
2 Gallery
3 Upper layer of asphaltic concrete, 4 + 5 cm
4 Bituminous drainage layer, 8 cm
5 Lower layer of asphaltic concrete, 4 cm
6 Binder course, 5 cm
7 Wedge of asphaltic concrete
8 Protective layer of asphaltic concrete, 5 cm
9 Slope debris or river sediments
10 Rock surface after excavation
11 Waterstop in the joint between structural blocks
12 Waterstop in the construction joint
13 Seepage controlling pipe
14 Pipe recess
15 Drainage
16 Bottom concrete
17 Upper concrete for steps
18 Rock grouting
19 Ground water control
20 Mastic filling and joint coating

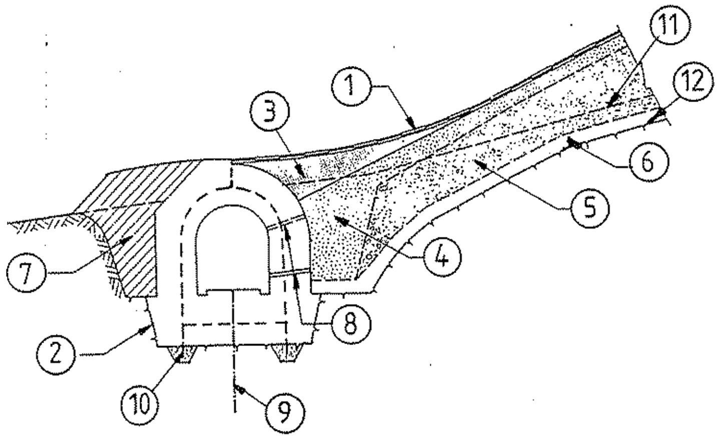


Figure 7.61. Embankment dam with uncontrolled face sealing of asphaltic concrete. Junction of the membrane and a gallery structure, example Pyrnos (Mornos project, archives LI 1976).

1 Asphaltic concrete, 2 × 12 cm + 7 cm binder course
2 Gallery structure
3 Wedge of asphaltic concrete
4 Gravel 5 to 60 mm
5 Gravel-sand
6 Filter layer
7 Lean concrete
8 Drainage holes
9 Grout curtain
10 Waterstop in the joint between structural blocks
11 Original ground surface
12 Rock surface after excavation

The junction of an uncontrolled face membrane and a gallery structure is shown in Figure 7.61. The design has shown proper function though there was a certain risk in the colluvium foundation (see Section 7.4.4.2). The water head is 100 m.

7.5.3.2 *Conventional concrete*

The junction of a membrane of conventional concrete and the plinth is shown in Figure 7.62. This design serves for one of the highest concrete face rockfill dams, namely the Salvajina dam in Columbia, 148 m in height. The perimetric joint is the critical element of concrete face rockfill dams. Tightness of the joint is approached by three lines of defense, consisting of mastic (2), PVC waterstop (4) and copper waterstop (5). According to Sierra et al. (1985) the neoprene and styrofoam pieces (6) and (7) are assembled to the PVC waterstop in order to avoid direct contact of the sharp edges of the concrete with the waterstop. Similar pieces in the bulb of the copper waterstop are designed to prevent concrete from filling the space and tearing and crushing of the waterstop where the joint is under compression. A compressible filler (3) is placed within the joint. It is thought to prevent crushing

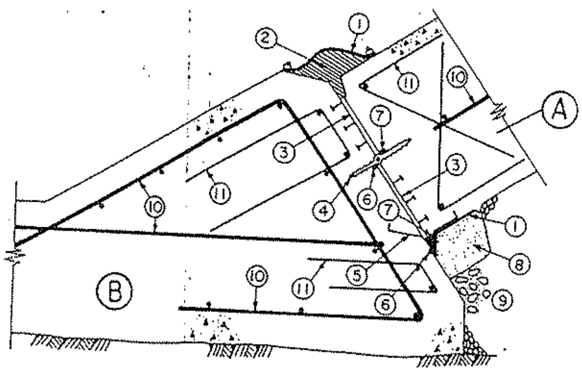


Figure 7.62. Concrete face sealing. Junction of the sealing and the plinth, perimetric joint (example Salvajina dam, Sierra et al. 1985 and ICOLD 1989a).

A Concrete face sealing
B Plinth, reinforced concrete
1 PVC band
2 Mastic filler
3 Compressible filler
4 PVC waterstop
5 Copper waterstop
6 Neoprene cylinder
7 Styrofoam cylinder
8 Sand-asphalt mixture
9 Semi-pervious material ≤ 10 cm
10 Standard reinforcement
11 Reinforcement to protect concrete from crushing and to protect waterstop

of the concrete due to compressive stresses. The filler is nailed to the plinth in the upper part and to the face slab in the lower part. This is to prevent tearing of the PVC waterstop due to downward movement of the face relative to the plinth.

Defects of the perimetric joint due to excessive movements, or due to inappropriate design and construction, was frequently the reason for repair work. This experience will have contributed to the development of the berm of cohesive soil on top of the plinth and the lower portion of the face which is now a common design detail (Fig. 5.8).

7.6 FOUNDATION IMPROVEMENT

7.6.1 *Permeability*

7.6.1.1 *General*

With the great majority of dam projects the substrata are sealed by artificial means to reduce the permeability and the seepage flow around and below the structure. Such measures may be needed for safety reasons: hydraulic ground failure at the downstream toe, erosion of fines and piping must be excluded. The measures may also be needed for economic reasons: the water loss from the reservoir must be kept within tolerable limits.

Proven measures exist to seal rock and soil. The background of all of them is to establish an impermeable zone or a membrane. The zone must be non-erodible over its lifetime. It must be flexible to follow the deformations of the neighbouring zones. There are two main methods of establishing the zone or membrane: 1. Joints and voids of the substrata are filled by injecting hardening and sealing materials. 2. A membrane-like zone of the substrata is excavated and re-filled by conventional or plastic concrete. The most reasonable method is essentially related to the nature of the substrata.

The sealing system should penetrate into an impervious stratum at its lower end to achieve a maximum of effectiveness. The effectiveness is related to the permeability of the system, which again is related to the permeability of the material and to remaining or developing joints, windows and defects of all kinds. The effectiveness of a sealing zone or membrane which does not penetrate into an impervious stratum at the lower end is considerably reduced. The loss in effectiveness depends on the permeability of the stratum underneath and on the length of the membrane. Such systems are called 'hanging membranes'. Brauns (1978) and the author (1996) have reported on the effectiveness of such sealing membranes.

The tightness of a grouted zone and of a membrane is significantly affected by defects. Defects occur due to insufficient grouting and to improper construction of a membrane. According to Brauns (1978, 1987) the effectiveness is reduced to about 90% by a defective area of 0.002 F, where F is the whole sealing area. A defect of 0.01 F (1%) results in a reduction to about 35%. Accordingly, the width of the impervious zone should be increased with the risk of remaining seepage paths, channels and windows. With all construction methods this risk increases with the depth. The risk of defects of membranes is related to the verticality of the sections and to intrusions of bentonite and soil in the filling material.

Grouting is the conventional measure to seal jointed rock and non-cohesive soil. Design and construction details can be taken from standard books, such as Ewert (1985), Nonveiller (1989) and Kutzner (1996). In the present work, grouting techniques are not discussed further.

7.6.1.2 *Bored pile diaphragms*

Such diaphragms are rigid or plastic walls of intersecting piles. Intersection of neighbouring piles is required to achieve tightness. The piles consist of conventional or plastic concrete. Rigid piles are reinforced, if appropriate. The construction method is shown in Figure 7.63. Secondary piles should be bored at a time when the concrete of the primary piles is still easy to cut. Proper selection of the construction time contributes to the verticality of secondary piles and hence to the tightness of the system. The width of pile

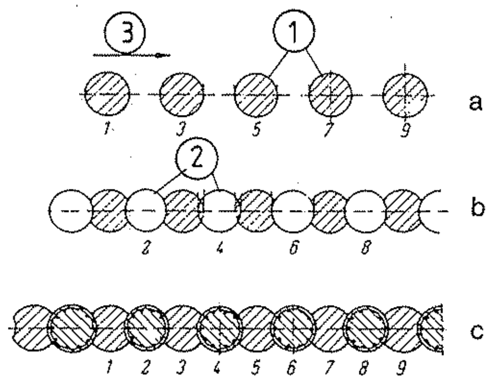


Figure 7.63. Diaphragm of intersecting piles, construction scheme.

a, b, c Construction sequence
1 Primary piles
2 Secondary piles
3 Working progress

walls is about 0.6 to 1.2 m. Intersection is 10 to 15 cm. Due to the nature of the soil the boreholes are drilled with or without casing and with or without stabilizing drill fluid.

Up to now, the deepest diaphragm of bored piles was made for the Manicouagan 3 dam. The system and its connection with the dam's earth core are shown in Figure 7.64. At the abutments, the system consists of trench diaphragms less than 50 m in depth. It is accomplished by additional rock grouting down to further 30 m at the abutments and 7.5 m in the valley.

In the section across the valley, the surface of the bedrock is U-shaped with moderately sloping abutments. In the middle of the valley it is rather V-shaped with steep slopes. The strata above the bedrock consist of silty fine sand, sand, gravel and cobbles up to 0.9 m edge length. The permeability varies from 10^{-7} to 10^{-2} m/s. Such substrata lead typically to the selection of a diaphragm of concrete. Grouting methods would not reveal complete sealing. There is a risk of leaving untreated, possibly erodible zones. In contrast, the jointed bedrock leads typically to the selection of a grout curtain. Drillings to produce a diaphragm of bored piles are too expensive.

A great number of different types of measuring instruments enables us to evaluate the performance of the substrata and the sealing system after 15 years of operation. According to Dascal et al. (1991) the longest piles have been compressed in length by about 160 mm. The lateral deflection of the pile heads is about 320 mm. The distribution of pore-water pressures in the substrata points to the existence – as expected – of a minor seepage flow across the sealing system. The pressure drop from upstream to downstream

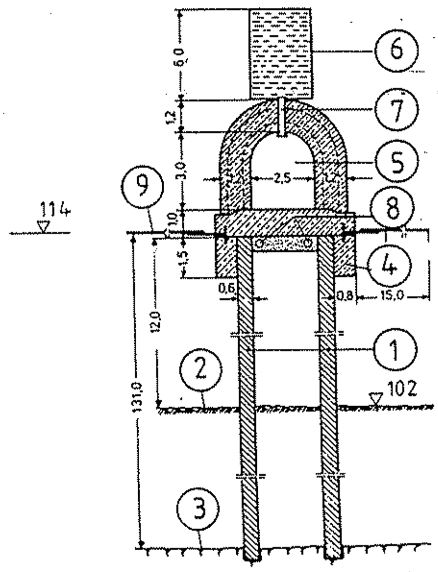


Figure 7.64. Manicouagan 3 gravel dam. System of substrata sealing, consisting of two diaphragms of intersecting piles with gallery structure on top, penetrating into the dam core (adapted from Lorenz 1976).

1 Intersecting pile diaphragm
2 Bottom edge of the dam core
3 Bedrock surface
4 Cap of reinforced concrete
5 Gallery
6 Pad of bentonite
7 Drainage
8 Drain pipes
9 Steel membrane
(Dimensions in m, elevations m a.s.l.)

corresponds to a water head of 90 to 95 m, resulting in an effectiveness of the system of about 90%. The seepage quantity is reported to be 40 to 50 l/s.

The measurements allow us to conclude that the pile diaphragms of Manicouagan 3 are slightly more effective than the trench diaphragms. Dascal et al. assume increased flexibility of the pile diaphragms with respect to deformations of the foundation, owing to the increased number of joints between neighbouring piles. The piles are 0.6 m in diameter; the panels of the trench diaphragms are 3.3 m in length. According to common experience from other projects, the great number of joints is regarded as a disadvantage. The contrasting performance of the Manicouagan pile diaphragms may be due to the curvature in plan of only 600 m radius, leading to increased joint pressure and improved tightness.

7.6.1.3 *Trench diaphragms*

A trench diaphragm is an alternative to the pile diaphragm. This structural element is explained in Section 7.3.2.2 as a measure to seal embankment dams. The trench diaphragm consists of sections – frequently called panels – 2 to 4 m in length. The wet methods N1 and N2 are applied to construct trench diaphragms in the foundation, typically consisting of soil or highly weathered rock which is not groutable. The construction procedure is shown in other books, e.g. Xanthakos (1979), quoted in Fell et al. (1992).

The method N1 creates a membrane-like wall of plastic concrete. After hardening of the primary panels the secondary panels are excavated, whereby the excavator scrapes some centimeters of the neighbouring primary panels away. The material of the secondary panel connects without, or almost without, joint with the primary panels.

With the method N2 the excavated primary panels are stabilized by bentonite slurry until the concrete is poured. If conventional concrete is used it is necessary to place a stop-end tube to prevent the fresh concrete from collapsing. The tube is removed prior to complete hardening. The remaining half round key is seen as a guide to excavate the secondary panel. Joints exist between neighbouring panels which may require subsequent treatment to block seepage (references see above).

Rigid concrete is subjected to compressive stresses due to settlements of the adjoining soil and related negative skin friction. It must be ensured that the stresses do not lead to concrete cracking. Lorenz (1976) reports an example where the calculated stresses in a 88 m deep diaphragm exceed 20 MPa at settlements of 50 cm. In addition, there is a tendency of the rigid diaphragm to penetrate into the sealing element or the structure on top. Elaborate measures are required to exclude damage to the two parts. Examples are given in Figures 7.54 and 7.64.

Diaphragms of plastic concrete are more flexible in following the deformations of the foundation. The failure strain of plastic concrete is up to 10% (Table 9.6). A boundary of application is given by the cement content: it must be low to guarantee a maximum of flexibility, related to the adjoining soil. But the cement must guarantee enough strength to exclude erosion of the material. The erosion behaviour of plastic concrete is largely known (Sections 7.3.2.2 and 9.6.1).

The construction of trench diaphragms was considerably improved by the development of hydrofraise equipment (Fig. 7.65). The hydrofraise loosens the material in the trench and crushes gravel and cobbles to a maximum grain size of 60 mm. The loosened and crushed material is mixed with the stabilizing slurry in the trench and pumped to the surface by hydraulic pumps. The slurry is continually screened and desanded by vibrating sieves. The desanded mud is used again as stabilizing fluid. The loosening and crushing capacity of hydrofraises enables the excavation of much stronger material than conventional slurry trench machinery. Upper limits of strength are about 150 MPa of the material in situ and 200 MPa of individual pieces of rock. That means weak rock as in Figure 4.13, conglomerate as in Figure

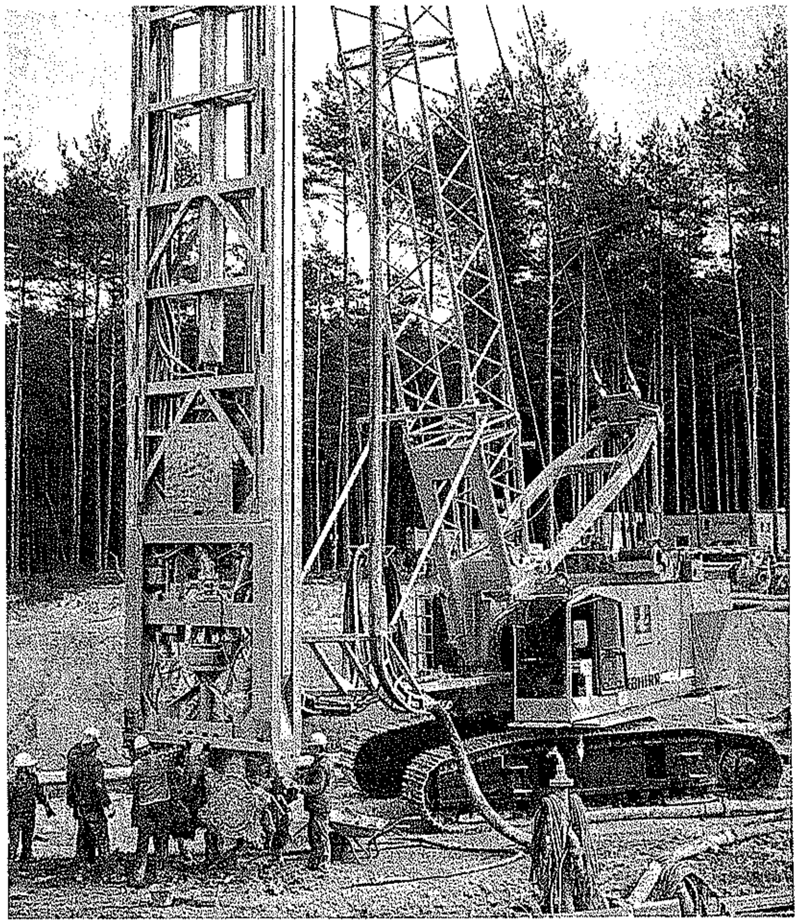


Figure 7.65. Hydrofraise and carrying crane (courtesy of Bauer).

4.7, but of great strength, and the coarsest river sediments can be fraised and excavated to produce the trench. Any need for chiselling is eliminated.

Up to now depths of more than 100 m have been reached at 0.6 to 1.2 m width of the trench. The verticality is reported to be 0.1 to 0.2% of the depth. This accuracy is achieved by electronic control and regulation of the fraise. According to Graybill & Levallois (1991) the deviation from vertical of a 120 m deep trench in a dam core of sandy gravel with 20% silt was less than 0.2%. In this example the lower end of the trench is keyed into strong, volcanic agglomerates of andesite and tuff with a compressive strength of 185 MPa.

7.6.2 *Bearing capacity*

Usually, the bearing capacity of rock foundations is satisfactory. Only existing large voids or caverns require an improvement. An example is karstic limestone (e.g. Fig. 4.11). Underground caverns must be cleaned and then backfilled with concrete via boreholes or shafts. Subsequent grouting provides strong contact of the concrete with the adjoining rock.

Caverns at the surface must be cleaned of loose material and then backfilled. Underneath dam cores and other sealing elements, concrete is used for backfilling. Underneath the dam shells any material may be used for backfilling which can be properly compacted and which shows sufficient strength.

Frequently, consolidation grouting serves to improve the bearing capacity of rock foundations (Fig. 7.40). Such grouting is performed – down to 6 to 12 m – in addition to the grout curtain. Consolidation of the foundation means minimizing settlements. This can only be achieved by filling large voids and joints with hardening material. The dead load of the dam itself will contribute to the consolidation of minute joints that are not groutable.

Soil foundations may require the consolidation of compressible layers down to 10 and more meters. Proven means for that are vibro compaction of non-cohesive soils, vibro replacement of cohesive soils by gravel columns, fill grouting of non-cohesive soils, crack grouting of cohesive soils, pore pressure release by drainage and dynamic consolidation by falling weight (up to 40 tons and 30 m drop height). An example of the effect of vibro compaction is shown in Figure 7.66. The sand in situ was in a loose to medium dense state. The density was increased by vibro compaction. Control was done by static cone penetration tests. The penetration resistance – in terms of tip pressure – of the sand in situ was about 5 to 8 MPa and 30 to 40 MPa of the compacted sand.

An example of a particular soil improvement by a mix-in-place procedure is given by Luebke et al. (1991). It is the stabilization of liquefiable sediments in the foundation of a 12 m high embankment. The procedure is similar to jet grouting, which is also a mix-in-place method. Double and triple auger drill rigs are used instead of the jet grouting rods and nozzles. The cement suspension is pressed through the drill rods, and mixed with the loosened soil around the auger by the auger's rotation. Pressures are in the common range of conventional grouting, instead of around 40 MPa as in the case of jet grouting.

The foundation was stabilized by producing a honeycomb pattern of soil cement columns with adjacent columns having continuous contact (Fig. 7.67). The strength of the treated soil columns was 1.0 to 6.0 MPa, depending on the original soil properties, the cement content of the columns and the drilling and withdrawing procedure. The diameter of the soil mass confined

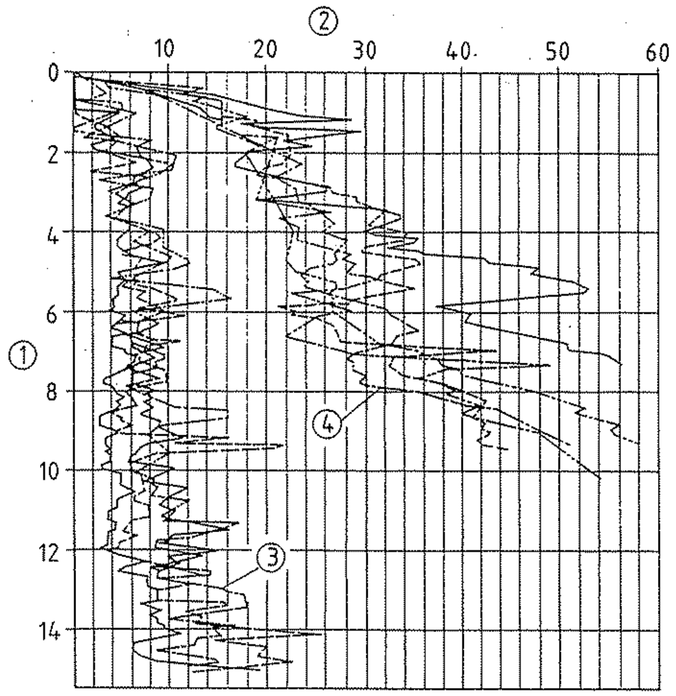


Figure 7.66. Penetration test results in sand before and after vibro compaction (adapted from Schultze & Muhs 1967).

1 Depth (m)
2 Tip pressure (MPa)
3 Sand, uncompacted
4 Sand, compacted

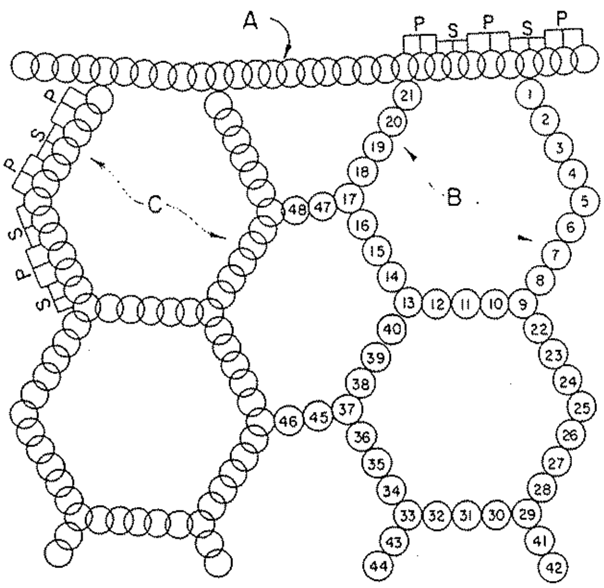


Figure 7.67. Honeycomb pattern of soil cement columns for soil stabilization (Luebke et al. 1991).

A Cut off wall
B Double auger pattern
C Triple auger pattern
P Primary stroke
S Secondary stroke

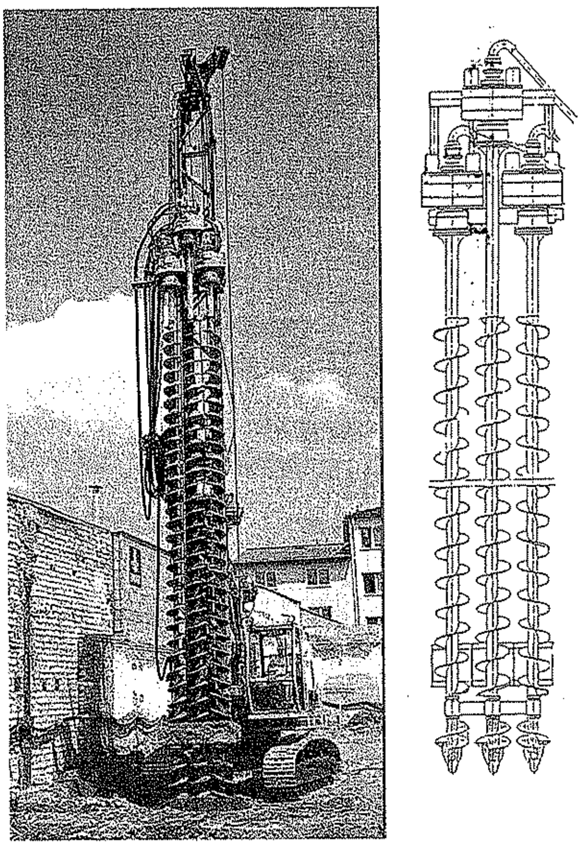


Figure 7.68. Triple auger for construction of slot diaphragms (courtesy of Bauer).

in the honeycombs is in the range of 5 to 7 m. The strength of the columns and the selected pattern were deemed adequate to increase the shear resistance of the soil mass as a whole and to guarantee safety against liquefaction under earthquake conditions.

In the opinion of the author this procedure is an innovation in soil improvement, using the experience of jet grouting without taking the high costs into account and without stabilizing the whole soil mass. The following may be seen as a disadvantage: it is practically impossible to give evidence of the stabilizing effect in respect of the whole soil mass. Evaluation is mainly a matter of quality tests on columns and of a computational approach by parameter studies – and of engineering judgement.

In Germany the described soil mixing procedure is used to construct slot diaphragms (Fig. 7.68). In Japan it is used for the stabilization of marine soils (Kawasaki et al. 1981, Saitoh et al. 1985).

CHAPTER 8

Stability analysis

8.1 INTRODUCTION

This chapter will show which parts of the design work are usually a matter of computational analysis and which questions can be answered by the analysis. It does not explain the methods of analyses to the reader.

All analyses are based on assumptions. Such assumptions are mainly related to the properties of the soil and rock mass which is affected by the structure and to the properties of the construction materials. The assumptions are derived from the results of previous substrata and material investigations and from experience of similar projects.

Furthermore, all analyses need some simplifications. Such simplifications are related mainly to the geometry of the structure, limiting the effort in preparing and performing the computations in a practical sense, without deteriorating the reliability of the results thereof. Computations according to the finite element method (FEM) do not only simulate temporarily existing conditions but also processes which last over a certain period of time. Also, such processes need simplification, e.g. the construction process and the process of reservoir impounding.

During the last decades analytical techniques have been developed for all questions to be answered by analysis. The monitoring of the performance of existing structures has given evidence that most of the analytical techniques cover the given problem satisfactorily, at least within permissible tolerances. Such techniques are laid down in recommendations and guidelines, or they have developed to standards which are in common use for design work.

For a number of problems, different techniques, competing with each other, have evolved. It is up to the designer to select one of them or to follow two or more different ways in analyzing. He has then to decide on the value of the results. It reflects sound engineering to follow this approach and to consider his own experience, and that of others, when a decision is made.

For static and for dynamic loads the analysis is usually made separately. Typical load cases are:

1. End of construction, with and without earthquake. Only the dead load of the structure is effective. The consolidation is not completed; there is no impounding.

2. Steady seepage conditions, at full supply level, with and without earthquake. The consolidation is completed.

3. Operational drawdown of the reservoir water level, with and without earthquake.

4. Rapid drawdown, with and without earthquake, as an exceptional case which is not a normal operation.

5. First impounding. This load case relates to the pore pressures, i.e. to the degree of consolidation at the time of impounding.

6. Temporary construction conditions, such as construction flood, excavations, temporarily existing slopes.

It is common practice to apply the operational basis earthquake (OBE, Section 6.2) for the rapid drawdown case. An individual decision must be made on the application of earthquake loading for the load cases (5) and (6), according to the particular circumstances.

As a rule, safety against static, dynamic and hydraulic loads and safety against cracking have to be analyzed (DIN 19 700, part 10). Safety against static and dynamic loads refers to the slope stability and to the compatibility of stresses and strains with the construction materials. There are two basic methods for the respective analyses: the limit equilibrium and the finite element methods. Both methods are applied; they complement each other.

The limit equilibrium method leads to a factor of safety. It is 'the factor by which the shear strength parameters may be reduced in order to bring the slope into a state of limiting equilibrium along a given slip surface'. The analysis answers to the question: 'How safe is the structure against a partial or total failure?' (Quotations refer to ICOLD 1986a). Usually, deformations are not considered.

The finite element method allows us to investigate stress and strain conditions. It covers all time-dependent processes leading to changes in the stress and strain distribution. It answers the question: 'Will the deformations of the structure remain within limits tolerable for the operation and function of the structure?' Respective analyses include considerations relating to safety against cracking.

8.2 STATIC LOADS

8.2.1 *Limit equilibrium*

The limit equilibrium state is considered with respect to slip surfaces through the dam body along which the overlying portion of the mass may slide. The slip surfaces are constituted by circular or polygonal lines – as in Figures 6.7 and 6.8 – or other lines, such as a logarithmic spiral. The movement of the sliding mass along the slip surface must be kinematically possible. Larger internal deformations during the movement of the sliding mass lead to inaccuracy of the analysis.

The sliding mass is usually divided into a number of vertical slices. Then the equilibrium conditions of the slices are analyzed. A number of procedures has been developed, which differ mainly in two aspects:

– Consideration of either force or moment equilibrium,

– different consideration of the forces acting on the boundaries between the slices.

Investigations described by ICOLD (1986a) arrived at the following result: 'The factor of safety obtained by satisfying moment equilibrium is relatively insensitive to the interslice force assumption'. In contrast, procedures based on force equilibrium lead to factors of safety that differ by ±10%, according to the interslice force assumption. It is common practice to use one of the procedures based on moment equilibrium and vertical slices (e.g. Bishop 1955, Morgenstern & Price 1965, DIN 4084). Recommended factors of safety do not vary significantly in the standards of different countries. DIN 19 700 (part 10) sets the factor of safety at 1.3 for the above load cases with the exception of load case (5) where 1.2 is recommended.

The limit equilibrium procedures suffer from theoretical shortcomings which are frequently criticized: the different stress-strain properties of the soils in the slip surface and related non-linear mobilization of shear strength are not considered, which leads to the incorrect assumption that the factor of safety is the same for every slice. This results in the fact that the overall factors of safety calculated by Bishop's method differ by up to 8% from those obtained from FEM. For dams on weak foundation it was found that 'a sliding wedge (or a shallow non-circular surface near the surface) would usually be more realistic' (ICOLD 1986a). Irrespective of this the recommended moment equilibrium procedures are in use worldwide and have proven satisfactory in practice.

The computations are supported by electronic means. This misleads many designers to analyze a large number of slip surfaces. It is noted that some experience allows us to localize the zone of potential critical slip surfaces and hence to find out the most critical one by analyzing a small number of slip surfaces only.

There are attempts made to mobilize FEM in computing the slope stability, as to eliminate the a.m. shortcomings. ICOLD (1986a) concludes that 'the finite element method can usefully complement limit analyses by providing information about the development of failure'. However, the two methods of application are expensive, either in computing or in man time, 'and in any case the accuracy may not be better than that obtainable from the more sophisticated of the equilibrium methods'. So the use of FEM for slope stability computations 'will probably not normally be justified'.

8.2.2 *Stresses and deformations*

The following processes of dam construction and reservoir operation are computed with the aid of finite element methods: dam construction in several steps, reservoir impounding in several steps, steady seepage at full supply level and all modes of drawdown. Critical stability conditions may occur during each of these processes.

The main purposes of FE-analyses are the determination of stresses including pore pressures, of stress distribution and the prediction of deformations. Thereby it is possible to verify the interaction of the dam zones, the location of potential cracks and the risk of hydraulic fracture. Computed stresses and strains can be expressed in terms of a coefficient of utilization, which is the ratio of the existing stress or strain level to the failure stress or strain level. In this way the analyses will give valuable information for design work, and on the dam's performance.

The majority of analyses is performed assuming two-dimensional plane strain conditions (2D-analysis). Such simplified analyses proved to provide reasonable results for structures where the deformations of the materials are mainly two-dimensional. A three-dimensional analysis (3D) is recommended, for instance, for dams in narrow valleys with an arching effect and dams with very different abutment conditions. 3D-analyses would apply mainly to cross-valley sections.

Figure 8.1 demonstrates the element grid of a 2D-analysis of the typical section of the Kinda dam. It consists of 218 elements and 248 nodes. The 2D-simplification is justified because of the great length of the dam in comparison to the height (L/H = 8.2). The figure shows the load case 'end of construction' after nine steps of material embanking. Each horizontal line marks one of the steps. The layer thickness is 5.5 to 9.1 m. Such simplification of the construction process is added to the simplification of the dam's geometry. The construction process is seen as an individual problem regarding the varying stress conditions and related varying material parameters, with reference to the changing height of the embankment. Also, the process of impounding is simulated by dividing it into a small number of intervals.

In this example the dam's geometry is simplified, in that the core and crest elevations are the same and the downstream berms are not modelled. The assumed foundation line is a simplified picture of reality. The foundation is not modelled. The foundation rock is assumed to be incompressible. The deformation modulus is estimated as at least 3000 MPa with related settlements of 3 cm as a maximum. Individual parameters are introduced for

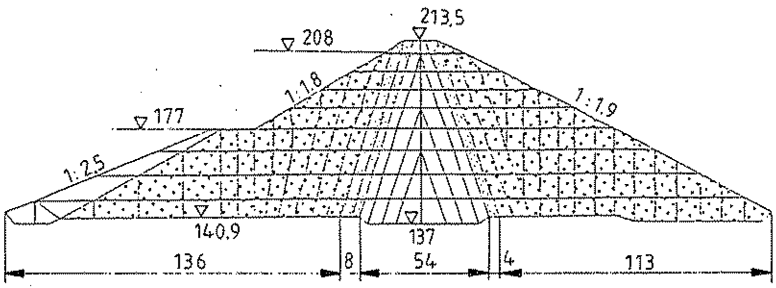


Figure 8.1. The Kinda rockfill dam, as in Figure 5.4. Finite element model for 2D analysis (dimensions in m, elevations m a.s.l., Kutzner et al. 1988).

the shells, the transition and filter zones and for the core (see Table 4.10, nos 3, 7 and 10). In this case the parameters are obtained from back-analyses of measured dam deformations. They are usually obtained from triaxial tests, the results being modified according to experience, if appropriate.

It is an essential of FE-computations to adjust the stress-dependent material parameters in steps to the actual stress conditions of the structure. Frequently, the adjustment follows a non-linear elastic model of the material's stress-strain behaviour, which is expressed by dimensionless parameters (e.g. after Duncan & Chang 1970 or Duncan et al. 1980). Examples of stresses and deformations being computed this way are shown in Figures 6.2, 6.6, 7.45 and 7.46. Alternatively, other material models may be used, such as a linear elastic or a more refined elasto-plastic model.

The variation of the principal stresses of a rockfill dam with inclined earth core from the end of construction to full impounding is shown in Figure 8.2. In the upstream shell the principal stresses are reduced by uplift. In the core and the inner part of the downstream shell they are increased due to the rising water load. It can be seen that the additional loading affects only the inner part of the downstream shell and area A at the contact with the foundation. The direction of the principal stresses is rotated counter-clockwise. This points to differential movements of the core and the upstream shell. Parameter studies with varying core inclinations would demonstrate the effect on the differential movements.

The value of FE-computations in recognizing the risk of hydraulic fracture and cracking is illustrated by Figure 8.3. It demonstrates the effect of a key trench with steep slopes for the Teton dam. The dam consisted of a very wide core of clayey and sandy silt with filters, transition zones and shells. Due to arching across the key trench a significant deficit of vertical stress developed which is seen as one of the reasons for the dam's collapse. Computations, with the result shown in Figure 8.3, had been made only after the event. ICOLD (1986a) comments as follows: 'Had such an analysis been

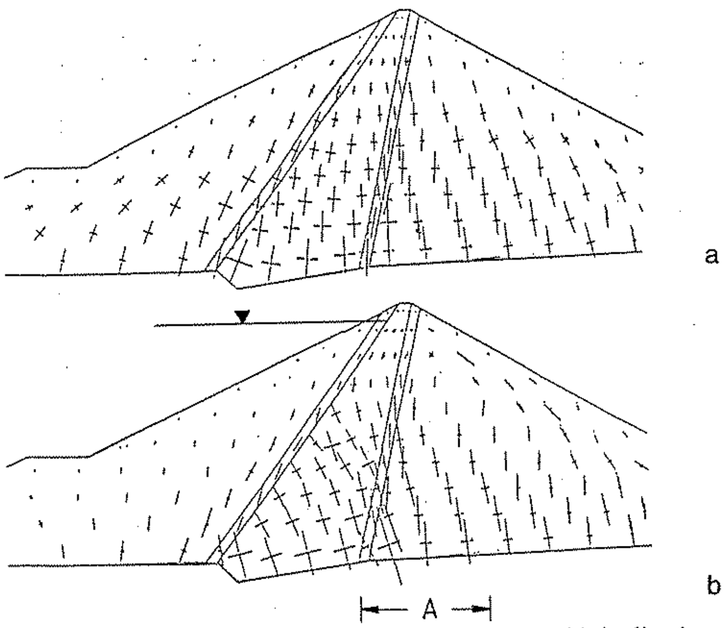


Figure 8.2. Simplified 2D analysis of a rockfill dam with inclined core as in Figure 5.5 (archives LI).

a Principal stresses at end of construction
b Principal stresses after impounding to full supply level
A Zone of increased stresses in the foundation area

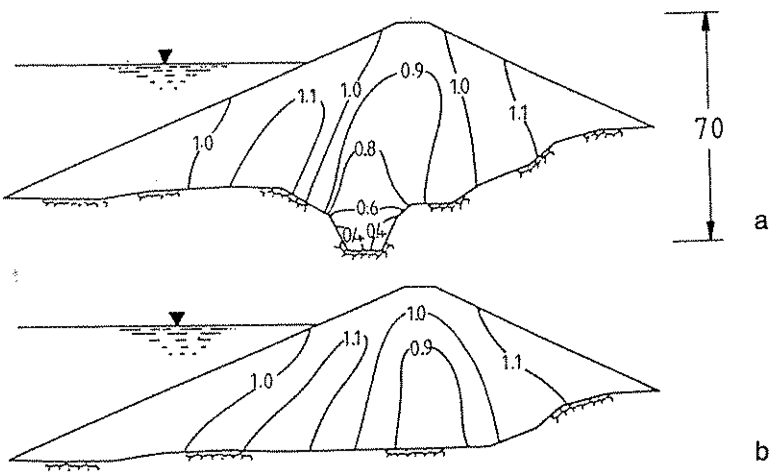


Figure 8.3. The Teton earthfill dam. Computed distribution of vertical stress to overburden ratio (adapted from Penman 1977).

a As-built critical section with key trench
b Section with no key trench

carried out beforehand and interpreted in the light of hydraulic fracture potential, then the design would surely have been changed'.

Figure 8.4 demonstrates the result of FE-computations performed for the tender design of the Chico dam, which is shown in Figure 7.14. Some problems of the design are discussed in Section 4.2.6.1. Figure 8.4 shows the minor principal stresses and the distribution at full supply level. The stresses are computed for steady seepage conditions with the assumption of permeabilities k_V:k_H as 1:9. At the upstream toe of the core we notice a zone of minimum stresses with the risk of hydraulic fracture. The core should be widened to reduce the exit gradient of the seeping water. It is noted that no dangerous events have been observed with dams of similar geometry. Probably, the input of the computation is too pessimistic.

As to static FE-computations the following is noted: such computations are indispensable for the design of embankment dams and for predicting their performance under operation. The key to the accuracy of the computations and related interpretation is the correct approach to the stress-strain behaviour of the materials and the parameters derived therefrom. Up to now a non-linear elastic model is mainly used which is based on triaxial tests. However, such tests do not correctly reflect the stress path of the principal stresses during dam construction since – during the test – the deviator stresses ($\sigma_1 - \sigma_3$) increase at constant confining stresses σ_3. Given steady seepage and drawdown conditions, other stress paths may be relevant. Therefore, ICOLD (1986a) concludes that 'there is a need to obtain more experience with elasto-plastic models... and a need to assess the differences between non-linear elastic and elasto-plastic models'. Apart from this, measurements of existing dams should increasingly be evaluated and made public worldwide, with respect to improvements and calibrations of the computational approaches.

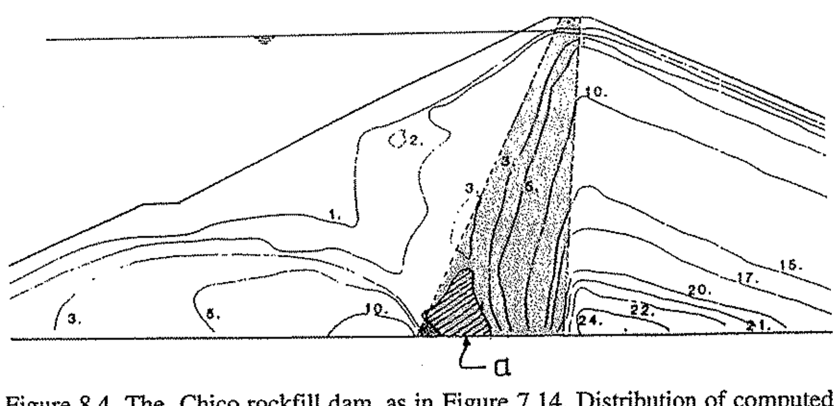


Figure 8.4. The Chico rockfill dam, as in Figure 7.14. Distribution of computed minor principal stresses at full supply level (figures ×100 in kPa, archives LI).

a Zone of effective stresses $\sigma_3' < 0$.

8.2.3 *Hydraulic conditions*

Here, our considerations are focused mainly on the seepage conditions and seepage quantities which must be conducted to the area downstream of the dam with no deterioration of the structure. Proof must be given that the filters are properly designed with respect to erosion stability and drainage capacity. The proof includes surface and internal erosion (see Section 7.3.3.1).

The analysis of seepage paths and quantities can benefit from the use of FE-methods. These permit incorporation of the effects of seepage through the foundation, and of anisotropic permeability, which cannot easily be quantified. Such analyses of flow nets result in filter dimensioning, seepage quantities and exit gradients for all possible cases, for instance leakages in surface sealings of given size and location. The value of exit gradients is important in respect of hydraulic ground failure at the downstream dam toe. Usually the exit gradient at the interface of an erodible base soil and the protective filter does not create problems if the filter is designed according to the geometrical approach, as in Figure 7.30. Dispersive clays are an exception, since the problem of colloidal erosion through filters needs individual clarification, for instance by NEF-tests (Fig. 7.33).

8.3 DYNAMIC LOADS

8.3.1 *Limit equilibrium*

The stability of sliding masses under dynamic loads is analyzed for assumed slip surfaces, similar to the stability of sliding masses under static loads. The dynamic loads are simulated by amplifying the gravity forces of the sliding mass. One of the classical approaches is the model of the rigid block on an inclined plane (or slip surface) being prevented from sliding by the friction and cohesion acting in the interface. To simulate dynamic effects, the gravity forces of the block are magnified by the seismic coefficient, which is a fraction of the acceleration due to gravity. It is common practice to set 0.05 g to 0.25 g as the horizontal seismic coefficient, and 1/3 to 1/2 of this as the vertical seismic coefficient, both in relation to the magnitude of the anticipated earthquake.

The method is known as pseudo-static analysis. (It would be more precise to call it pseudo-dynamic analysis because it is no true dynamic analysis.) It is a simplification of the physical reality in that the seismic coefficient is set constant from the bottom to the crest of the dam and in that it is set as invariable with time and acting in one direction only, instead of being cyclic. The simplification does not reflect the actual, much more complex conditions as can be seen e.g. from Figure 6.9, where the acceleration varies from the bottom to the crest. In addition it varies with time, and it is cyclic. The analysis can be improved by the use of a seismic coefficient varying from the bottom to the crest and varying with the location of the slip surface (Ambraseys & Sarma 1967). The effect of the variable frequency is still not considered, even with this improvement.

After the development of refined earthquake response analyses and after monitoring of dams that experienced earthquakes, it was found that the pseudo-static analysis provides conservative safety factors of dams consisting of cohesive materials and dense non-cohesive materials. Accordingly, Idriss & Duncan (1988) conclude that pseudo-static analyses 'continue to offer a simple and reasonable means for evaluating the potential behaviour of embankments', provided the undrained strength of the dam construction and foundation materials is not significantly affected by the periodic straining, i.e. no pore pressures are generated.

In the opinion of the author, it is a reasonable approach for feasibility studies. It is acceptable also for tender designs because – provided the design is sound in an engineering sense – variations between the tender design and the more refined final design for construction will refer only to the volume of the dam and its different zones. This again applies to the previously mentioned cohesive and dense non-cohesive materials with no pore pressure generation.

Recommended factors of safety are 1.2 for the load cases of regular operation and 1.1 for the load cases of irregular operation, such as rapid drawdown as an emergency case. It was shown for an existing gravel dam with earth core that factors of safety < 1.0 can be tolerated (Kutzner 1980). The precondition for this is the proof that respective deformations of the slopes are not detrimental to the structure. The rate of deformation can be estimated after Ambraseys & Sarma (1967). The deformations will be within tolerable limits at safety factors of 0.9 to 1.0 (Sarma 1975).

8.3.2 *Stresses and deformations*

The main purposes of dynamic FE-analyses are the determination of stresses and strains, of pore pressure development and dissipation, and the prediction of cumulative deformations, now under the influence of dynamic loads with respect to the time history of stresses and strains. Respective computational techniques are complex. Different approaches and computer programmes exist. Some of the references for detailed studies of the matter frequently recommended are: Newmark (1965), Seed (1979), Seed (1981), Seed et al. (1984), and Idriss & Duncan (1988), all of them listing more references. ICOLD (1986b) presents techniques for evaluating the response of dams to specified input motions.

The preconditions for a complete earthquake response analysis are:

– Development of a design earthquake that corresponds satisfactorily to the peak ground acceleration and related time history of anticipated earthquakes at the location of the dam (e.g. Fig. 6.9a).

– Determination of the initial stress conditions by a non-linear static FE-analysis.

– Availability of appropriate dynamic parameters of the construction and foundation materials, derived from cyclic triaxial and other tests.

The static and the dynamic analyses are performed using the same grid of finite elements. The effect of the shape of the valley on the individual motion and related deformations of the structure is simulated by an amplification of the shear modulus of the materials. This way, the use of a 2D-model is justified. It should be borne in mind that the 3D-constitutive laws are not yet completely discovered. It is usual practice to double the shear moduli for V-shaped valleys up to a ratio of length/height = 5/1 to 6/1.

In the course of the dynamic analysis cyclic stresses are superimposed on the previously established static stress conditions of the structure. The cyclic stresses are related to the selected design earthquake. The shear moduli and the damping factors of the materials are adjusted in steps to the actual stress and strain conditions. The relation of shear moduli and damping factors to the shear stresses is computed from standard curves for typical materials. The functions are usually implemented in the computer code.

As an example, the result of an analysis performed for the tender design of the Chico dam is described here, which enabled the designer to evaluate the construction materials in respect of earthquake-dependent deformations.

A decisive output are the histograms of shear stresses of selected elements (Fig. 8.5). The initial and short-term amplification of stresses and the modification of the frequency of the shaking on the way from the lower to the upper part of the dam are noted. The latter corresponds to the computed crest acceleration (Fig. 6.9b), which is approximately equal to 1.0 g. For a number of reasons this acceleration – conservatively determined – is seen as tolerable in the present case.

For further evaluation of the structure it must be checked whether or not the cumulative deformations can be tolerated. For this purpose dynamic stress-strain properties must be introduced which are derived from triaxial tests. Figure 8.6 shows the results of such tests made on the earth-rock material of Chico (Table 4.10, no 1 and Fig. 4.32, no 12). Figure 4.13 shows the material in situ. The shear strains increase with the number of load cycles. Necessarily, the number of cycles required to achieve any selected strain level, e.g. 5%, decreases with increasing deviator stresses ($\sigma_1 - \sigma_3$), i.e. with decreasing confinement. Within this material the pore-water pressure develops instantaneously.

The tests allow us further to determine the stress conditions which are re-

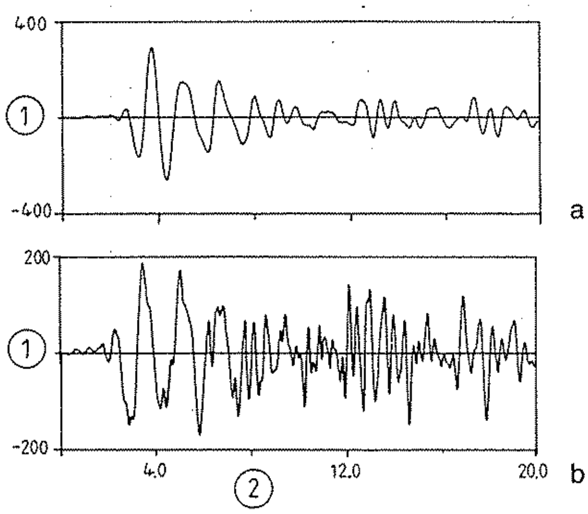


Figure 8.5. The Chico rockfill dam, as in Figure 7.14, upstream shell. Computed shear stress histograms at full supply level and earthquake (archives LI).

a Element in the upper third of the dam, close to the core
b Element at the bottom of the dam, below the cofferdam crest
1 Shear stress (kPa)
2 Time (s)

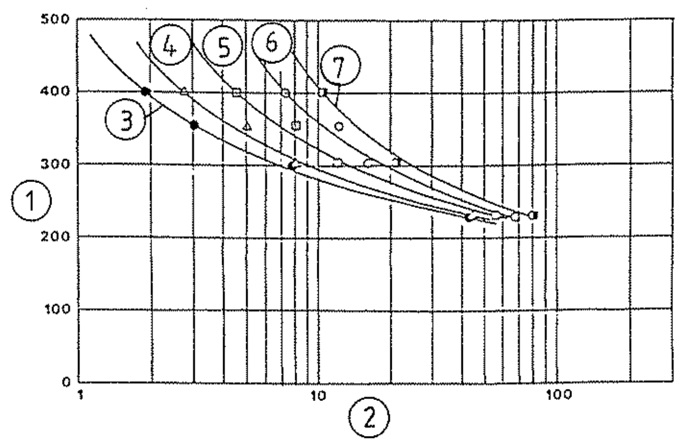


Figure 8.6. The Chico rockfill dam. Results of cyclic triaxial tests on earth-rock material (no 1 in Table 4.10, archives LI).

1 Cyclic deviator stress $(\sigma_1 - \sigma_3)$ (kPa)
2 Number of cycles
3 Pore-water pressure 100%
4 Shear strain 2.5%
5 Shear strain 5%
6 Shear strain 10%
7 Shear strain 15%
Consolidating stresses $\sigma_1/\sigma_3 = 2$
$\sigma_3 = 200$ kPa

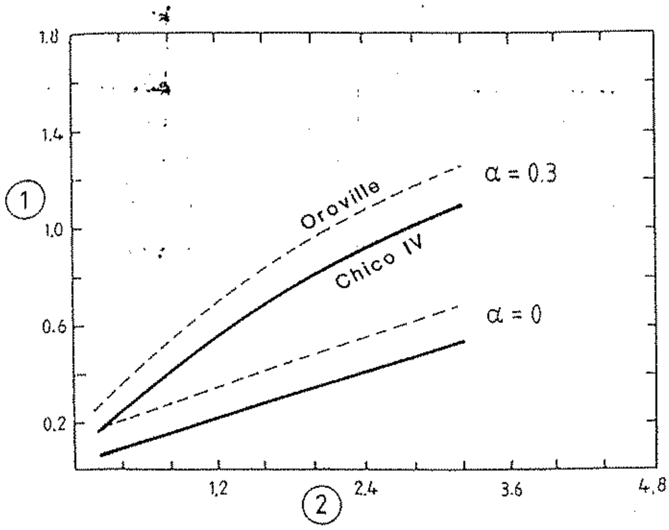


Figure 8.7. Residual axial strains from cyclic triaxial tests on Chico earth-rock material and on Oroville gravel (nos 1 and 6 in Table 4.10, respectively). Conditions for 5% residual axial strain in 6 cycles (archives LI).

1 Cyclic shear stress τ_{xy} (MPa)
2 Consolidating stress σ_1 (MPa)
$\alpha = \tau_{xy}/\sigma_1$

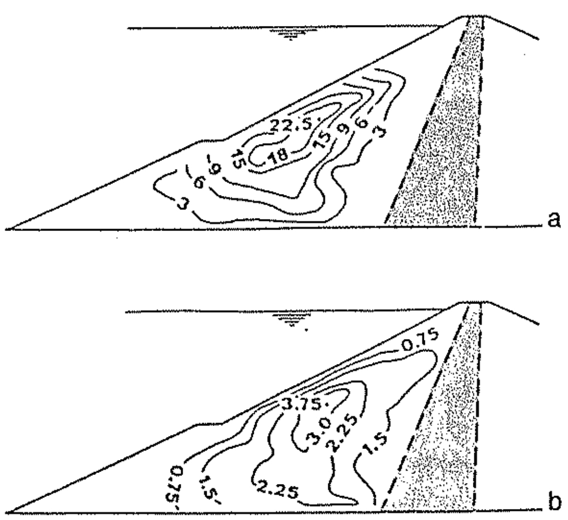


Figure 8.8. The Chico rockfill dam, upstream shell. Computed distribution of shear strains at full supply level and earthquake (archives LI).

a Chico earth-rock material
b Oroville gravel

sponsible for any selected strain level. As an example Figure 8.7 shows the conditions for 5% residual axial strain in 6 cycles of the Chico earth-rock material. It is compared to the same conditions of the lesser sensitive Oroville gravel (Table 4.10, no 6 and Fig. 4.32, no 9). The number of 6 cycles was selected as an equivalent number for the cyclic stresses of elements located at half the dam height. The figure points to the importance of cyclic tests for a dynamic analysis. The shear resistance of both materials increases with the initial vertical stress. At all stress levels the Oroville gravel is stiffer than the Chico earth-rock.

The resulting cyclic shear deformations can now be computed from the known stress and strain conditions of each element. Because of the non-linear behaviour of the material the computation must be made iteratively. The contours of the shear strain potential of the Chico and Oroville materials, if used in the upstream dam body, are shown in Figure 8.8. It is obvious that the sensitive earth-rock material of Chico cannot be used to build the saturated upstream shell. The deformations to be expected are excessive (22.5% strain). In contrast, a free-draining, well graded material, such as Oroville gravel or similar, presents satisfactory safety against unacceptable deformations (3.75% strain).

CHAPTER 9

Construction of earth and rockfill dams

9.1 RIVER DIVERSION

The work of river diversion marks the beginning of the construction period after the site has been made accessible by roads and after the first steps of site installation permit such work. Small river discharges are conducted across the site through pipes. Large discharges require cofferdams and different structures of the diversion system. Frequently, river diversion is contracted ahead of the main works as an individual contract.

An upstream main cofferdam serves to retain the anticipated construction floods and to conduct the permanent river discharge and the construction floods to the diversion structures. This way, dewatering of the construction area is limited to precipitation and surface water from its catchment area. In particular, the dam foundation area must be dewatered until material dumping starts, after the necessary foundation preparation.

A downstream cofferdam serves to protect the construction area from inundation by tail water. This cofferdam is always lower than the upstream main cofferdam, because its height relates only to the maximum discharge through the diversion system.

The upstream main cofferdam requires a sealing element. The construction of its lower part needs dry conditions for material dumping. Therefore, an auxiliary cofferdam (or pre-cofferdam) is necessary to protect the foundation area of the main cofferdam's sealing from inundation. Figure 4.1 shows this auxiliary cofferdam close downstream of the entrance to the diversion channel. The upstream main cofferdam is – in this example – incorporated in the main dam. The auxiliary cofferdam is much lower than the main cofferdam because it must function only over a short period. The lower part of the auxiliary cofferdam must be constructed to close the river in a period of low river discharge.

For large projects it is advisable to assess the risk of inundation of the site in the period from river closure until the completion of the main cofferdam.

With projects of significantly differing river discharges, for instance during tropical dry and wet seasons, river closure as the beginning of the diversion period is frequently on the critical path. Major parts of the construction process will be delayed if the components of the diversion system are not completed at the time of the minimum river discharge. The delay will last one year, with all the problems of delayed completion and financing. With respect to the risk of delay and related consequences Fetzer (1988a) concludes: 'The designer should preconstruct the cofferdams *on paper* based on a thorough knowledge of the site conditions, and should develop a time and materials chart for each step of the construction work'.

Embankment dams, separated from or incorporated into the main dam, are usually designed as cofferdams. Examples can be seen from Figures 7.13, 7.14 and 7.18. The river closure is usually effected by large blocks of rock, or precast concrete units, being end-dumped from both river banks into the water. The size of the blocks is selected with respect to the sweeping force of the water. As soon as this embankment reaches the water level and bulldozers can pass over it, rockfill, gravel, sand and finally cohesive material is dumped on the upstream side to establish a practically tight barrier across the river.

The other components of the diversion system may be channels and lateral tunnels through one of the abutments. Examples can be seen in Figures 4.1, 4.3 and 9.1. Concrete culverts through the dam and its foundation are also designed. Such a design must take account of settlements due to the dam load. The culvert must be tight over its lifetime. The problem of settle-

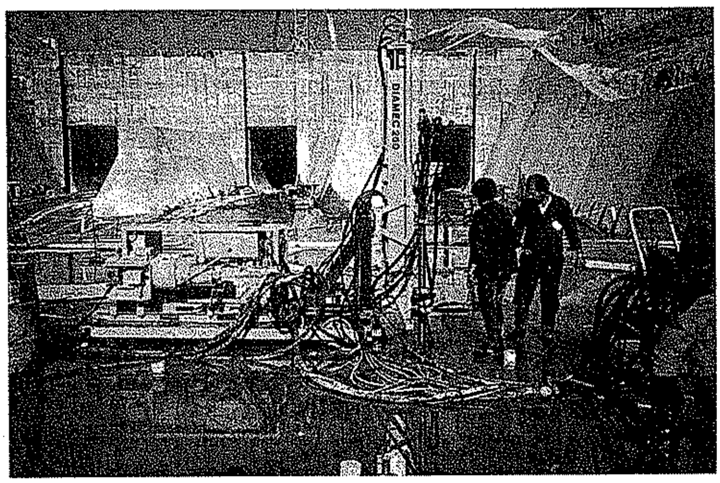


Figure 9.1. The Atatürk rockfill dam. Diversion outlets for about 8000 m³/s, seen from the stilling basin.

ments was addressed by, e.g., Rutledge & Gould (1973) and Rizzoli et al. (1991).

As an exception in embankment engineering, river diversion can be managed by narrowing the river in a first stage and diverting it along a part of its bed, while concrete structures are built in the other part, which is dewatered. In a second stage, the river is diverted through the concrete structures, for instance power intake and spillway. In this case a cofferdam is needed along the two parts parallel to the river. Such a cofferdam may be designed as an embankment or as a cellular cofferdam.

9.2 EXCAVATING AND QUARRYING OF NATURAL CONSTRUCTION MATERIALS

All the materials are collected according to common methods of earth and rock moving works. Due to the variety of materials and methods only a short overview is given here.

The cohesive materials are usually excavated in dry conditions (Figs 9.2 and 9.3). Excavators, wheel and track-type loaders and wheel tractor-scrapers are in use. Excavators and loaders transfer the material to dump trucks or bottom-dump cars which transport it to the embankment. Bottom-dump cars are suitable for gravel, cobbles and small-size rockfill. Scrapers are in use where the material is excavated from wide, flat deposits and where the road to the embankment is flat or almost flat. The loading capacity of trucks and scrapers is up to about 80 and 40 m^3, respectively.

The non-cohesive materials can be excavated in dry and wet conditions.

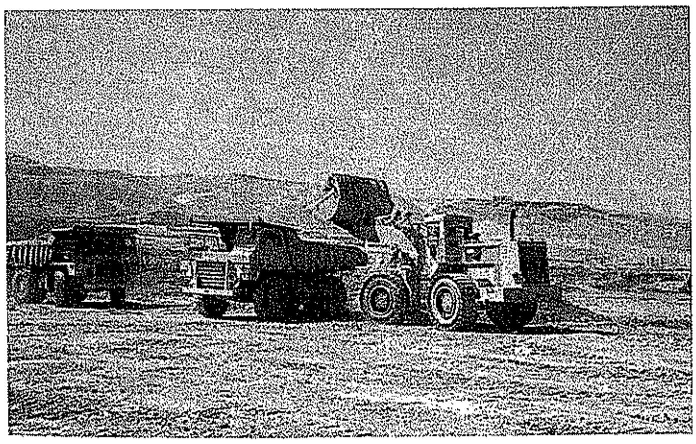


Figure 9.2. Earth moving work using wheel loader and dump trucks.

Figure 9.3. Excavating and hauling of cohesive materials using wheel tractor-scraper.

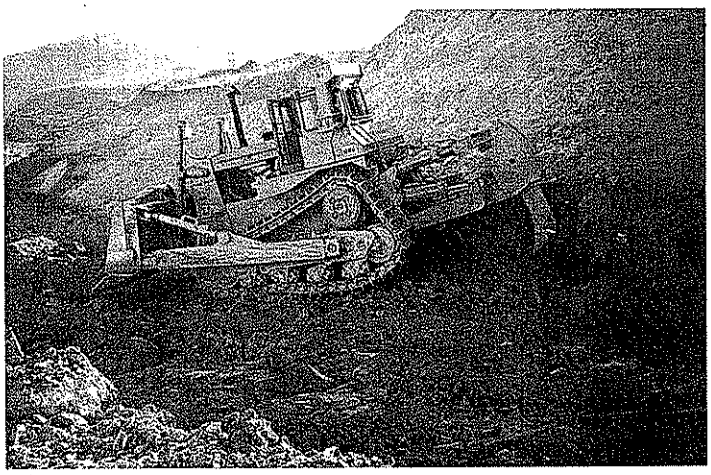


Figure 9.4. Ripping of weak rock using track-type tractor with ripper.

Common and dragline excavators and wheel and track-type loaders are in use. Draglines are mainly used for excavation under water. The construction materials for the dam shells are usually hauled directly to the embankment. In contrast, filter materials usually have to be transported to a processing plant.

Rockfill material is excavated or quarried in dry conditions. We distinguish rock to be ripped and rock to be quarried. Weak rock is loosened by rippers (Fig. 9.4), then loaded on trucks and brought to the embankment. Hard rock is loosened by blasting in quarries (Fig. 9.5). Prior to routine work

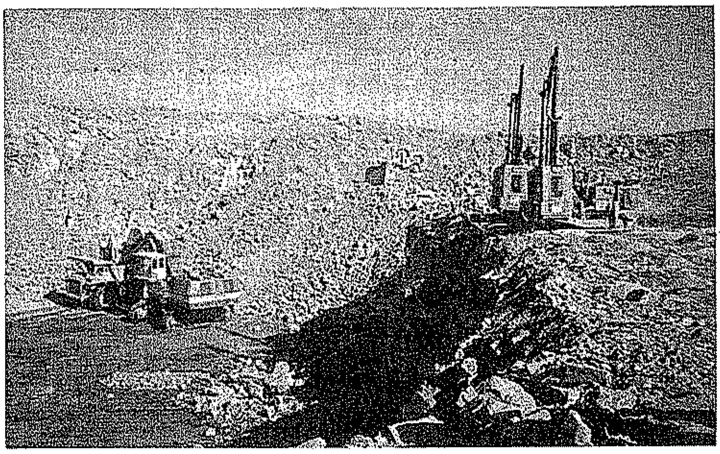


Figure 9.5. Rockfill quarry with drill rigs, excavator and dump truck.

blasting tests are performed to find appropriate blasting parameters (Section 4.2.6.1). The aim is to produce material according to the specification as to dump it 'quarry-run' in the embankment, without any further handling or processing. The blasted material in the quarry is transferred to dump trucks by excavators and loaders. Oversize material is separated for later use as riprap.

All the borrow areas must be stripped of rooted top soil and other unsuitable materials. The content of organic matters in construction materials must not exceed 1 or 2%. In most cases, recultivation of the exploited areas is required, by use of the stripped material or as otherwise specified.

9.3 PROCESSING OF NATURAL CONSTRUCTION MATERIALS

9.3.1 *Cohesive soils*

9.3.1.1 *Material moistening*

Cohesive soils, when used as dam sealing materials, must be dumped and compacted at a water content close to the optimum to achieve a maximum of unit weight and strength and a minimum of permeability. The soils must be moistened or dried out before being placed if the natural water content deviates too much from the optimum water content. An exception is the 'wet core' which is dealt with at the end of Section 7.3.2.1. The additional water must be regularly distributed in the soil mass to establish the same soil properties all over the material and hence homogeneity, within practicable limits.

The moistening can be performed on the embankment by a tank car moving on the dumped and levelled, but not yet compacted, layer (Fig. 9.6). A perforated pipe is attached to the rear of the car from which the water is sprayed over the layer. The width of the pipe is equal to the width of the compactor. The method is limited by the required quantity of water and by the time needed to penetrate the soil. Too much water will create a slippery surface on which the construction equipment cannot maneuver. Too long a penetration time will prevent homogeneous distribution and might delay the construction process.

According to experience the upper limit for the method is an increase of the water content by 2 to 3%. It looks attractive to spray more water, if required. But this is not advisable because the penetration will not lead to a satisfactory homogeneous soil mass.

An attempt to shorten the penetration time and to improve homogenizing was made by blending the water with a disc-harrow into the soil prior to compaction. Disc-harrows are known from agricultural techniques (Fig. 9.7). Prusza V. & Choudry report on tests where this method failed to achieve the desired homogeneity. Instead, water was concentrated locally in pockets, and other parts of the soil remained at the natural water content. The soil was a saprolite with about 25% clay < 0.002 mm, on an average.

As an alternative, the water was added to lifts of 15 cm thickness and two such lifts were compacted in one procedure. An example is described by De Cossio et al. (1982). The precondition is that the compaction is fully effective down to a depth of 30 cm plus the scarified superficial portion of the

Figure 9.6. Moistening of cohesive soil in the embankment, using tank car with perforated pipe moving on the uncompacted, levelled layer.

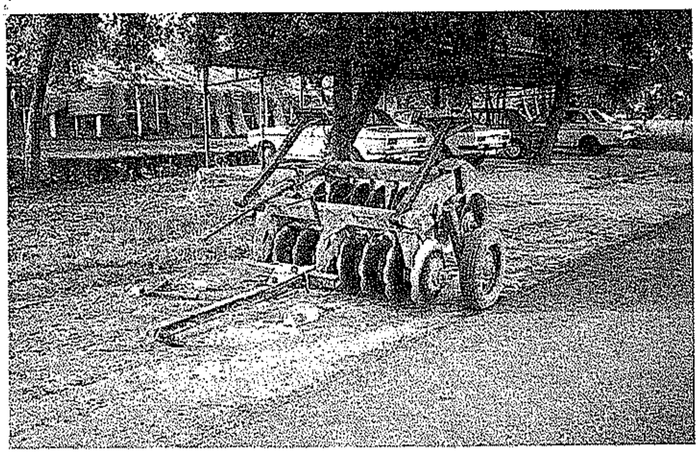


Figure 9.7. Disc-harrow.

layer underneath. Again, the upper limit of additional water will be 2 or 3% in terms of increased water content.

In case the water to be added exceeds this upper limit, the soil must be moistened in the borrow area or in an intermediate deposit. The simplest way to water the loosened soil is by the use of hoses. Homogeneity must be achieved by repeatedly moving and turning the material prior to placing it on the embankment. The correct quantity of added water and the final water content of the soil must be estimated on site by visual inspection and later be confirmed by tests. In many cases the site supervisors will be able – after some days experience – to estimate the water content to a reasonable accuracy.

The procedure can be improved by placing the material in an intermediate deposit. There the soil is dumped in layers, the layers being individually moistened and homogenized by disc-harrow. After a waiting time for reasonable penetration, the material is hauled to the embankment, placed and compacted. The above mentioned report of Prusza V. & Choudry (1982) includes such a procedure. The saprolite was dumped in layers to a final height of 4 m. The area of the intermediate deposit was 200 m × 400 m. The resulting production rate relates to the quantity of water to be blended, to the layer thickness and to the equipment employed.

Another method of moistening material is executed between the borrow area and the embankment as shown in Figure 9.8. The material slides down a ramp constructed for this purpose. On its way from top to bottom water is sprayed onto the soil from monitors to achieve the desired water content. The processes of loading at the toe of the ramp and spreading and levelling on the embankment result in a well homogenized soil. The method leads to

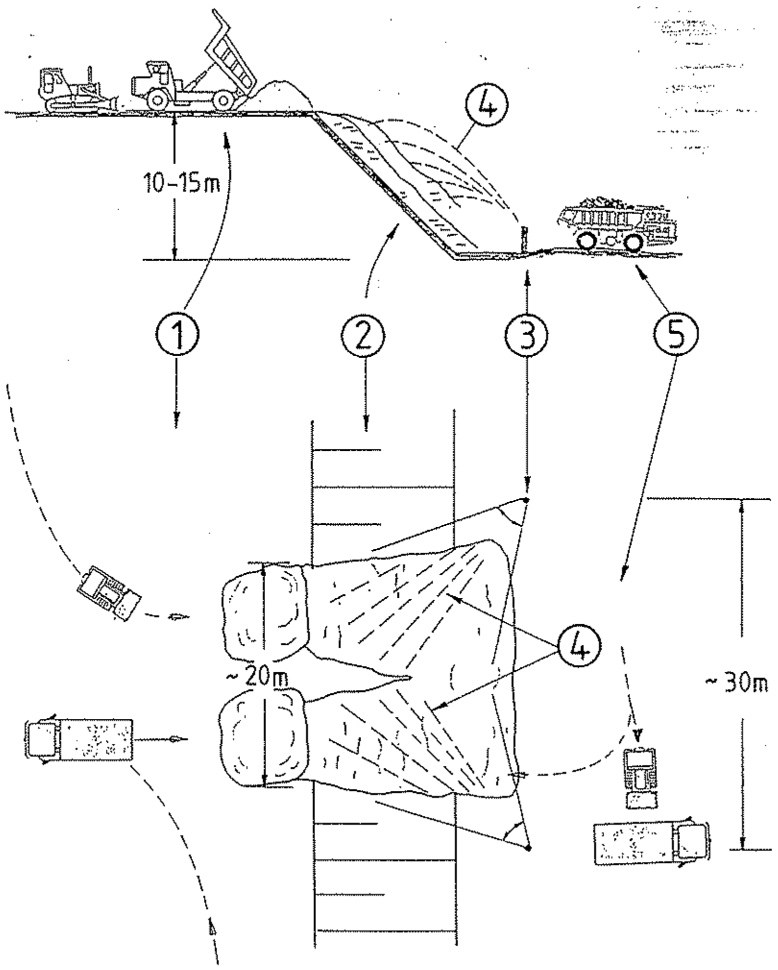


Figure 9.8. Moistening of cohesive soil between borrow area and embankment, using monitors.

1 Delivery of dry soil
2 Ramp
3 Monitors (spray towers)
4 Sprayed water
5 Hauling of moistened soil

high production rates provided the number of monitors and the water quantities are properly selected. Again, the supervisors will estimate the water content by visual inspection, with subsequent confirmation by tests.

Proven large-area methods to moisten the material in the borrow area are sprinkling and infiltration. Both methods require long waiting times until a reasonable volume of the borrow area is wetted. Both methods lead to high production rates once the moistening process is satisfactorily completed.

Prusza V. & Choudry (1982) give an instructive example for sprinkling, including how the correct water quantity and time of sprinkling can be calculated. An amount of 57×10^6 m^3 of soil had to be moved for the heightening of the existing Guri dam. A saprolite as a weathering product of gneiss was available. The clay content of the material is reported to be in the range of 12 to 25%; the plasticity index is 10 to 25. The natural water content was 8 to 9% below the optimum water content, which is 15 to 20%.

The system of sprinklers as shown in Figure 9.9 was designed for a large-scale test made in an almost flat area. The essential parameters are:

- Grid of sprinklers 12 m × 18 m,
- artificial precipitation 7.19 mm/h,
- area to be sprinkled 2.07 ha,
- quantity of sprinkled water ≈ 150 m^3/h.

For sloping areas the artificial precipitation must be reduced to limit the surface run-off. The sprinkling time can be estimated from the desired penetration depth and the permeability of the soil in situ. After Darcy's law it is

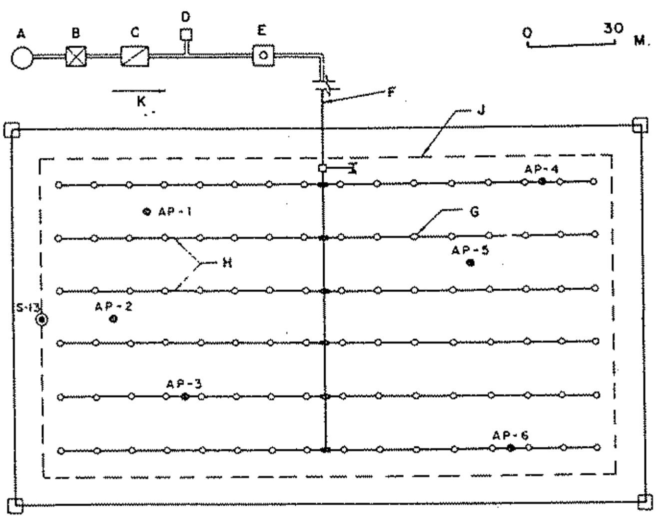


Figure 9.9. Moistening of cohesive soil in the borrow area. Layout of sprinkling test (Prusza V. & Choudry 1982).

A Pump	H Laterals
B, C, D Controlling valves	I Reducer
E Flow meter	J Boundary of the sprinkling area
F Mainline	K Flow
G Sprinkler	AP, S Exploratory holes

$$T = q/k \cdot i \cdot A \tag{9.1}$$

where: T = sprinkling time (s)
q = total quantity of water to be added (m^3)
k = permeability (m/s)
i = hydraulic gradient = 1 for vertical flow with no water head above ground level
A = sprinkled area (m^2).

Given the dry unit weight of the soil as 15 kN/m^3, the addition of around 15 l = 0.015 m^3 of water per each m^3 of soil leads to an increase of the water content by 1%. At $k = 2 \times 10^{-6}$ m/s the sprinkling time to moisten the soil down to 1 m depth is about 7500 s (2.1 h). The authors set a reduction factor of 0.75 to account for evaporation and system efficiency whereby the sprinkling time to achieve a one per cent increase in the water content calculates as T = 2.78 h at an assumed penetration depth of 1.0 m. For 20 m depth and 8% water content increase, the sprinkling time will be around 450 h.

The large-scale test revealed a moisture increase of 12 to 13% after 456 h (19 days) of sprinkling time and an additional waiting period of 4 to 5 days. The penetration depth was 12 to 15 m only, instead of the calculated 20 m, probably because of lower permeability in places.

Prousza V. & Choudry classify this method of soil moistening in the borrow area as the most economical method given the conditions of large quantities to be added and high production rates of moistening to be achieved. In the example given around 2×10^6 m^3 of soil have been wetted in one operation in a sprinkled area of 18 ha. The operation lasted for less than one month. Because of the inevitable variation of the permeability and the natural water content of the soils in the borrow area, it is recommended to verify the actual requirements of sprinkling time and artificial precipitation by a large-scale test, to be performed ahead of contractual work.

The moistening of soils in the borrow area by infiltration of water from trenches, pits and shafts is less elaborate than artificial precipitation. It is more tentative. An advantage is the small rate of water loss due to evaporation. The depth of the trenches is limited. The spacing of the trenches and the necessary sequence of filling them with water should be investigated, using tests showing the penetration depth and the infiltration time until the desired water content is achieved.

A report of Zahaf et al. (1982) describes an example where the water content of clayey, silty sand (SC) and sandy, silty clay (CL) has been raised by 4 to 5% by using infiltration trenches. The depth and width of the trenches was 1.0 m and 1.0 to 1.5 m, respectively, at a spacing of 4 to 5 m. Given the rather unfavourable conditions of the CL-material, the waiting time was approximately one month until the water content was satisfactorily increased down to a depth of 3 to 4 m. The effective depth was 10 m and more with the SC-material. For both soils the method proved to be very successful.

9.3.1.2 *Material drying*

In practice the following methods are in use to dry off materials in order to reduce the water content:

– Drying in the open air,
– addition of soils of low water content, and
– drying in a furnace.

Drying in the open air is appropriate in tropical areas of low precipitation. The wet soil is spread in thin layers to be exposed to the sun's rays. It may be necessary to turn the material repeatedly. The method is simple, cheap and not time-consuming. Drying off takes hours or a few days as a maximum. As a disadvantage, double-handling of the material is required prior to its final placement in the embankment.

The addition of soils of low water content is more elaborate, particularly if the soil must be dried beforehand. An example is described in the next section (Hammamji et al. 1982).

The first experiences in using furnaces to dry core materials off occurred in Austria in the 1960s with the projects Durlassboden and Gepatsch. In both cases the water content of the treated core material was reduced by 3% (Gschaider & Schlosser 1968, Neuhauser 1970).

A recent example from Southern Italy is described by Baldovin et al. (1991). Drying the core material of the Castagnara dam was one part of the material processing. In addition, bentonite was added. This is discussed in Section 9.3.1.4. The cylindrical, rotating furnace is shown in Figure 9.10.

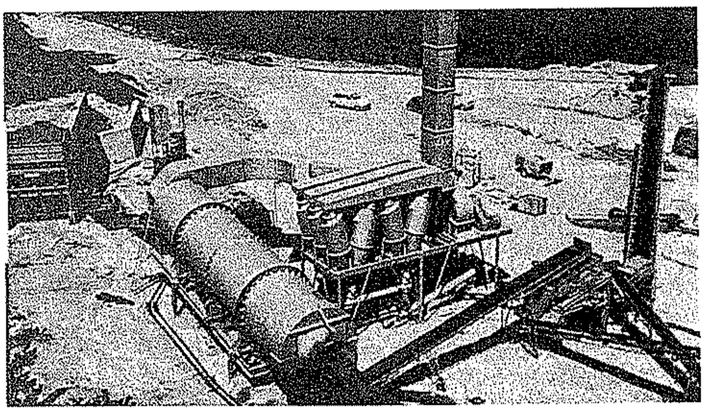


Figure 9.10. Furnace in the core material's drying and mixing plant, output up to 1000 m^3/h (Baldovin et al. 1991).

The rotational speed can be adjusted to the initial water content of the soil. The material is supplied by conveyor belts on which lumps of clay can be broken down. The capacity of the plant is 800 to 1000 m^3/day. The natural water content of 25 to 27% was reduced to 12 to 13% which corresponds to the optimum water content at the modified Proctor compaction.

9.3.1.3 *Material blending with sand and gravel*

It is occasionally useful to blend cohesive material with sand and gravel to achieve the proper conditions for compaction and performance. This can be done by building a sandwich-like deposit consisting of alternating layers of the original and the additional material. The layer thickness corresponds to the mix proportions. The deposit is exploited by scraping the material from an almost vertical slope. Handling the material by scraping, loading onto trucks and levelling on the embankment results in the required mixing and homogenizing.

An exceptional case of mechanical mixing at the Outarde 2 project in Canada is reported by Hammamji et al. (1982). The task was to blend a marine clay with a gravelley sand. The clay was too weak to be placed in a sandwich deposit. The natural water content was 8% above the liquid limit.

The mean data of the original materials and of the mix and the compaction procedure are listed in Table 9.1. The ratio of sand/clay is about 3/1 by weight. Blending of the liquid clay with the previously furnace-dried sand

Table 9.1. Mean properties and compaction procedure of a mechanically produced mixture of very weak clay and sand (produced after Hammamji et al. 1982).

		Clay	Sand	Mixture
< 0.002 mm	(% by weight)	37	–	4 to 12
< 0.074 mm	(% by weight)	96	1.7	15 to 30
< 1.0 mm	(% by weight)	100	50	40 to 85
< 10 mm	(% by weight)	–	83	80 to 100
Natural water content	(%)	33.2	6.4	–
Optimum water content	(%)	–	–	7.6
Water content of sand after kiln drying	(%)	–	1.4	–
Fill water content	(%)	–	–	7.5
Liquid limit	(%)	25.0	–	–
Plasticity index		7.2	–	–
Permeability	(m/s)	4×10^{-10}	–	7.5×10^{-8}
Dry unit weight	(kN/m^3)	–	–	21.7
Layer thickness	(cm)	–	–	30
Compactor	Vibratory smooth drum roller, axle load 60 kN			
Number of passes				min. 4
Operating speed	(km/h)			max. 5

was executed in two mechanical mixers of 4.6 m^3 capacity each. The output of the plant was 150 m^3/h. More examples of mechanical mixing are described in the next section.

9.3.1.4 *Material blending with clay*

The properties of slightly cohesive soil with regard to their use as sealing material can be improved by adding clay. Usually bentonite is added, which is a swelling clay of montmorillonite with a water absorption potential of at least about 400%. By adding bentonite the material is made more plastic. That means that the fill water content can be increased and hence better adjusted to the requirements of appropriate compaction. The shear strength is decreased, but the tensile strain is increased. High plasticity and tensile strain are desirable properties with respect to differential deformations. The permeability is decreased, which is also desirable. The technique to modify favourably the properties of given soils has been used to improve low plastic soils for use as core material for dams. Typical examples from Germany are the dams at Sylvenstein (1958) and Mauthaus (1972).

The design of Sylvenstein (Fig. 9.43) demands particular attention, because of the irregular shape of the valley and the lack of natural core material (Beier et al. 1979, List & Strobl 1991). The available slightly clayey gravel was blended with clayey silt from a moraine, cement and bentonite. The content of particles below 0.06 mm in the blended material is only 13 to 22%, but the material's permeability is as low as 10^{-9} m/s. The considerable content of gravel > 2 mm of 50 to 60% results in a friction angle of 36°. The optimum water content is 8%, the fill water content is slightly more.

The addition of bentonite varies between 2 and 0% (Fig. 9.44), depending on the location of the material in the core directly above the abrupt change of the rock contour (2%), above the valley in the upper portion (1%) and in the lower portion (0%). Other details are discussed in Section 9.9.3.

The 61 m high Mauthaus dam is a rockfill dam with vertical earth core sloping 1V:0.2H on both sides. The core is surrounded by filter zones which act also as transition zones between the core and the shells. The center portion of the core was improved by blending the clayey and sandy gravel (GC) with 1% bentonite and 5% sand 0 to 2 mm (Fig. 9.11 and Table 9.2). The percentages refer to the dry unit weight of the base material which is a weathering product of clay slate and greywacke with a natural water content of 13.5%.

The mixing plant was composed of compartments for GC-material and sand, silos for bentonite, dosing and conveying devices and 2 mechanical mixers of 1.5 m^3 capacity each. The production rate was 1400 m^3 per 12 hours (Gebhardt 1971). The duration of the mixing process was at least 2 minutes. The mixing plant served also to process the inner transition material

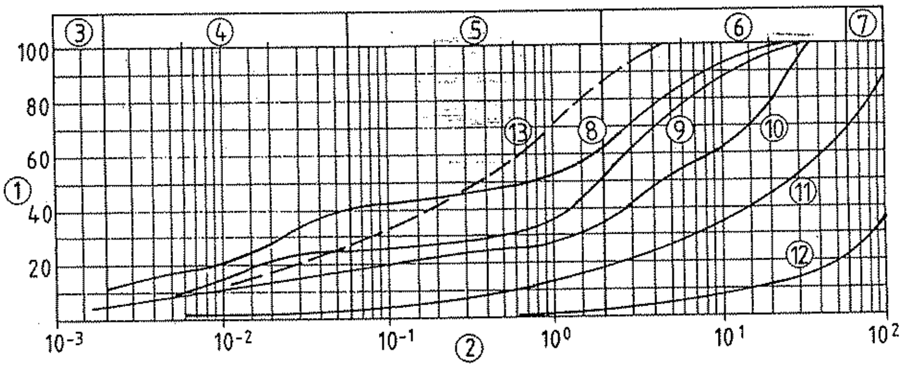


Figure 9.11. The Mauthaus rockfill dam. Mean gradations of the construction materials (adapted from Gebhardt 1971).

1 Percent finer by weight
2 Grain size (mm)
3 Clay
4 Silt
5 Sand
6 Gravel
7 Cobbles
8 Processed material for the center portion of the core
9 Natural core material
10 Processed material for the inner transition zone (fine filter)
11 Sandy gravel (coarse filter)
12 Rockfill material for the shells
13 Plastic concrete of the Förmitz dam (Lorenz & List 1976)

Table 9.2. The Mauthaus dam. Material properties and compaction procedure (produced after Gebhardt 1971).

Material	Dry unit weight (kN/m^3)	Friction (°)	Cohesion (kPa)	Permeability (m/s)	Fill water content (%)	Layer thickness uncompacted (cm)	Number of passes
Center of core: loam + 5% sand + 1% bentonite	19.7	30	9	10^{-9}	12 to 14	35	10[3]
Core: unprocessed parts	19.9	32	12	10^{-8}	12 to 14	35	10[3]
Fine filter: loam + 40% crushed gravel	21.0	33	12	10^{-7}	8 to 9	35	10[3]
Coarse filter: sandy gravel	19.6	33	–	10^{-5}	8 to 9	30	8[4]
Shell: rockfill	19.7	37 to 43	–	10^{-3} to 10^{-2}	u/s[1] d/s[2]	60 inner part 120 outer part	10[5]

[1]Addition of water 500 l/m^3
[2]Natural moisture
[3]Pneumatic roller 330 kN
[4]Static roller 125 kN
[5]Vibratory roller, static weight 125 kN

(fine filter) by adding 40% crushed gravel of 15 to 30 mm to the base GC-material (Fig. 9.11).

Lorenz (1973) reports on the properties of the original GC-material and the processed material. The best way of improving the material was investigated in the laboratory. It was found that the original material with a plasticity index of only 5 was very sensitive to the water content when being compacted. The material tends to become brittle when compacted on the dry side of optimum. In the same sense the initial permeability of 10^{-8} m/s increases rapidly. Accordingly, the minimum fill water content was set at 12%.

The addition of 1% bentonite and 5% sand 0 to 2 mm causes an increase of the plasticity index to 10 (instead of 5) and a decrease in permeability to 10^{-9} m/s. More bentonite made the mixing process difficult; and more sand led to an undesirable increase in permeability. The erosion stability of the processed material was found to be satisfactory. It was checked by permeability tests in the laboratory at hydraulic gradients up to $i = 32$. The sample size was 1.0 m in diameter and 1.5 m in height.

The most reasonable, property-dependent zoning of a dam can be approached by FE-computations. Therefore, the blending of base materials with bentonite and its appropriate placement in the dam offers promising techniques, depending necessarily on the available base materials. The variation of the deformation properties by adding bentonite is affected by the initial soil properties, by the type and by the amount of bentonite added. In every case laboratory tests are required to study the effects.

A recent example is described by Baldovin et al. (1991). A part of the core material of the 100 m high Castagnara dam in Southern Italy had to be improved because of the lack of other suitable material, of the high incidence of earthquake hazard and of the unfavourable climatic conditions for dam construction. The construction schedule required a daily placement of 1000 m^3 of core material during the dry season. The wet season from November to April was excluded from dam construction because of 1800 mm average precipitation, essentially concentrated in the wet season.

The available core material was taken from very heterogeneous deposits of alternating silty and clayey sands, clayey silts and silty-gravelley sands. It had to be homogenized in the course of excavation by a mixing plant, to achieve the material shown in Figure 9.12. Final homogenizing was done by a rotating furnace (Fig. 9.10). Subsequent to the drying process the material was further improved by the addition of bentonite in different amounts. The liquid limit of the bentonite was $\geq 400\%$.

The zoning of the improved materials in the dam is shown in Figure 9.13. The material with 2.5% bentonite was placed on the foundation where great shear stresses are to be expected. Selected zones of increased stresses – according to computations – were built of material with 1.3% bentonite. In the

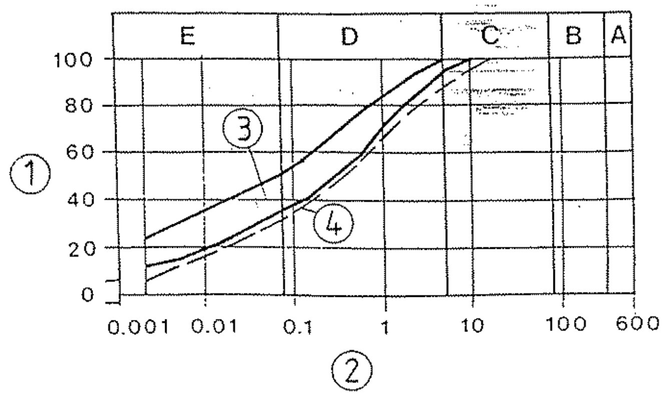


Figure 9.12. Processed core materials.

A, B Cobbles
C Gravel
D Sand
E Silt and clay
1 Percent finer by weight
2 Grain size (mm)
3 Range of Castagnara material (Baldovin et al. 1991)
4 Mean gradation of Sugar Pine material (adapted from Davidson 1982)

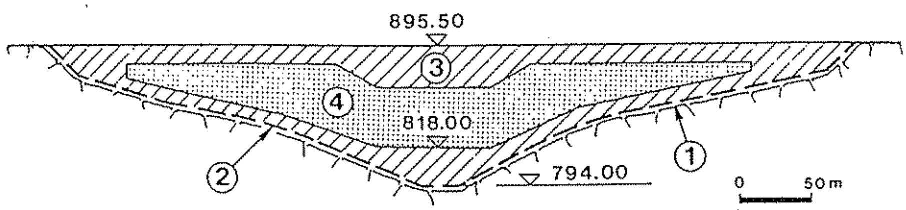


Figure 9.13. The Castagnara rockfill dam. Core zones with bentonite addition (Baldovin et al. 1991).

1 Core foundation
2 Base material plus 2.5% bentonite
3 Base material plus 1.3% bentonite
4 Base material occasionally mixed with 1.0% bentonite

center portion improved material was used only occasionally if the silt content of the base material was less than 30% or its plasticity index less than 10.

The effect of bentonite on the soil properties can be seen from Table 9.3. The increase of the plasticity index is noted. It makes about 4 units per 1% bentonite. The liquid limit and the failure strain increase in the same sense. The permeability is in the order of 10^{-9} m/s, as desired.

Table 9.3. Mean properties of clayey, silty sand after addition of bentonite (produced after Baldovin et al. 1991).

Bentonite	(% by weight)	0	1	2	3
Liquid limit	(%)	40	45	48	52
Plasticity index		14 to 18	22	25	28
Natural water content	(%)	25 to 27			
Optimum water content	(%)	12 to 13			
Modified Proctor density	(kN/m^3)	19.4			
Angle of friction	(°)	30			30
Undrained cohesion	(kPa)	50	55	60	62
Uniaxial compressive strength:					
– non-saturated	(kPa)	300	350	400	450
– saturated	(kPa)	75			80
Failure strain:					
– non-saturated	(%)	2.1			2.6
– saturated	(%)	2.0			3.2
Permeability	(m/s)	4×10^{-9}	2×10^{-9}	10^{-9}	6×10^{-10}

Similar effects have been found by Davidson (1982) with the addition of up to 10% Na-bentonite to a well graded silty sand (Fig. 9.12). The effect on the shear strength is noted. In CU-triaxial tests the friction angle was found as 34.5° (no bentonite) against 29.2° (5% bentonite). The respective values of cohesion were 21 and 16 kPa.

9.3.2 *Non-cohesive soils*

The non-cohesive soils for dam shells usually do not need processing. Only the rooted topsoil must be removed from the borrow areas prior to the excavation of construction materials.

Filter materials are treated like concrete aggregates. They are washed, screened and newly mixed according to specified gradations. A simplified method was applied at the Kinda site where the well graded filter had to be composed from three poorly to moderately graded base materials (Fig. 9.14). The three materials were placed by wheel loaders into three pyramidal containers (Fig. 9.15). Through the outlets at the bottom of the pyramids the materials poured onto a belt conveying them to a dump. Dosing was done by the outlets. Vibrators, attached to the containers, eased the continuous flow of the moist material. Mixing was subsequently achieved by the dumping, then the loading to trucks and finally placing in the embankment. The effectiveness of the procedure must be checked continually by sieve analyses.

Non-cohesive soils are also used for the construction of non-plastic dam cores. An example: a 200 m high dam in Lesotho had to be designed – in the feasibility phase – as a dam with a non-plastic core of processed material.

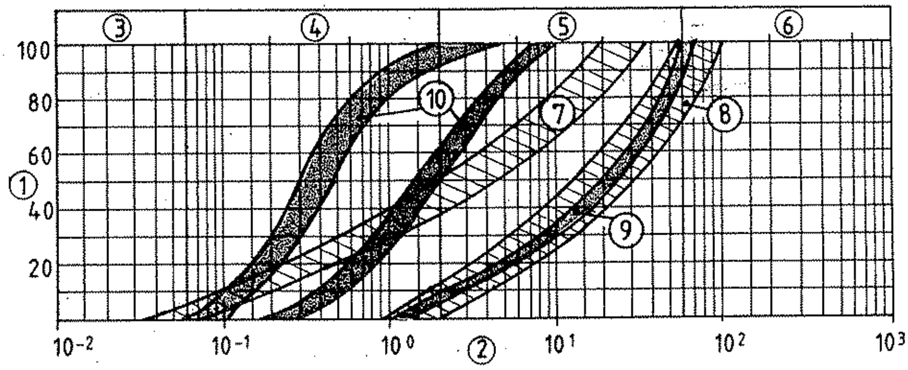


Figure 9.14. The Kinda rockfill dam. Gradations of specified filters and their original materials.

1 Percent finer by weight
2 Grain size (mm)
3 Silt
4 Sand
5 Gravel
6 Cobbles
7 Fine filter
8 Coarse filter
9 Tunnel muck
10 Sand and sand-gravel from borrow areas

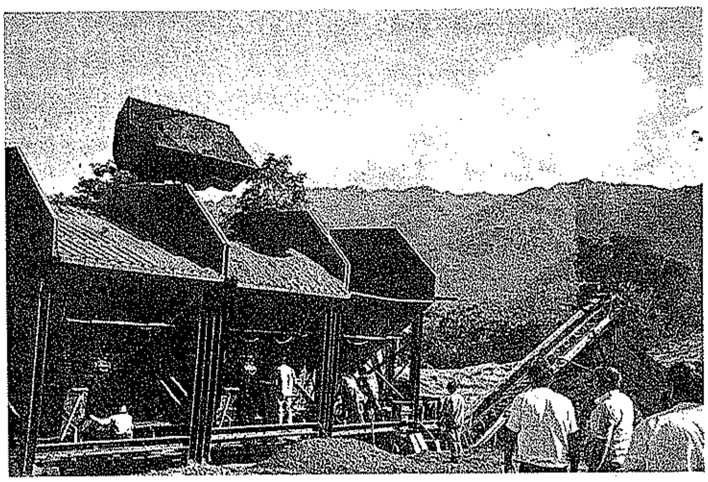


Figure 9.15. The Kinda rockfill dam. Filter processing out of 3 materials (courtesy of LI).

The potential material was a well graded sand of basaltic origin with 10% silt and 50% gravel. The idea was to screen the particles > 50 mm, to crush them down to particles ≤ 20 mm and to add the output of the crusher to the base material. It was found by tests that the crushed material had enough fines – originating from the crushing process – to increase the total content of non-plastic fines ≤ 0.06 mm of the final material to 15 to 20%. The permeability $k \leq 10^{-6}$ m/s was seen as satisfactory.

9.3.3 *Rockfill materials*

The rockfill materials are commonly dumped 'quarry-run', i.e. with no processing. It is the definite target of the quarry operation to deliver the material according to the specifications. Individual overlarge blocks can be broken in the quarry or on the embankment, for instance by pneumatic hammer.

There are exceptional cases where the quarries do not deliver the material as desired. Then additional breakage is required. A typical case is reported by Baldovin et al. (1991) of Castagnara dam. The rockfill material had to be produced by breaking and sieving oversize conglomerates. The mean gradation of this material corresponds to a moderately graded rockfill with $d_{10} \approx 25$ mm and $d_{max} \approx 300$ mm.

9.4 PLACING AND COMPACTING OF NATURAL CONSTRUCTION MATERIALS

9.4.1 *Cohesive soils*

Cohesive soils usually form the sealing element of embankment dams. Therefore, permeability is the most important property. Minimum permeability is obtained by compacting the material to the densest possible state. This is achieved by placing the material in fairly thin layers, with a favourable fill water content, and compacting it by heavy equipment. Attention must be given to the homogeneity of the embankment to avoid considerable differences in horizontal and vertical permeabilities, i.e. to minimize anisotropy.

The relation of fill water content and obtainable density is discussed in Section 4.3.1.3 and demonstrated in Figure 4.21. It is worldwide practice to take the results of the standard *or* the modified laboratory compaction test (Proctor test) as a measure of the most appropriate fill water content and the related dry unit weight in the field.

Standard procedures for compacting construction materials in the field are listed in Table 9.4. For cohesive soils the layer thickness vary usually within 20 and 30 cm. Wet materials require reduced lift thickness. The compacting machines are static or vibratory padfoot rollers as shown in Figures 9.16 and 9.17. Such rollers are also available as tow type rollers. The weight is up to 200 kN. The use of vibratory rollers to compact some types of cohesive soils reflects practical experience. An additional compacting effect of the vibra-

Table 9.4. Standard procedures for compacting construction materials in the field. (Large field tests are recommended to specify the details).

Material	Group symbol	Layer thickness uncompacted (cm)	Appropriate compactor[1]	Number of passes
Highly plastic clay	CH	15 to 20	SPR	6 to 10
Plastic silt	MH	20 to 25	SPR	4 to 8
Low plastic clay	CL	20 to 30	SPR, VPR	4 to 8
Low plastic silt	ML	20 to 30	SPR, VPR	4 to 6
Clayey sand	SC	20 to 30	SPR, VPR	4 to 8
Silty sand	SM	20 to 30	SPR, VPR	4 to 6
Sand and sand-gravel, poorly graded	SP	30 to 50	VSDR	4 to 6
			PR	6 to 8
Sand and sand-gravel, well graded	SW	40 to 60	VSDR	4 to 6
			PR	6 to 8
Clayey gravel	GC	20 to 30	SSDR, VPR, PR	6 to 8
Silty gravel	GM	30 to 40	SSDR, VPR, PR	6 to 8
Gravel, poorly graded	GP	40 to 50	VSDR	4 to 6
Gravel, well graded	GW	50 to 60	VSDR	4 to 6
Rockfill		60 to 150	VSDR	4 to 6

[1]The average weight of all compactors is 100 to 150 kN. The heaviest available compactor should be used. Average operating speed is 5 km/h.
SSDR = Static smooth drum roller
VSDR = Vibratory smooth drum roller
SPR = Static padfoot roller
VPR = Vibratory padfoot roller
PR = Pneumatic tired roller

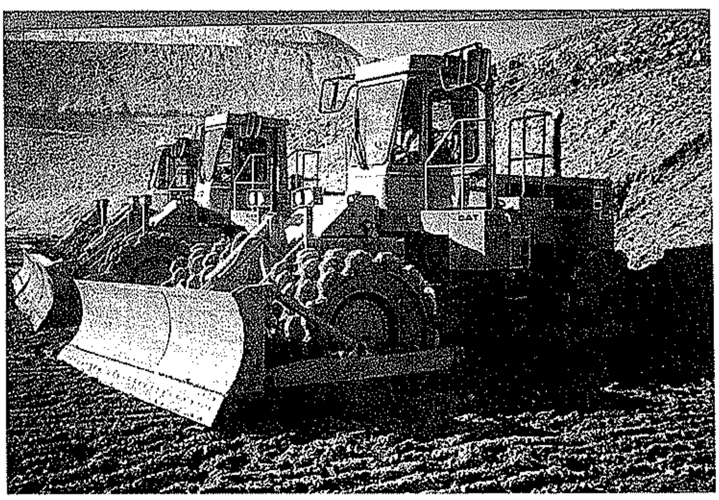


Figure 9.16. Static padfoot roller with dozer blade for the compaction of cohesive soils.

Figure 9.17. Vibratory padfoot roller for the compaction of cohesive soils.

tions cannot be explained by the conventional theory of vibratory compaction.

Pneumatic tired rollers are also in use (Fig. 9.18). The weight of such rollers is up to 300 kN (formerly up to 600 kN). They are mainly used for the compaction of asphaltic layers in road construction. In earth work they are appropriate to compact cohesive soils containing a considerable amount of gravel and cobbles which are an obstacle for padfoot rollers. They are not useful for compacting highly plastic soils, since such soils tend to fill the space between the wheels and to block their oscillating movement.

The most reasonable fill water content is close to the optimum moisture content (OMC) with reference to the standard *or* to the modified Proctor test, depending on the 'philosophy' of the geotechnicians involved. For practical reasons a margin must be left between the permissible maximum and minimum fill water contents. Frequently the boundaries are 'optimum minus 2%' and 'optimum plus 2%'. It is preferable to place the materials 'wet of optimum', because of the favorable deformation behaviour of plastic soils and because of the positive effect of pore-water pressure. Accordingly, the specified boundaries of the fill water content should be 'optimum' and 'optimum plus 2%'. For the same reasons it is preferable – in the opinion of the author – to specify the fill water content with reference to the optimum moisture content (OMC) of the standard compaction test since this OMC is slightly higher than that of the modified compaction test. The effect of pore-water pressure is discussed in Section 7.3.2.1.

It is a particular task to create the proper bond between subsequent lifts. The water content of the new layer and that of the surface underneath should

Figure 9.18. Pneumatic tired roller for the compaction of slightly cohesive soils (courtesy of Dynapac).

be the same or almost the same. It may be necessary to moisten the surface of the last layer. Penetration of the sprinkled water is eased by scarifying the surface. Such roughening, with or without moistening, will affect the bond favourably. Self-propelled vehicles with ripping teeth are in use for this work (Fig. 9.19). The loosening depth should not exceed about 10 cm. It must be ensured that the following compaction process will cover the thickness of the new layer, including the roughening depth. The quality of the bond is checked by infiltration tests in boreholes or – better – in excavated pits (see Section 4.2.6.2).

The procedures for compacting cohesive soils at the interfaces with the foundation and with structures differ from the above described methods. The material is placed with an increased water content (Figs 7.21 and 7.55). At the abutment slopes the compactors will not be fully effective. It is a proven technique to press the material onto the slope with the aid of available machinery, as shown in Figure 9.20. Also the levelling bulldozer will help in creating a proper bond by maneuvering as high up the slope as possible.

An exception arises in making the wet cores described at the end of Section 7.3.2.1. The lift thickness will not exceed 10 or 15 cm. Special low ground pressure (LGP) machinery is employed for material levelling and compaction.

In places which are not easy accessible, e.g. at the bottom of trenches or

Figure 9.19. Ripper for the roughening of soil surfaces.

Figure 9.20. Dumping and pressing of core material onto inclined rock slopes (courtesy of LI).

close to small adjoining structures, compacting equipment is the same as that used in road and sewer construction, such as mechanical tampers, tamping beams, mini-rollers and similar.

9.4.2 *Non-cohesive soils*

9.4.2.1 *Shell materials*

Non-cohesive soils are used to form the dam shells. The most important property is the shear strength, to guarantee stability. Maximum strength is attributed to maximum density. This is achieved by placing the material in layers and compacting it by heavy equipment. The fill water content – in contrast to cohesive soils – is of minor importance. Standard procedures of material compaction are listed in Table 9.4. Reasonable lift thickness of non-cohesive soils are 30 cm and more, increasing with the particle size. Appropriate equipment for compaction are smooth drum vibratory rollers and, occasionally, pneumatic tired rollers. The static weight of the compactors is up to 200 kN (frequently 120 to 150 kN).

A good bond between the layers is required to obtain homogeneity in respect of the strength. Usually, the natural roughness of the surface of the compacted layer provides a good bond. In doubtful cases additional scarifying may be applied.

The non-cohesive soils are usually placed at the natural moisture content. There are different opinions about the benefit of adding water to improve compaction. It is argued that the moist materials will have an apparent cohesion which might reduce the effect of compaction. When saturated the apparent cohesion is zero. Moreover, adding water would lead to a slight decrease of the friction between coarse particles. According to experience the capacity of modern compaction equipment is sufficient to overcome any negative influence of apparent cohesion and of natural friction between particles. In principle, vibratory compaction causes an initial short-term loosening of the particles, by which the internal friction is considerably reduced (Kutzner 1962).

Wherever water is added the quantity must be adjusted to the permeability of the material. The water must be completely drained away during the compaction process. Otherwise pore-water pressure may develop which is not desirable for non-cohesive soils. According to tests pore pressure was observed up to 50 m distance from the roller which, favourably, dissipated within a few minutes (Forssblad 1981). Proper compaction is affected negatively by puddles on the surface, to be seen from the surface irregularities and grooves in Figure 9.21.

9.4.2.2 *Filter materials*

The filter zones serve as erosion protection, as drainage and as transition zones. The last function applies to inclined and to vertical filters. They must bridge differential settlements of the adjoining zones. This can be achieved by not compacting the filters to the densest state. It is reasonable to aim at a relative density of 70 to 80% to establish the required deformation potential. This is done by compacting the neighbouring zones to their maximum density. The compactors must not move over the filter layers. In contrast, this does not apply to horizontal or near horizontal filters. They must be compacted to the densest state to minimize settlements of and inside the dam.

Usually, the filters are placed ahead of the neighbouring zones (Fig. 9.22).

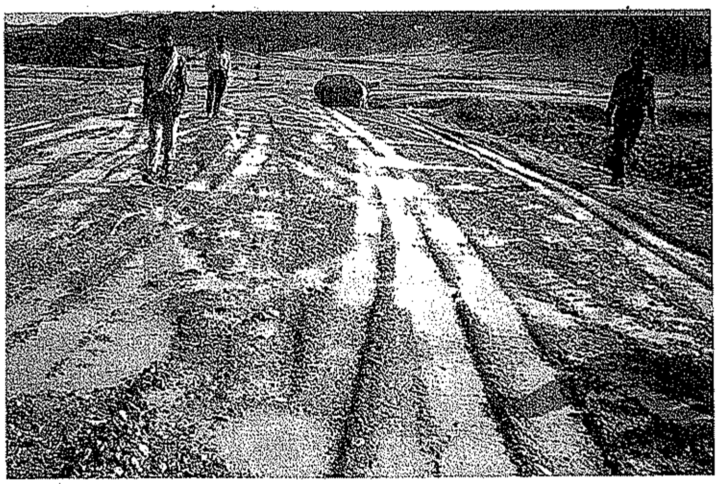


Figure 9.21. Placing of sand-gravel with addition of water. The amount of water is excessive; the compaction is not fully effective (courtesy of LI).

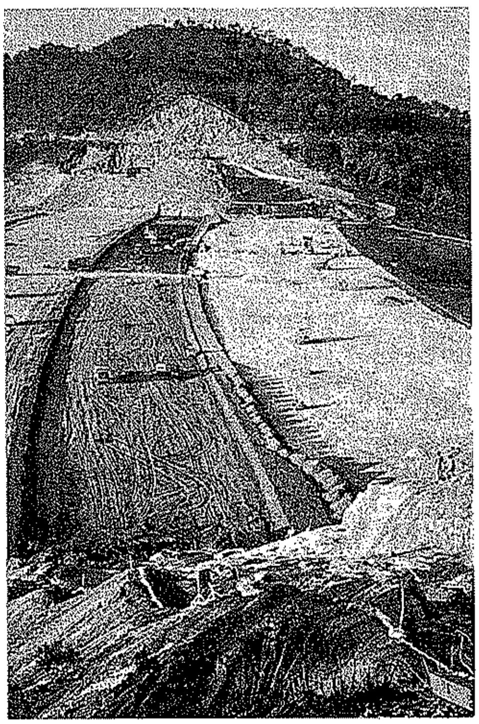


Figure 9.22. Placing of filter ahead of the neighbouring zones.

Foreground: Filter widening downstream close to the abutment (courtesy of LI).

Deleterious segregation of fine and coarse particles must be minimized, particularly at the interfaces with other zones (Fig. 9.23). Filters up to about 6 m in width can be placed using a spreader box (Fig. 9.24). The box is attached to the rear of the truck which carries the filter material. With the use of such boxes the lateral toothing of the filter, and hence the amount of material consumed, is limited.

With vertical chimney filters in homogeneous dams, the practice is to place the filter in a trench which is excavated after the embankment has been

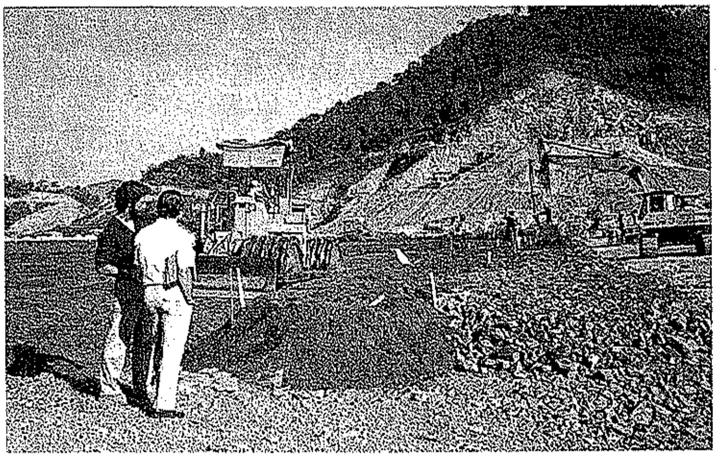


Figure 9.23. Filter dumped from trucks. Critical interface of fine and coarse filter. The excavator levels the surface (courtesy of LI).

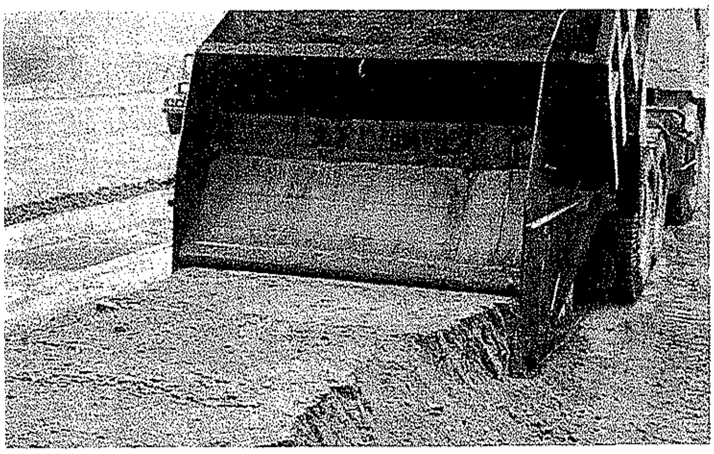


Figure 9.24. Placing of filter by spreader box (courtesy of LI).

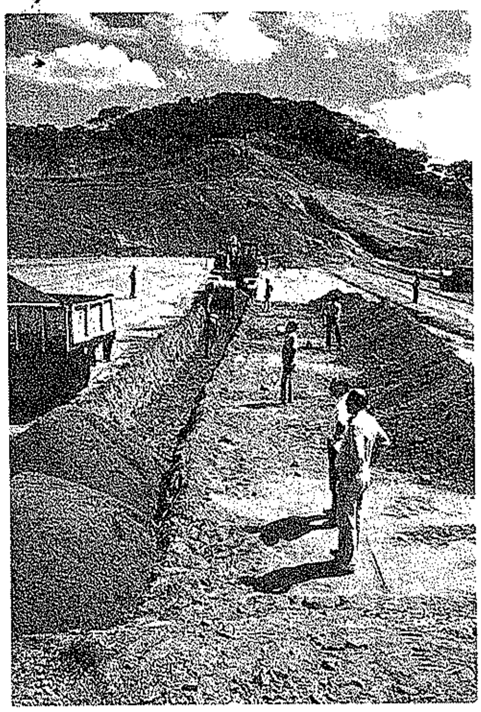


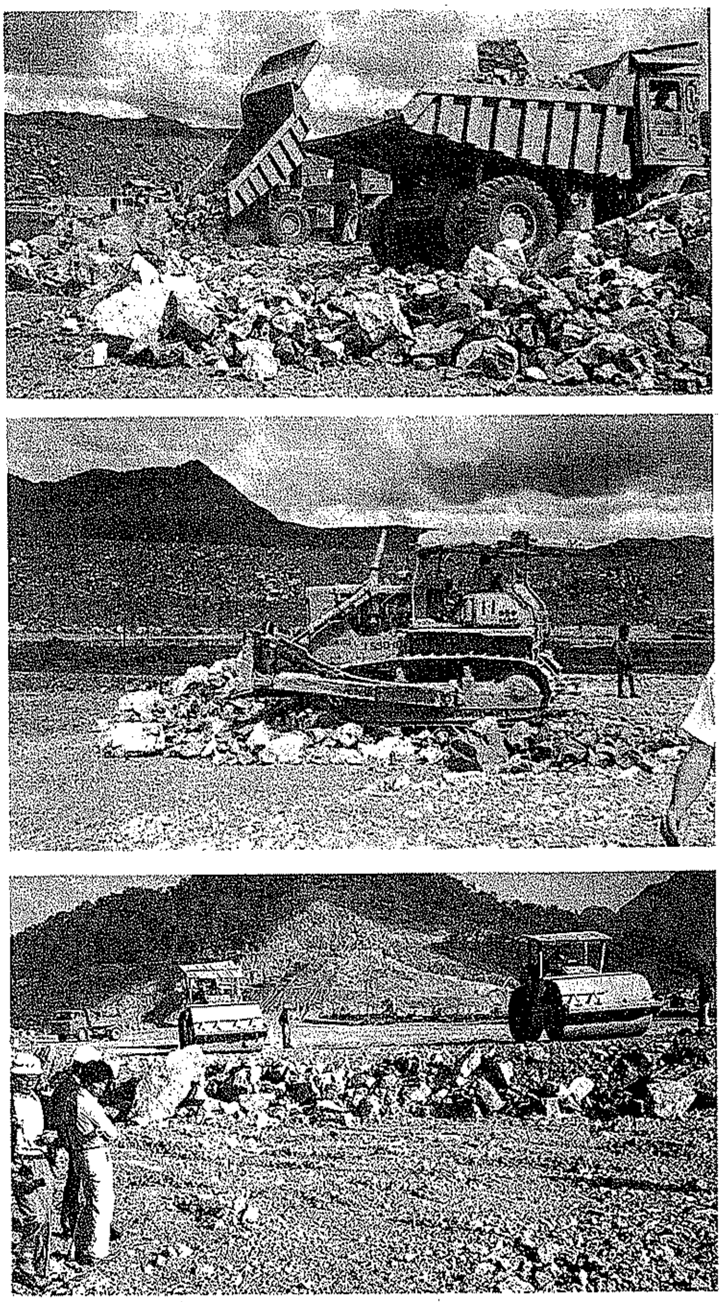
Figure 9.25. Placing of filter in a trench excavated after dumping and compacting several layers of the dam material without incorporating the filter (courtesy of LI).

heightened by several layers without incorporating the filter (Fig. 9.25). This procedure is repeated until the embankment has reached its final elevation. The method is suitable to minimize the amount of filter material required and to reduce unavoidable interference with construction work by placing the filter ahead of the adjoining zones. The filter can be kept as narrow as is permitted with respect to the discharge capacity. It must be ensured that the walls of the trench do not collapse before the filter is placed, and that remains of cohesive soil are left at the bottom of the trench. The filter must be isotropic, which needs equal permeability in vertical and horizontal directions.

On site, attention must be paid to keeping the filter clean. Contamination is caused by passing vehicles or by fines being washed onto the surface of the filter (trench method). Such contamination must be excluded, e.g. by preventing vehicles passing (filter placing ahead), or by covering the filter with sheeting in conditions of heavy rain and work breaks.

9.4.3 *Rockfill materials*

Rockfill material must be compacted to its maximum strength. Figure 9.26 demonstrates the working sequence of dumping, levelling and compacting.



a

b

c

Figure 9.26. Handling of rockfill material.

a Dumping
b Spreading and levelling
c Compacting by self-propelled vibratory smooth drum rollers

Lift thickness of 60 cm and more is appropriate (Table 9.4). The uncompacted layer thickness must be adjusted to the size of the largest pieces of rock. The compactors must not make irregular movements around blocks penetrating the surface which is being compacted. This would result in inhomogeneity around the block.

Only heavy vibratory smooth drum rollers are in use for compaction, of the self-moving or tow type. The weight of the rollers is up to about 200 kN (frequently 120 to 150 kN). Special rollers have been developed for the treatment of slopes. They are self-moving on flat slopes or operated by winches located on the dam crest. Such slope compactors are able to treat materials up to about 15 cm maximum size. That means, they are not capable of treating riprap and similar slope protective materials. They are indispensable for the construction of concrete face rockfill dams to achieve the required high deformation modulus and accurate geometry of the slope.

The wide gradation of rockfill cannot fully prevent segregation. It is usual that – to some extent – the larger stones will concentrate at the bottom and the gravel and sand at the top. This results in inhomogeneity concerning the permeability, the unit weight and the strength. The horizontal permeability may considerably exceed the vertical permeability. The unit weight may differ between extremes of 18.4 and 24.0 kN/m^3, as described by Leps (1988b) (Section 4.2.6.2b).

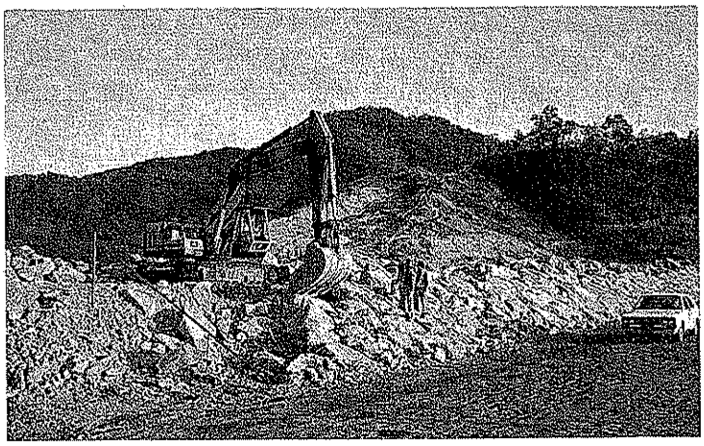
Such inhomogeneity can be accepted as long as the permeability does not fall below about 10^{-2} m/s if free-draining is required, and as long as the friction angle is not less than about 40° at low normal stress. This corresponds to a slope of 1V:1.4H given a satisfactory safety factor. The free-draining permeability relates to the characteristic d_{15} of about 1.3 mm, which is less than the common d_{15} of a good rockfill (A in Figure 4.28).

As to the addition of water to the rockfill, again different opinions exist. A convincing reason for adding water is given for rock with a noticeable potential to absorb water. The strength of such rock will differ according to the degree of saturation. In such cases, the addition of much water will force the material to breakdown under the attack of compaction. This way the expected breakdown and related settlements will occur during the compaction process instead of during first impounding. Breakdown is caused mainly by latent fissures and by edges that break.

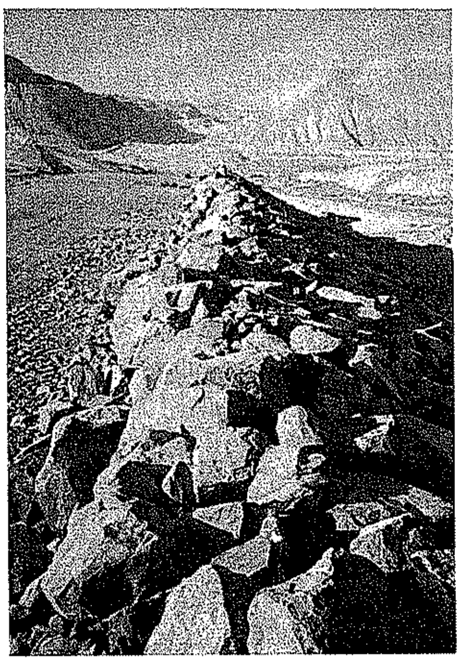
It should be kept in mind that large amounts of water are required to satisfy the demand of saturating those exposed edges which are likely to break. A considerable part of the water will be drained away before being effective. An average amount of water is about 20% by volume, i.e. 300 l per each m^2 of a 1.5 m thick lift. Fell et al. (1992) mention an excessive amount of 100% applied to layers of semi-impervious schist and – somewhat controversially – free-draining, fresh to moderately weathered gneiss. The procedure of adding water tends to wash fines to the lower portion of the layers.

The resulting difference in vertical and horizontal permeabilities should be seen in the light of the overall inhomogeneity of rockfill, which was discussed above.

Bulky material like riprap is not placed in layers. Either the blocks are arranged like a mosaic by an excavator, or the material is dumped by trucks before construction (Fig. 9.27). In developing countries, hand-placing is also



a



b

Figure 9.27. Placing of riprap.

a Placing by excavator like a mosaic after dam construction

b Dumping by trucks ahead of dam construction employed, which restricts the weight and size of the large pieces. With all the placing methods compaction is not possible. The smaller size pieces must fill the voids between large pieces, thus minimizing the deformability of the layer. The arrangement must prevent small pieces from being displaced by wave attack or being eroded by wave suction.

9.4.4 *Compaction under water*

Occasionally, dumping of construction materials under water is required. Non-cohesive, coarse soils and rockfill materials are suitable for that. Only these materials will ensure that an acceptable density is achieved and that the water is quickly drained away during compaction. The segregation of particles must be kept within acceptable limits. This results from dumping big charges in one operation, e.g. by the use of bottom-dump vessels. Compaction is performed from the surface as soon as the embankment emerges. Heavy vibratory rollers are in use. The effective depth is about 3 m, but a reduced effect was observed down to 6 m (Forssblad 1981).

The compaction of natural deposits under water can be done by deep vibro compaction down to depths up to 30 m. If the surface is above the water level, vibro consolidation by drop weight can also be applied (see Section 7.6.2).

9.5 QUALITY CONTROL

9.5.1 *Cohesive soils*

The targets of the compaction procedure, namely minimum permeability, maximum strength, and homogeneity, must continuously be controlled by checking related parameters. The check is occasionally made by geodetic survey of the layers before and after compaction, but this is not satisfactory. Such a survey will help to check the uncompacted layer thickness. With respect to the compaction effect it can be used as an additional control to cross-check the complete parameter control.

The parameters to be controlled are the fill water content and related dry unit weight of the compacted material and the bond between the layers. Water content and unit weight are controlled by laboratory tests on undisturbed samples and by alternative methods in situ. The bond is checked visually in trenches or with the aid of percolation tests (Section 4.2.6.2.a).

Undisturbed sampling sites must be regularly distributed over the embankment to give evidence of its homogeneity. The number of samples must be selected accordingly. Given high production rates, one sample should be taken per 1000 to 3000 m^3 of compacted material. Given low production rates, the number of samples will be greater. Sampling must be adjusted to the weather conditions if compaction is performed in varying – wet and dry – weather.

Recommended densities according to USBR (1974) are given in Table 9.5. Due to improved compaction techniques the author prefers to recommend 100% Standard Proctor density on an average. Minimum permissible density should be 98% Standard Proctor density. Each sample of less than 100% density must be accompanied by a sample of more than 100% density to satisfy the specified average of 100%.

The laboratory compaction test, being the measuring line, must be made at least once every day for each borrow area. The number of tests must be increased if there is an obvious variation in soil properties. The compaction test must be supplemented by the determination of the grain size distribution and the Atterberg limits. The dispersivity, the shear strength, the deformation behaviour under uniaxial or triaxial stress conditions, and the permeability, are to be checked according to the variation in the material properties.

The laboratory tests for water content and dry unit weight will take 2 days to carry out. Under conditions of high production rates on site the results will be available only at a time when the request for removal or improvement of a critical layer will meet great difficulties in practice. It was therefore attempted to avoid the time-consuming laboratory determination of the water content and to determine the material's water content and subsequently its dry unit weight by other means. One method of determining moisture content is the carbide bomb moisture meter, which is less accurate than laboratory determination (Section 4.2.4.2).

A rapid method of compaction control was developed by Hilf (1961). The

Table 9.5. Recommended degree of compaction of embankment materials (adapted from USBR 1974).

Material	> 5 mm (% by weight)	Minimum (%)[3]	Desirable average (%)[3]	Water content (% ± w_{opt})
Cohesive soils[1]	0 to 25	98	100	0 to + 2
	26 to 50	95	98	0 to + 2
	> 50	93	95	0 to + 2
Non-cohesive soils[2]	Fine sand with 0 to 25	75	90	Very wet
	Medium sand with 0 to 25	70	85	Very wet
	Coarse sand and gravel with 0 to 100	65	80	Very wet

[1]With reference to Standard Proctor density
[2]With reference to relative density (Equation 4.4)
[3]Percent of Standard Proctor and relative density, respectively

sample taken from the field is repeatedly subjected to a small amount of additional moistening or slight drying out. It is then compacted again in the Proctor mould whereby the relation of different moisture conditions and respective moist unit weights is obtained. Further evaluation is made with the aid of form sheets, where standard curves for soils in the normal density range are plotted. The procedure yields the ratio of fill dry density to laboratory maximum dry density and a close approximation of the difference between the optimum water content and the fill water content, within about one hour after sampling and without determination of water contents. It is sufficient either to approve the compacted material or to require improvement or removal.

Another method was developed in road engineering. It is the use of a nuclear gauge for combined determination of density and moisture (Fig. 9.28 and FGSV 1975). It consists of the gauge case, the measuring rod, the detectors and the electronic accessories to evaluate and plot the measuring data. The case is placed on the levelled surface of the layer to be tested, the radioactive source in the rod is placed in a hole penetrating into the layer (Fig. 9.29). The density determination is made by use of a gamma source. Water content determination uses a neutron source. The measuring depth is about 30 cm for the density, and 10 to 20 cm for the moisture content. Accordingly, the volume of the density-tested material is around 8000 cm^3. For comparison: the volume of an undisturbed, conventional sample is about 1600 cm^3 only.

The operation of the gauge is not time-consuming. The results are readily available within a short time and at satisfactory accuracy. Therefore, the number of tests done with the gauge can be considerably greater than with

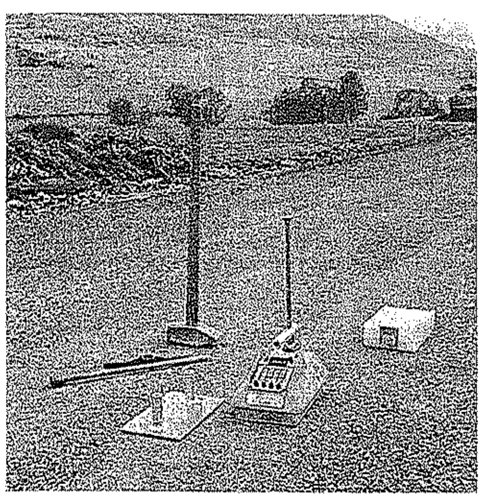


Figure 9.28. Nuclear gauge for combined determination of density and moisture of cohesive and non-cohesive soils (courtesy of Troxler).

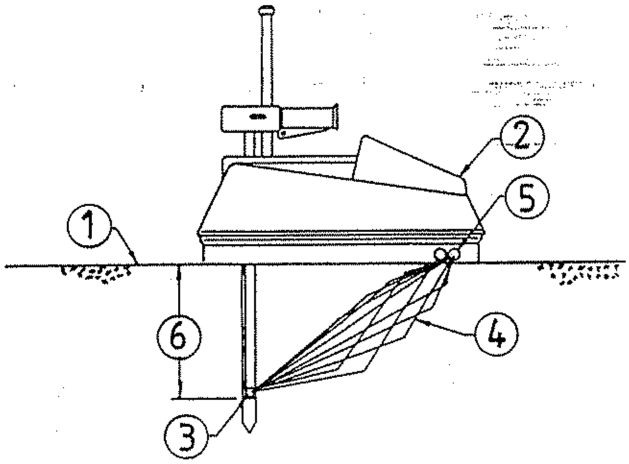


Figure 9.29. Working scheme of the nuclear gauge.

1 Surface of the soil layer to be tested
2 Gauge case with accessories
3 Measuring rod with radioactive source
4 Radiation
5 Detectors
6 Measuring depth

conventional methods. The evaluation of the embankment's homogeneity will benefit from this. Preparation for testing includes levelling the surface and drilling the hole to receive the radioactive source. The hole must be normal to the surface. This gives the limit of application: coarse particles of the soil, such as gravel, must not force the hole to deviate from normal direction.

The disadvantages of nuclear gauges are the price of the equipment and the need to train personnel. In most countries the operation and transportation of nuclear material are subject to strict safety regulations.

9.5.2 *Non-cohesive soils*

The state of compaction of non-cohesive soils is expressed in terms of the relative density D (Section 4.3.2.3). The moist unit weight of the compacted soil is determined in the field to calculate the void ratio of the soil in place, after determination of the water content or after drying the material out. It is compared to the void ratios in the loosest and most compact state, which are determined in the laboratory. The water or sand replacement method serves for moist unit weight determination (Section 4.2.6.2b). The weight of the sample and the volume of the pit must be adjusted to the maximum grain size of the soil. A sand-gravel with cobbles of 100 mm, for instance, requires a pit of 50 cm in diameter.

At least one relative density test is made for 3000 to 5000 m^3 of compacted soil. Each test is accompanied by a grain size analysis to enable estimation of the shear strength and the deformation modulus, which must fit with the reference data.

The recommended data of compaction after USBR are listed in Table 9.5. The reduction of the compaction degree from fine sand (D = 90%) to gravel (D = 80%) reflects the respective increase of the deformation modulus. Therefore, slightly decreased compaction is acceptable with coarse soils. All data are classified as 'dense' or 'very dense'. With respect to the water conditions in Table 9.5 reference is made to Section 9.4.2.1.

The filter materials require considerably more determinations of the relative density and the grain size distribution, since the latter is the criterion of the filter function. Usually, one test is made for 500 to 1000 m^3. The homogeneity of the material – and potential segregation – must be evaluated from the tests and by visual inspection.

The common laboratory equipment for the relative density determination does not fit for soils with a maximum grain size of more than about 50 mm. For coarser soils the test results are not accurate. Accordingly, the relative density cannot be used as a criterion to describe the state of compaction of coarse materials. In practice, it is common to compact the material in the field to a very dense state – or the densest state possible – by heavy equipment and to establish the percentage achieved in this state as a criterion, just as with cohesive soils. If, in parallel, the material was tested triaxially to obtain the shear strength and the deformation moduli, quality control in the field can be restricted to the evaluation of the moist or dry unit weight and the gradation.

With respect to deformation behaviour, it is occasionally suggested to perform plate load or penetration tests. However, plate load tests are not useful because of the long testing time required and because of the difficulties and costs of supplying and erecting the counterweight. Penetration tests are not useful because the penetration depth required is only 1.0 or 1.5 m, hence leading to inaccurate results. The results of penetration tests to greater depth will be obtained too late to request additional compaction or removal, should the material be found to be insufficiently compacted.

One recent development is called surface covering dynamic compaction control. The measuring principle is based on the fact that the dynamic response of the soil being compacted varies with the state of compaction. The variation is registered by one or more accelerometers mounted rigidly on the vibrating drum of the roller. The stiffer the soil, the more the vibration of the roller varies. The principle and the variation are demonstrated in Figure 9.30. The signals of the accelerometer – called a compaction meter or compactometer – are converted to compaction meter values (CMV) by a computer on the roller. The CMVs are recorded, printed out and stored. They are displayed for the operator or, alternatively, transmitted to a remote control

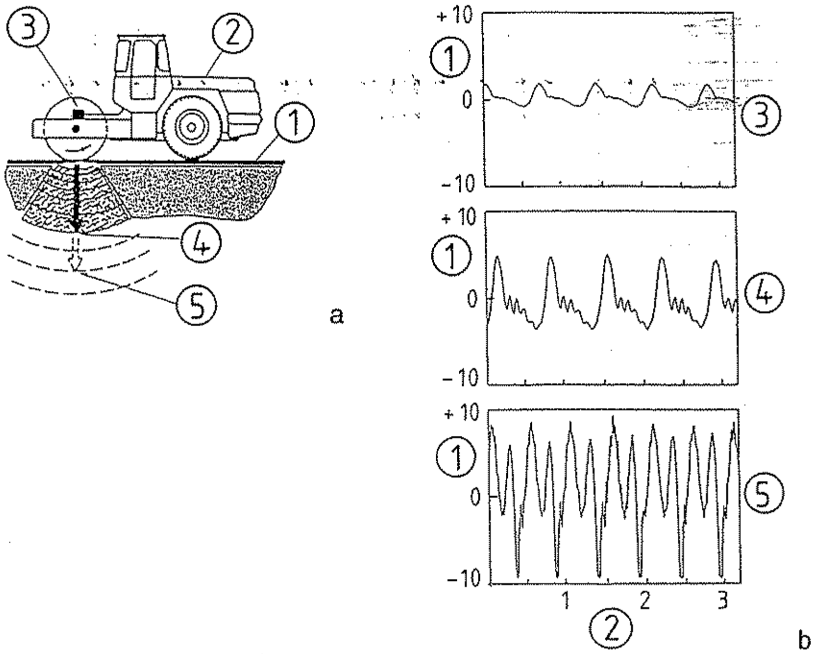


Figure 9.30. Dynamic compaction control of granular soils by roller-mounted compaction meter.

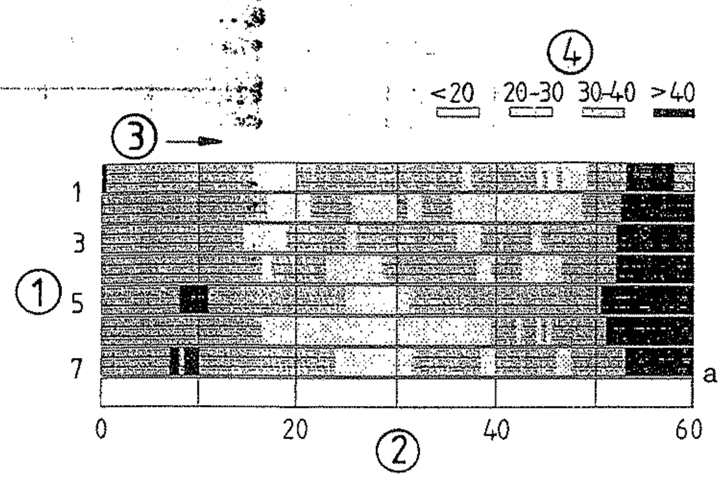
a Working scheme
(adapted from Forssblad 1988)
1 Surface of the layer to be tested
2 Vibratory roller
3 Accelerometer
4 Effective depth of compaction
5 Measuring depth

b Acceleration measurements
(adapted from Tykal & Schwab 1986)
1 Acceleration (multiples of g)
2 Time (ms × 100)
3 1st pass
4 3rd pass
5 7th pass
Sand-gravel 0/15 mm

station. The method was introduced to the profession by Forssblad (1981). The theoretical background can be read e.g. from Floss (1992) or Floss et al. (1983). Recommendations for the application are given by FGSV (1993). An instructive way to present the measuring data is shown in Figure 9.31.

The variation of the dynamic behaviour of the system soil/roller depends on both, the soil and the roller. Therefore, the compaction meter values must be calibrated for each type of soil and each type of roller. The values are affected also by the lift thickness and the stiffness of the base. This is reflected in Figure 9.30 by the difference between the effective and the measuring depths. The lift thickness should be slightly less than the effective depth.

The calibration is made by use of a test embankment, the compaction state of which is determined by conventional methods. In this case, plate load



1	4 min.	4 mean	4 max.	5
1	28.7	33.9	45.5	3.0
2	26.4	33.0	47.5	3.0
3	28.2	33.9	44.2	3.1
4	27.8	33.1	46.6	2.9
5	27.3	35.4	46.5	2.9
6	26.8	32.7	46.6	3.0
7	27.3	33.9	46.1	3.3
8	0.0	0.0	0.0	0.0
		33.7		

b

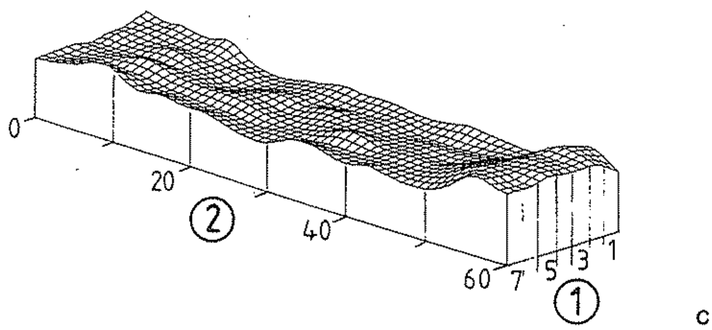


Figure 9.31. Presentation of measurements of the dynamic compaction control (example, adapted from Thurner 1986).

a Plan
b Digital print-out
c Perspective drawing
1 Working lanes
2 Length (m)
3 Working progress
4 Compaction meter values (CMVs)
5 Driving speed (km/h)

tests may be useful. The compaction meter values of coarse materials are proportional to the deformation modulus of the soil at the second load cycle of a plate load test, and proportional to the compaction degree expressed as a percentage of the unit weight of the soil in its densest state (Forssblad 1986, 1988, Tykal & Schwab 1986, FGSV 1993).

Dynamic compaction control by use of a compactometer can be applied to cover the whole surface of the embankment to be tested. The preconditions are that the same details apply to the embankment to be tested and the one which was used for calibration, and that the soil properties do not vary over a reasonable time and volume of embankment dumping and testing. Otherwise, a new calibration would be required. The compaction effect can be controlled continuously. Additional compaction, if requested, can be performed immediately. This is a valuable advantage of the compactometer method over conventional compaction control, in that the operator of the roller – or a supervisor in the remote control station – can evaluate the quality of the compaction work continuously and during the work, without any delay waiting for test results. Uneconomic and unnecessary compaction time and roller passes can be avoided.

It must be possible to identify the location of the roller and to correlate it with the respective measuring data. For this purpose markers at the driving lanes, start and stop lines and the record of the driving speed, the date and time are required. The speed should be constant. Localization is also possible by electronic identification of the roller position and related coordinates.

The limits of application are given by the soil properties, mainly by the content of fines and by the water content:

– For coarse grained soils, such as gravel, sand and sand-gravel having no more than 5% fines below 0.06 mm (group symbols GW, GP, SW and SP), there is a definite relation between the compaction meter value and the degree of compaction or the deformation modulus. This applies also to gap-graded soils of these groups. The effect of the water content is negligible. There must be no water puddles on the surface to be tested.

– For slightly cohesive soils, such as silty and clayey gravel and sand having up to 15% fines below 0.06 mm (group symbols GM, GC, SM and SC), there is a definite relation between the CMV and the degree of compaction or the deformation modulus, provided the water content is below the optimum water content of the standard compaction test. The relation is indefinite for water contents around and above the optimum. That means: the dynamic compaction control is not fully satisfactory if such soils are used as sealing material and accordingly compacted at water contents around or above the optimum. The method can be applied to find out weak zones and to evaluate the homogeneity of the tested material. The degree of compaction must be determined by conventional control.

Table 9.6. Properties of soil-cement for trench and slot diaphragms.

Project	Ref.	Base material Type	(kg)	Additional materials Clay (kg)	Cement (kg)	Water (kg)	Weight (kN/m³)	k-value (m/s)	Compressive strength (MPa)	Young's modulus (MPa)	Failure strain (%)	Procedure	Width (cm)
Brombach	a	Rock flour	112	50[2]	225	860	12.5	$\leq 10^{-8}$	≤ 0.7	150 to 400	2	N1	60
Förmitz	b	Sand with 20% silt	1345	165[1]	90	400	20	$\leq 5 \times 10^{-8}$	0.7	40	4	T	60
Frauenau	c	Sand with 15% silt						5×10^{-9}			10	T	60
Erosion tests	d	Sand	1200	80[1]	100	400	20	$3 \text{ to } 5 \times 10^{-9}$				N2	
		2 to 8 mm, rock flour	200	20[2]									
Kehl	e	Rock flour	670	20[2]	185	775	16.5	4×10^{-9}	≤ 2.0	200	1	S	8 to 12
River Lech	f	Rock flour	805	15[2]	165	645	16.3	$3 \text{ to } 5 \times 10^{-8}$	≤ 0.5			S	8 to 12
Average	g	Aggregates < 30 mm	1300 to 1500	40 to 50[2]	100 to 200	350 to 2000		$10^{-9} \text{ to } 10^{-8}$	≤ 2.0	$E_{wall} \leq 5\,E_{soil}$	5 to 10	N2, T	
		Sand < 1 mm	500 to 1000	35 to 75[2]	75 to 290	300 to 1500		$10^{-8} \text{ to } 10^{-7}$				N1	

References:
a Lorenz (1976), Beier & List (1982), Strobl (1989)
b Lorenz & List (1976), List (1980)
c Beier & List (1982)
d Strobl (1982), Beier & Strobl (1985)
e Archives LI (1990)
f BAWAG (1986)
g ICOLD (1985b)

[1]Powdered clay
[2]Bentonite
N1, N2 Trench diaphragm, wet methods (Section 7.3.2.2)
T Trench diaphragm, dry method (Section 7.3.2.2)
S Slot diaphragm (Section 7.3.2.3)

not show any change in permeability after a test period of one year. Leaching out under the lower gradient was negligible. Initial slight leaching under the higher gradient ceased after 20 weeks and was then close to zero until one year later.

Beier & Strobl found a limiting gradient of $i_{\text{limit}} \geq 300$. It is recommended to design diaphragms for a gradient of $0.5 \times i_{\text{limit}}$. The permeability should not be more than 10^{-8} m/s, since the risk of erosion increases abruptly at a higher permeability.

An additional test on the erosion stability was developed for the slot diaphragms of the river Lech (Example f in Table 9.6). A disc-shaped sample of hardened slot filling material is punched with a hole 1.0 cm in diameter. The sample is then percolated by water at a gradient of $i = 240$. Erosion-resistant material will not show any change in the hole or in the sample surface after 14 days.

A relation was found between erosion stability and uniaxial compressive strength after DIN 18 136 (controlled feeder speed). Samples according to the composition shown in Table 9.6 (Example f) were found to be erosion resistant, provided the strength was ≥ 200 kPa after 28 days. The relation must be verified for each type of mixture. With this precondition the uniaxial compressive strength can be taken as the erosion controlling parameter. The tests are cheaper than permeability and other erosion tests, and the results are available within a short time.

9.6.2 *Asphaltic concrete*

The strength and the permeability of asphaltic concrete are related to the properties of the components and the composition. Satisfactory strength is achieved by the use of strong and durable aggregates. The required minimum permeability is achieved by the extremely low pore volume in the order of 3% which is again related to the correctly selected content (by weight) of mineral fines and bitumen. A typical mixture is as follows (communication from Geiseler):

– Maximum grain size about 16 mm, so as to minimize segregation when being placed and to avoid inhomogeneities at the surfaces.

– 55 to 50% (by weight) crushed material of the sizes 2/5, 5/8, 8/11 and 11/16 mm.

– 21 to 24% crushed sand 0/2 mm.

– 11 to 12% natural sand 0/2 mm.

– 13 to 14% filler.

These aggregates accumulate to 100%. The first figures are typical for internal sealings, the second figures typical for face sealings. 6 to 7% bitumen is added to the above aggregates. Recommended aggregate gradations

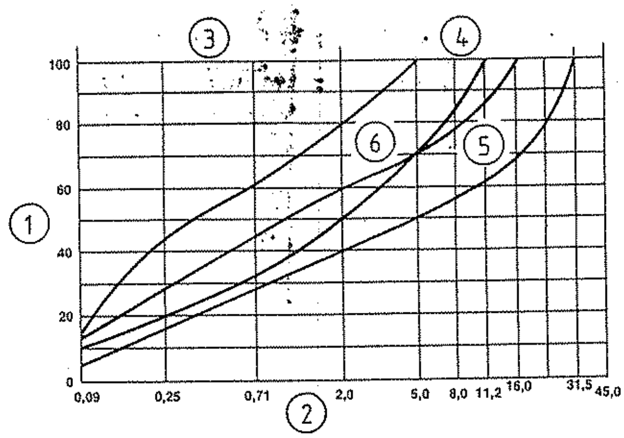


Figure 9.32. Gradation of aggregates for asphaltic concrete (adapted from DGEG 1983).

1 Percent finer by weight
2 Grain size (mm)
3 Sand
4 Gravel
5 Asphaltic concrete 0/16 to 0/32 mm
6 Asphaltic concrete 0/5 to 0/11 mm

are shown in Figure 9.32. Preferred types of bitumen are B 80 and B 65 after DIN 1995, in selected cases also B 200 or B 45.

The permeability of correctly composed asphaltic concrete must not exceed 10^{-8} m/s. This upper limit was also found by Breth & Schwab (1979) on samples of asphaltic concrete after 4 to 5% volumetric strain. The permeability was 10^{-10} m/s after 3% volumetric strain (see also Section 7.4.2.2a). The deformation behaviour of asphaltic concrete is, in principle, discussed in Section 7.4.2.1.

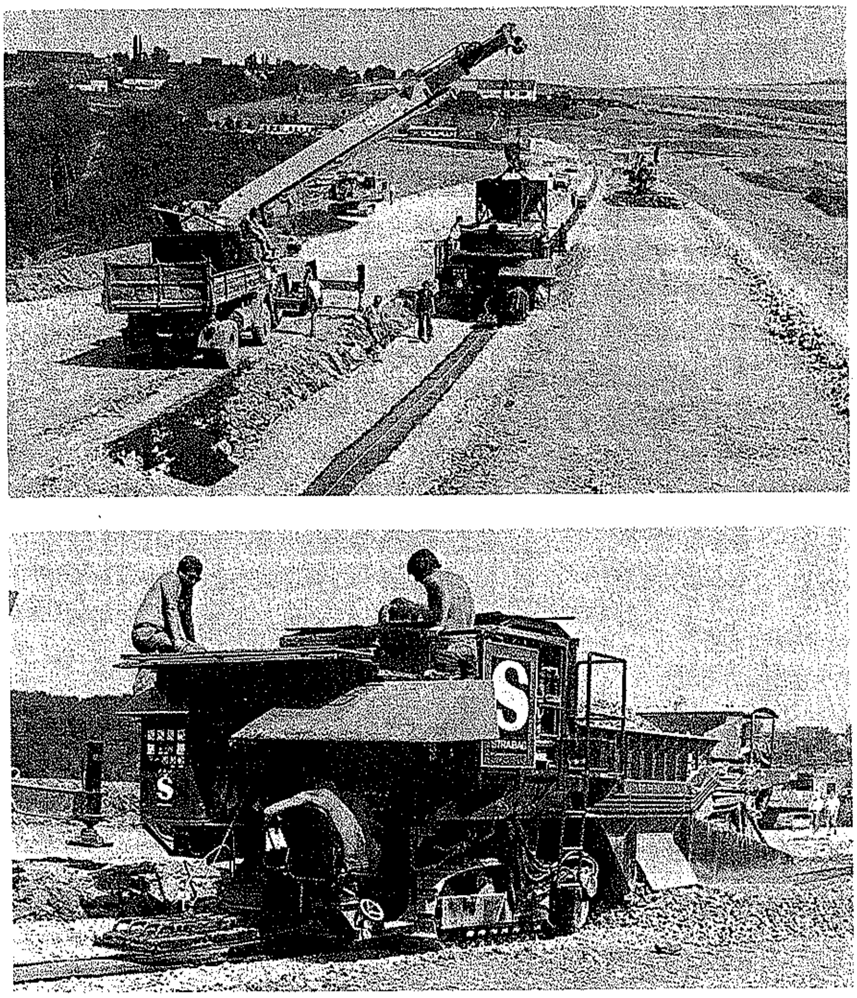
9.6.3 *Conventional concrete*

The maximum grain size of the aggregates is about 40 mm. The cement content was initially 350 kg/m^3. It is now preferably 300 kg/m^3. Accordingly, the water/cement-ratio is now about 0.55. The development of shrinkage cracks can widely be avoided by curing of concrete with water until filling commences. More details can be seen, e.g. from Cooke & Sherard (1985).

9.7 PLACING OF ARTIFICIAL SEALING MATERIALS

9.7.1 *Internal sealings of asphaltic concrete*

Special machinery serves to place vertical and inclined internal sealings and the upstream and downstream transition zones in one operation. An example is shown in Figure 9.33. The placing unit is moved by crawlers on the transition zones. The width of the crawlers can be adjusted to the width of the sealing membrane. Figure 9.34 shows a longitudinal section of the placing unit. The insulated hopper for asphaltic concrete is at the front. The material



a

b

Figure 9.33. Internal sealing of asphaltic concrete. Placing unit in operation (courtesy of Strabag Bau-AG).

a Transfer of hot asphaltic concrete in insulated bucket from truck to placing unit by means of a mobile crane (instead of conveyer belt as no 7 in Fig. 9.34)

b From left to right: Infra-red heaters, compartment for hot asphaltic concrete, compartment for transition materials

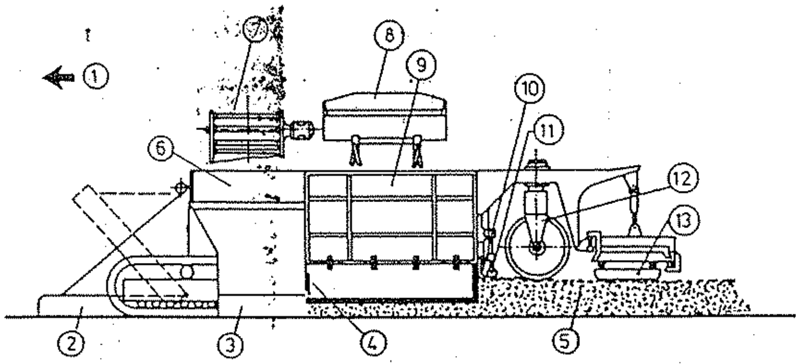


Figure 9.34. Internal sealing of asphaltic concrete. Longitudinal section of the placing unit (courtesy of Strabag Bau-AG).

1 Working progress
2 Infra-red heaters
3 Sliding steel shuttering
4 Levelling beam for asphaltic concrete
5 Transition zone
6 Hopper for asphaltic concrete
7 Conveyer belt for feeding with hot asphaltic concrete
8 Wheel loader for feeding with transition material
9 Hopper for transition material
10 Levelling beam for transition material
11 Pre-compaction device for asphaltic concrete
12 Rubber tired wheels
13 Vibrators (center unit is heatable)

slides from the compartment into the sliding steel shuttering which is adjustable to the width of the membrane (Fig. 9.35). The material is levelled to the selected layer thickness, typically 0.2 m. The crawlers and the hopper can be maneuvered to achieve very accurate placing of the membrane. The transverse tolerance is reported to be ± 1 cm.

The hopper for transition materials is in the middle (Fig. 9.34). The compartment is separated into two boxes to enable placement of different materials upstream and downstream. Due to the flexible shuttering the transition zones upstream and downstream may be different in width (Fig. 9.35). The material is automatically levelled flush with the membrane. A protective roof prevents the surface of the membrane from being contaminated by dust and sand. In the rear the machine is supported by a pair of rubber tired wheels, followed by the compaction unit consisting of three vibrators. The center unit for compaction of the asphaltic concrete is heatable.

The daily production rate of placing units is about 300 tons of asphaltic concrete, i.e. a membrane of approximately 200 m^2 at 0.6 m width. Given greater width, the rate is slightly more. The fill temperature is between 160 and 180°. The compactability is considerably affected by the temperature. To achieve an optimum of temperature and compaction the placing unit is equipped with infra-red heaters at the front and the heatable vibrator in the

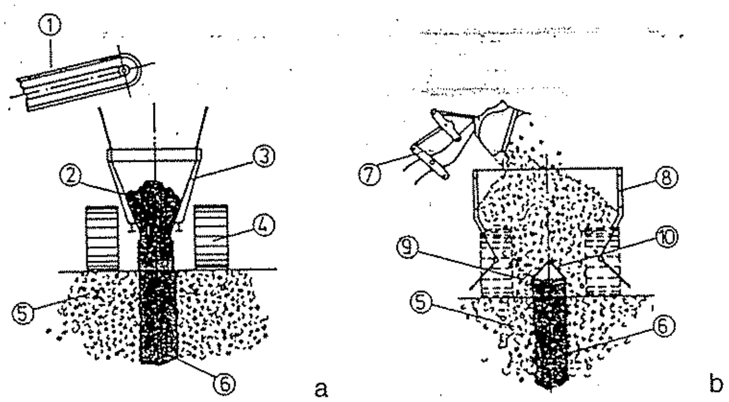


Figure 9.35. Internal sealing of asphaltic concrete. Cross sections of the placing unit (courtesy of Strabag Bau-AG).

a Cross section in the front part of the unit
b Cross section in the rear part of the unit
1 Conveyer belt for feeding with hot asphaltic concrete
2 Asphaltic concrete
3 Hopper for asphaltic concrete
4 Crawler
5 Transition zone
6 Asphaltic concrete membrane
7 Wheel loader
8 Hopper for transition material
9 Transition material
10 Protective roof

rear. Warming up of the previous layer contributes to a good bond between subsequent layers. Heating the vibrator prevents it from adhering to the membrane and from pushing asphaltic concrete forward (communication from Geiseler).

The suitability tests and the quality control of asphaltic concrete are described by ICOLD (1992). The requirement of suitability tests applies to each project since the properties of the aggregates vary due to the geological conditions. The properties of the available bitumen may also vary. The tests include bitumen, aggregates, filler and water and different mixes of asphaltic concrete out of which the most suitable one will be selected.

Quality control applies to the materials prior to mixing and to the asphaltic concrete prior to placement. The control serves to guarantee consistent properties of the asphaltic concrete. In addition, the dimensions of the structure made of asphaltic concrete must be controlled. The extraction of drill cores from the structure should be restricted to cases of particular concern and to a minimum number of samples.

9.7.2 *Face sealings of asphaltic concrete*

Face sealings of asphaltic concrete can be produced normal to the dam axis (in plan) or parallel to it. The latter may be favourable for long dams. It is proven with dams up to 25 m in height. The compaction is done along the maximum slope line so that better compaction of the joints may be obtained. An example is described by the Working Group of the Italian National Committee on Large Dams (1988). At production normal to the dam axis the equipment moves partly on the crest and partly on the maximum slope (Fig. 9.36). On the crest are the main winch unit with ramp and feeding equipment and the winches for the rollers and for accessories. On the slope are the finisher, the supply truck, the rollers and the heating equipment for the joints.

It can be seen that the productive equipment on the slope is operated by the winches on the crest. The relocation of the slope equipment from band to band, parallel to the crest, causes unproductive waiting time, particularly when the finisher must be relocated. There exists a most favourable slope length for each set of equipment which is a typical parameter in respect of the costs and the production rate. For the Pyrnos face (Mornos project) the optimum slope length proved to be 75 m, resulting in 60% productive and 40% waiting time (Efremidis & Lehnert 1979). The technique of running the finisher parallel to the crest diminishes the waiting time considerably.

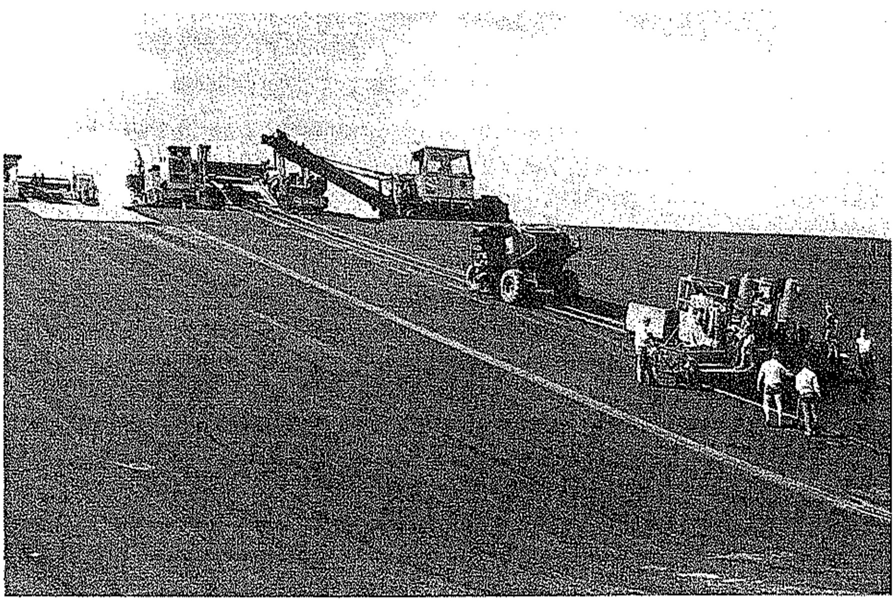


Figure 9.36. Face sealing of asphaltic concrete. Placing procedure and equipment (courtesy of Strabag Bau-AG).

From left to right: Main winch unit with ramp, feeding equipment, supply truck, slope finisher (vibratory rollers not shown)

The quality of the face sealing is affected by the temperature and the homogeneity of the hot mix. The temperature will be, and the homogeneity may be, affected on the way from the mixing plant to the final placement. A critical detail is to convey the hot asphaltic concrete to the supply truck which moves on the slope. A loading machine was developed to facilitate this process and to lower costs (Fig. 9.37). The machine is moved along the crest together with the main winch unit. The material is re-homogenized by augers in the intermediate container. The container serves also as a buffer to bridge the different loading capacities of the delivery truck on the crest and the supply truck on the slope which are about 20 and 4 tons, respectively.

The joints between bands are a critical detail of face sealings of asphaltic concrete. The bands are 3 to 5 m wide. Systems of several layers do not have joints from top to bottom. The layers overlap each other. The more modern system of one thick layer has such joints. The joints between neighbouring bands are welded by gentle reheating and further compaction by heated tampers. According to Haas et al. (1988) the tensile strength of the joint areas is much higher than that in the middle part of the band. The joint treatment helps achieve a permeability of the system in the order of 10^{-9} m/s.

The placing temperature and compaction are the controlling parameters of quality and hence durability, in addition to the material composition. Therefore, the equipment must be able to place thick layers at a high temperature and maximum compaction. The temperature is affected by the processes of mixing, transportation and placing. Tschernutter (1988) concludes that 'embrittlement of the bitumen from first processing and placement outweighs by far the ageing phenomena occurring during the first ten years of operation'.

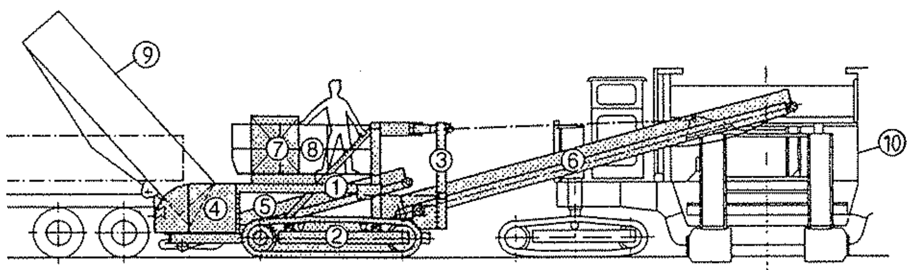


Figure 9.37. Face sealing of asphaltic concrete. Working scheme of the loading machinery (courtesy of Strabag Bau-AG).

1 Main frame
2 Crawler
3 Swinging frame
4 Intermediate container for hot asphaltic concrete
5 Intermediate conveyer belt
6 Conveyer belt to load the supply truck
7 Motor/generator
8 Control panel
9 Delivery truck for hot asphaltic concrete
10 Main winch unit

The compaction benefits from the delayed cooling process of a thick layer in comparison to a thin layer. It is seen favourable to have the minimum permeability at the surface and slightly greater permeability towards the bottom of the layer. Such a fabric contributes to quick drainage of seeping water. The surface compaction of thick layers favours the development of such a fabric.

The production rate is in the range of 25 to 30 tons per hour, corresponding to 120 to 140 m^2/h of an uncontrolled membrane 0.1 m in thickness (without the bituminous binder course). Suitability tests and the means of quality control are described by ICOLD (1982). Tests and control are very similar to the respective procedures on asphaltic concrete for internal sealings. In addition, permeability tests on the readily hardened face sealing are recommended. Non-destructive tests are preferred, such as vacuum tests. Tests using drill cores should be restricted to a minimum.

9.7.3 *Face sealings of conventional concrete*

The construction of concrete faces demands experience. Accordingly, contracts should be awarded to experienced contractors. It is argued by those engineers who favour concrete faces that the construction of concrete faces would demand for less experience than the construction of asphaltic face sealings. Accordingly, international competition would be less restricted in contracting concrete faces. It is not the place here to comment on this debate. It can be stated that a number of concrete face rockfill dams has been constructed worldwide and is performing satisfactorily, probably under conditions where not only specialized contractors had been awarded with contracts. However, there is no doubt that the contractor must be aware of the boundary conditions of the project, of the regional weather conditions and of the material 'concrete'.

The following principles have been developed in the course of the last decades which indicate how concrete faces on embankment dams should be constructed. The arrangement of the equipment doing the work is shown in Figure 9.38. The face consists of parallel slabs 6 to 14 m in width. The typical layout of joints can be seen from Figure 7.49. The construction of the face follows the construction of the embankment and the transition zone supporting the face. In principle, the embankment and the face may be constructed in steps until the final height is reached. In the lower portion of the face, starter slabs are required extending from the plinth to the horizontal contraction joint where the operation of the concrete placing unit and the slipform starts (Fig. 9.39).

On the crest rails are laid down to carry the winches and a tower crane (Fig. 9.38). The two main trolleys (9) and (10) serve to place the rails, the reinforcement, the slipform and side formwork, and the concrete. The rein-

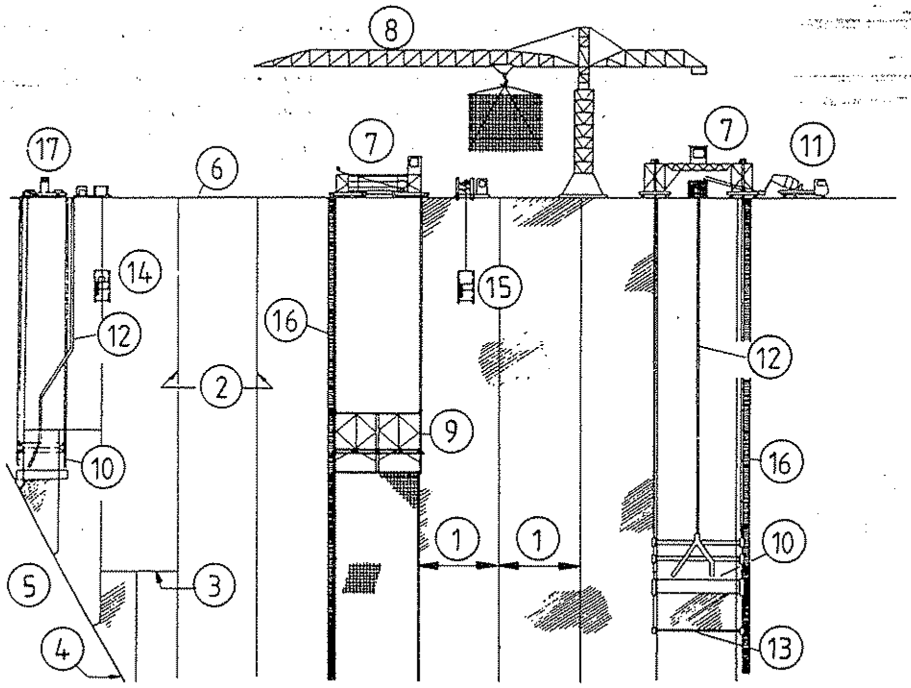


Figure 9.38. Concrete face sealing. Typical arrangement of construction equipment (adapted from Varty et al. 1985 and Fell et al. 1992).

1 Concrete slabs, width here 12.2 m
2 Vertical contraction joints
3 Horizontal contraction joint
4 Perimetric joint
5 Plinth
6 Dam crest
7 Rail-mounted transfer trolley and winches for nos 9 and 10
8 Tower crane mounted on crest rails
9 Rail and reinforcement trolley
10 Slipform and concrete placing unit
11 Concrete supply
12 Concrete delivery by pump or chute
13 Curing trolley
14 Trolley to place mortar pad and waterstops
15 Face access trolley and winch
16 Staircase
17 Crest winch for lateral concrete slabs of reduced width

forcement is prefabricated at the crest and transported to the reinforcement trolley by the tower crane. The long reach of the crane allows placement of reinforcement on the upper part of the face. In principle, reinforcement and concrete are placed from bottom to top. The reinforcement is anchored to the rockfill.

The concrete is pumped to the concrete placing unit or conveyed via chutes. The concrete composition and the slump must be adjusted to the climate and to the work progress. The placing unit must counterweight the uplift to enable concrete placement to the designed thickness. Irregularities of

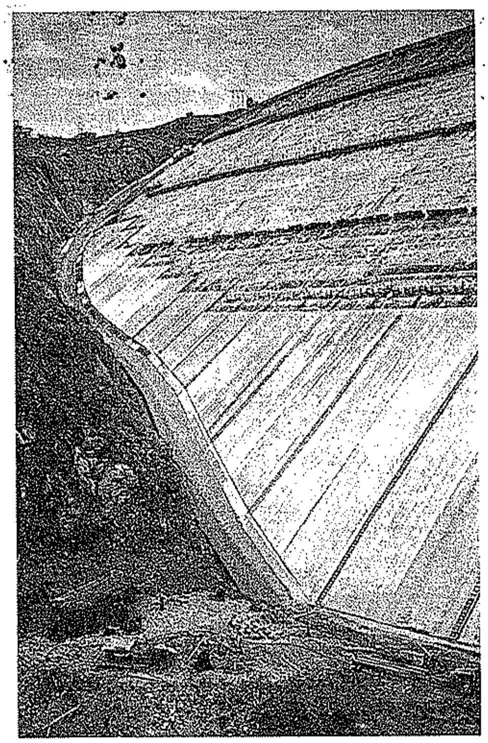


Figure 9.39. Concrete face sealing. Photograph shows the starter slabs from the plinth to the horizontal contraction joint and the shotcrete cover on the transition layer.

the base are commonly tolerated up to about 25 mm. Before concreting, the mortar pad and the W-shaped copper waterstops are placed by the respective trolley (14) and welded.

The slipform can be moved at a velocity in the range of 1.5 to 5 m/h, on average 3 m/h. According to Varty et al. (1985) the monthly production rate is up to 3300 m^2 of the face. More details can be seen from Cooke & Sherard (1985).

9.8 PREPARATION OF THE FOUNDATION

9.8.1 *Foundation of natural sealing elements*

With rock foundations the final foundation area is reached by excavation of loose material and weathered rock. Excavation must be done without damage to the remaining rock. Therefore, it is frequently requested to loosen the last 50 cm of material by use of pneumatic tools only, not by blasting.

The exposed irregular foundation area must be treated in detail. Typical irregularities and their respective treatment are shown in Figure 9.40:

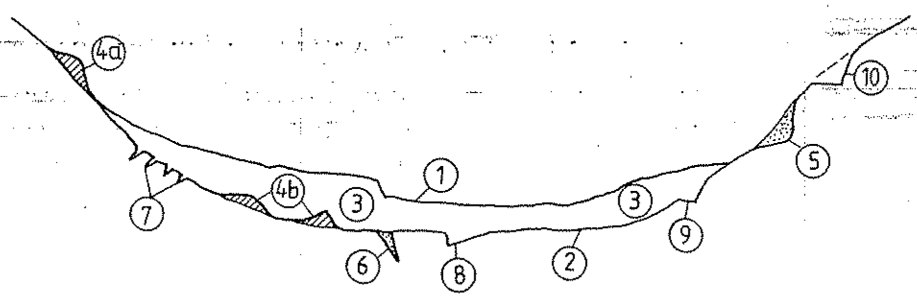


Figure 9.40. Typical foundation preparation under a dam core of natural materials.

1 Original river bed
2 Core foundation line, fresh or slightly weathered rock
3 Excavation of river sediments, slope debris and weathered rock
4a Removal to flatten the slope
4b Removal to smooth the foundation area
5 Concrete backfill
6 Concrete backfill after cleaning out to 3 times the width
7 Sealing of superficial cracks by slush grout or shotcrete
8 Steps acceptable, less than 1 m high and 2 m long
9 Ledges acceptable, not wider than half the thickness of plastic clay
10 Concrete backfill to level the slope at construction roads

– Steep slopes are flattened to 1V:0.6H or flatter (4a).

– Peaks are removed to smooth the foundation area (4b).

– Depressions are cleaned and refilled with concrete (5). Such concreting can be seen in Figure 9.20.

– Joints are cleaned out to three times the width and refilled with concrete (6).

– Superficial cracks are cleaned and covered with shotcrete or slush grout (7).

– Large steps are smoothed (8). Steps acceptable are up to 1 m high and 2 m long.

– Large ledges are smoothed (9). Acceptable are ledges up to half the thickness of the plastic material to be dumped in the interface of the core and the foundation.

– Construction roads crossing the foundation must be refilled.

The treated foundation area must be cleaned of all loose material and water puddles. Compressed air and pressure water are proven means for the final cleaning. Weak rock is covered by shotcrete to prevent damage. Some geotechnical engineers stabilize the whole foundation area of the core with shotcrete or slush grout. Rogers & Pearce (1991) report on such a case using slush grout. The decision was made 'because sections of the foundation would not be covered with fill material for up to 18 months and because of the numerous areas of dense jointing, sheared zones and clay filled seams'. The slush grout cover was expected 'to prevent deterioration due to prolonged exposure and to minimize further clean up costs'. The slush grout was a water cement suspension with a water/cement-ratio of about 0.5 by weight. It was applied pneumatically.

Joints and cracks in the foundation area including the walls of the core trench must be sealed with mortar or shotcrete. This measure is indispensable at the downstream wall. In addition, a filter must be placed there, as shown in Figure 7.14. Filter and shotcrete must prevent fines of the core from being washed into joints of the adjoining rock. The filter is required, in addition, to cover deformation cracks of the shotcrete.

Consolidation grouting can be performed at an intermediate stage of excavation, e.g. 1 m above the final depth. The remaining material serves as a counterweight to the grout pressure. However, it is a disadvantage to interrupt the excavation work. Contractors will tend to perform grouting after final excavation. In this case attention must be paid to the final cleaning of the foundation area of the remains of the grouting work.

It is common practice to start filling work with a layer of plastic material (Fig. 7.21). The thickness of this layer must be adjusted to the remaining irregularities of the surface and on the ability of the construction machinery to maneuver on the weak layer. An average thickness is 0.6 m.

The sealing element must be placed on weathered rock or on soil if fresh rock cannot be reached, for economic or technical reasons. The engineer must decide on the end of excavation work and hence on the compatability of the foundation area with the design. Settlements are minimized by the excavation of obviously loose material. The surface of a soil foundation should be treated by several passes of compactors.

9.8.2 *Foundation of the dam shells*

The strength of the common shell foundation is equal or greater than the strength of the shell material. This usually applies to moderately weathered rock – grade WIII of Table 3.2 – or to coarse soil in a dense to moderately dense state. The foundation area must be free of rooted soil and of obviously weak, compressible material. Unavoidable settlements of the foundation material will occur during dam construction and will not be deleterious.

Natural depressions and caves which remain after the excavation of weak materials can be filled with shell material. It must be compacted to the same dense state as the dam body. If such compaction is not possible due to the confined space, the cave is filled with concrete. The strength of the foundation should be a maximum at the place of maximum stresses. This is the inner part of the downstream shell.

A typical localized foundation treatment was carried out at the Kinda dam. There was a geological fault striking across the dam (Fig. 9.41). A V-shaped trench remained after excavation of disintegrated rock. Underneath the core and the filters the trench was filled with concrete sloping 1V:4H upstream and downstream. Underneath the shells the trench was filled with rockfill material. The requirement of concrete underneath the filters is noted, to exclude settlements there which might deteriorate the function of the filters.

The shells may also rest on soil instead of rock, taking regard of the required shear strength. In this case the foundation preparation consists of the removal of rooted top soil and of weak, compressible material. The final foundation area should be compacted by several passes of the usual compactors.

Attention must be paid to clay foundations. The critical slip planes will cross the foundation emerging upstream or downstream of the dam body (Fig. 6.7c). Slope stability can be achieved by berms as in Figures 7.43b and 7.52. The dimensions of such berms are often designed only after foundation excavation and after additional investigation of the subsoil, e.g. by penetration tests.

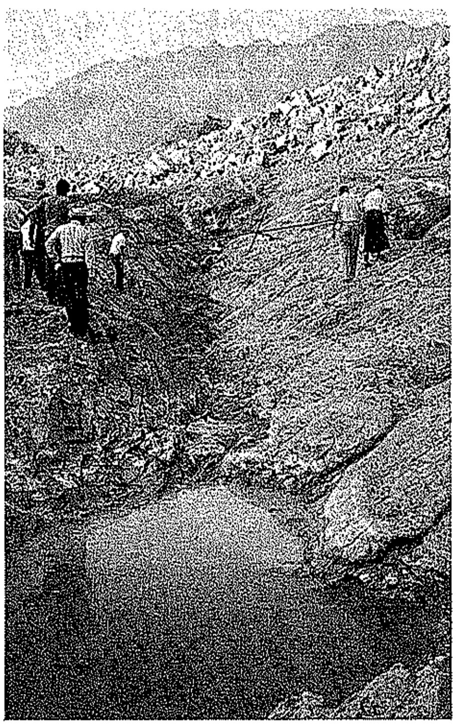


Figure 9.41. V-trench after excavation of disintegrated rock of a geological fault (courtesy of LI).

The stabilizing effect of berms can be improved by relief holes which help to accelerate the dissipation of the pore-water pressure in the weak foundation. The progress of construction of the dam and the berm must be adjusted to the shear strength of the foundation which is time-dependent due to pore pressure dissipation.

9.8.3 *Foundation of artificial sealing elements*

The sealing element and its concrete support are more sensitive against deformations than a wide sealing element of cohesive soil. Therefore, special attention must be paid to such deformations and their consequences. Close cooperation of the supervising geologist and the design engineer is requested, because insufficiencies of the foundation may be detected only during the excavation procedure. Then, a prompt decision is required and partial re-design, if necessary.

The foundation area for the supporting structures of artificial sealings is reached by the excavation of soil and weathered rock. Excavation must be made with a minimum of rock disturbance. Particular care in this respect is needed for narrow, deep excavations as e.g. shown in Figures 7.58 and 7.60. Light blasting and the use of pneumatic tools are recommended. The exposed rock is treated as described before (in the sense of Fig. 9.40). Further work to construct the concrete support of the sealing needs the approval of the foundation area by the supervising engineer or geologist.

The plinth of a concrete face, according to Figure 7.53, usually serves as a counterweight for curtain and contact grouting. Any lifting of the plinth must be avoided to exclude the development of seepage paths along the interface of concrete and rock. During the grouting work, potential movements of the plinth must be controlled by geodetic survey or with the aid of automatic movement indicators.

9.9 EMBANKMENT DEFECTS

This section deals with defects of embankment dams consisting of natural materials. The defects are mainly caused by:

- Unfavourable stress distribution, with subsequent hydraulic fracturing,
- unfavourable deformation, with subsequent cracking,
- ageing processes.

The defects are not caused by faulty design or by an offence against accepted rules or against the state of the art. The examples are selected to demonstrate the consequences of such defects with reference to the dam body. Defects of joints and connections to structures are discussed in Section 7.5.

In the literature ageing processes are also dealt with under the term 'durability'. This term includes processes in the foundation which are started or accelerated by the stored water in a reservoir. Examples are the dissolution of gypsum, the development of karst or the leaching out of a grout curtain. Such cases are not presented here. The durability of asphaltic concrete is discussed in Section 7.4.2.2.

9.9.1 *Hydraulic fracturing*

Hydraulic fracturing may lead to the failure of a natural sealing element with subsequent erosion and destruction of the dam. Hydraulic fracturing occurs under conditions where the water pressure of the reservoir exceeds the earth pressure in the dam. The magnitude of earth pressures in different directions depends on the shape of the sealing element and on the deformation properties of sealing and adjacent materials (Section 7.3.2).

The occurrence of hydraulic fracturing is made more likely by sealing material which was dumped and compacted in a state too dry to develop tensile strength. The water pressure may then initiate fracturing like a brittle failure and may create cracks. Water that enters a crack may widen and extend it. As is known from grouting techniques, the pressure needed to widen and extend a crack is smaller than the pressure needed to create the crack. It must not extend across the sealing element to its downstream edge because the earth pressure increases from the upstream edge towards the center. However, the existence of cracks is a risk for the durability of the sealing.

Sherard (1986) explains hydraulic fracturing in embankment dams as a common process. All impervious sections of embankment dams contain multiple cracks which develop from drying, from maneuvering of the equipment and from incomplete bond between layers. These cracks are not visible because they are closed by the overlying weight, but they exist in the fabric of the compacted soil 'which the reservoir water can probably enter when the pressure is greater than the earth pressure'.

It appears to be obvious that material which tends to brittle failure will increase the risk of hydraulic fracturing. This points again to the advantage of material which was compacted wet of the optimum. It is also obvious that hydraulic fracturing does not need high excess pressure but may occur at a water head slightly above the location of the cracks, because the cracks 'invite' the water to enter. This is confirmed by known incidents where hydraulic fracturing started during, not after, the first reservoir impounding.

9.9.2 *Cracks in homogeneous dams*

Hydraulic fracturing in homogeneous dams may be the main reason for a concentrated leak. This is in spite of the facts that the internal earth pressure increases towards the center of the dam and that the crack, once opened, must penetrate the dam at great length. Bertram (1967) and Sherard (1973, 1986) report on the 30 m high Wister dam where a concentrated leak developed during the first rapid impounding due to a large storm which, fortunately, did not lead to destruction of the dam. As in most cases of this kind the defect arose for several reasons. The example demonstrates the interaction of deformation and hydraulic fracturing.

Figure 9.42 shows a longitudinal section of the dam. The section above the river channel was constructed as a closure section. It acts like a plug between the adjacent parts of the dam. The right river bank (in Fig. 9.42 left) consists of a 10 m high step of incompressible rock. The left bank consists of clayey, compressible sediments. These conditions caused a differential settlement within the dam above the river channel in the order of 25 cm.

Sherard describes an arching effect in the plug (as indicated) resulting in reduced vertical stresses and horizontal cracks in connection with hydraulic fracturing as the reasons for near failure piping. Erosion tunnels became visible after the reservoir level had been lowered. Eroded dam material was observed in the seepage water. According to Sherard the dam was built of dispersive clay. This may have had an adverse influence on the amount and the velocity of erosion, but it was not the initial reason for hydraulic fracturing.

It cannot be excluded that the arching effect was supported by incomplete compaction of the lower portion of the plug, due to the confined space and the effort to complete the closure in time. Arching effect and plug are most

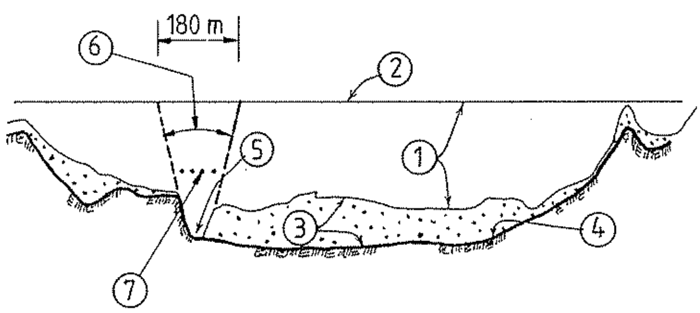


Figure 9.42. The Wister dam. Longitudinal section showing location of erosion tunnels with respect to closure section (adapted from Sherard 1973, 1986).

1 Dam body
2 Dam crest
3 Foundation alluvium
4 Shale bedrock surface
5 River channel
6 Arching across closure section
7 Location of erosion tunnels

probably connected with each other. Without the plug, or with flat slopes of the adjacent parts, arching would not occur.

As a conclusion the following is stressed: differential settlements, arching effect and hydraulic fracturing are potential risks for the stability of homogeneous dams made of non-dispersive and dispersive materials. The construction method with a closure section within steep slopes – like a plug – is an additional risk because it favours the development of local deficits in earth pressures.

The Wister dam has a bottom drainage of sufficient length to drain common seepage. It was, necessarily, ineffective in controlling the concentrated leak. The question arises whether a chimney drain would have been effective in making the incident less hazardous. It can be assumed that hydraulically opened cracks cannot proceed across a non-cohesive filter. Such a filter will act as a crack-stopper. Seepage is non-deleterious if the drainage capacity of the chimney and the bottom filter is sufficient. Also, the erosion of fines is stopped at the filter if its gradation fits with proven filter rules.

It is, therefore, strongly recommended to equip homogeneous dams with an inclined or a vertical filter according to Figure 7.10, in addition to the bottom drainage. Such a filter is a considerable gain in the dam's safety, irrespective of a given sufficient capacity of the bottom drainage as a sole draining element.

9.9.3 *Cracks in dam cores*

Considering the failure mechanism in homogeneous dams, it is obvious that dam cores may also be endangered by interaction of deformation and hydraulic fracturing for the following reasons:

– The risk of arching exists in longitudinal and transverse direction. In the longitudinal direction the core is supported by the two abutments, while the center portion may settle. The tendency increases with decreasing width of the valley, with the steepness of the abutments and with irregularities of the foundation area. In addition, the tendency increases with the compressibility of the foundation in the valley in contrast to its compressibility at the abutments.

– In the transverse direction the tendency of arching increases with increasing difference in stiffness of the core and the shells and with decreasing width of the core.

– The hydraulic gradient of seepage across the core is high. The order of magnitude is $i = 2$ to 3 with dam cores and $i = 0.2$ to 0.3 with homogeneous dams.

The Sylvenstein dam is an example for arching in longitudinal and transverse directions (Fig. 9.43). The dam, consisting of stiff gravel shells and a narrow impervious core of processed moraine material in the center, rests on

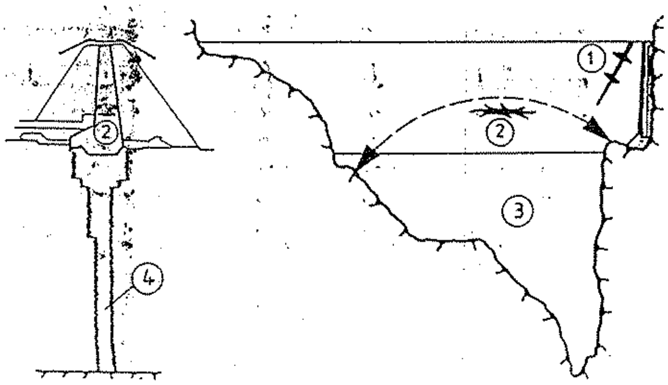


Figure 9.43. The Sylvenstein gravel dam. Probable cracking of the core due to arching (adapted from Beier et al. 1979).

1 Transversal shear crack
2 Horizontal cracks due to arching
3 Compressible valley sediments
4 Grout curtain

compressible valley sediments. These consist of gravel and sand layers with intercalations of silt. Considerable differential settlements had to be expected due to the heterogeneity of the sediments and due to the extremely irregular contour of the bedrock surface. The processed dam materials have been placed in the dam so as to contribute to an acceptable distribution of differential settlements (Fig. 9.44).

Experts have repeatedly reported on the experiences gained from about 35 years of dam operation (Beier et al. 1979, List & Sadgorski 1981, List & Strobl 1991). The settlement measuring devices shown in Figure 9.44 indicate the settlements of the dam core. The location of the instruments is noted: S4 rests on incompressible rock at the right abutment, S5 and S6 rest on compressible alluvions. The thickness of the alluvions is about 100 and 50 m, respectively. The measuring results allow the following conclusions (Beier et al. and the author):

– After completion of the structure in 1958 the consolidation process of the foundation is still continuing (measurements of 1959 and 1966).

– The compensating pad of plastic soil above the rock has considerably reduced the differential settlement to be expected there due to the almost vertical rock contour. The settlements are about 20 cm at marker 2 of S4 and about 27 cm at marker 2 of S5.

– The different foundation conditions of S4, S5 and S6 are clearly indicated by the settlements of the bottom edge of the core (markers 1). In contrast to the expected and typical behaviour, the settlement of S6 is greater than that of S5. This indicates the heterogeneity of the sediments.

– The maximum settlement of zone 4 (2% bentonite) is greater than the

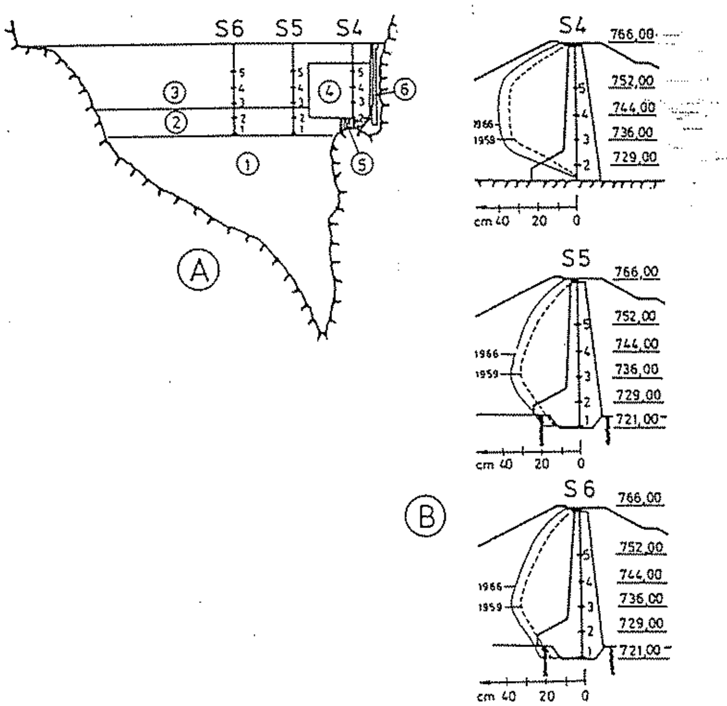


Figure 9.44. The Sylvenstein gravel dam. Material distribution and core deformations (adapted from Beier et al. 1979).

A Longitudinal section along dam axis
B Core settlements
1 Alluvial valley sediments, grouted
2 Soil cement (base material moraine)
3 Soil cement + 1% bentonite
4 Soil cement + 2% bentonite
5 Settlement compensating pad of plastic soil
6 Sluiceway buttress
S4, S5, S6 Settlement measuring devices with markers 1 to 5

maximum settlement of the adjacent zone 3 (1% bentonite). The maximum of zone 4 extends over the full height, due to the high plasticity of the material.

– Atypically, the maximum settlements of S5 and S6 are located in the lower portion of the core, instead of, as usual, in the upper portion. This is probably due to the heterogeneity of the sediments.

The settlement measuring devices are located approximately in the center line of the core. They allow an assessment of points of the water level in the core. Until about 1960 the water level in the core corresponded, with a short delay, to the storage level. The level in S6 was almost equal to the storage level, the levels in S5 and S4 were below the storage level.

In 1965 and the following years the water level in S6 dropped down and rose again simultaneously with the storage level, indicating a communication from the reservoir over S6 to the tail water. In 1969, after a drawdown and subsequent rising of the storage level, the water level in S6 did not follow the storage level but remained at an elevation about 20 m below. This indicates an increase of the core's permeability. The amount of seepage increased from about 5 l/s in 1962 to 10 l/s in 1970. This was not seen as alarming, but the need for repair was not excluded.

The reporters assume the events to be a consequence of arching, as indicated in Figure 9.43, for the following reasons:

– The narrow plastic core may hang up between the stiff adjacent shells by stress transfer.

– The arching effect across the core is supported by the wide core in the lowest portion. Core widening was designed to give the grout curtain a safe support.

– An arching effect may develop in the longitudinal direction due to the stiff abutments of the narrow valley and due to the compressibility of the sediments. S6 with the observed changes in the core's water level is located at the place where cracks due to arching have most probably to be expected.

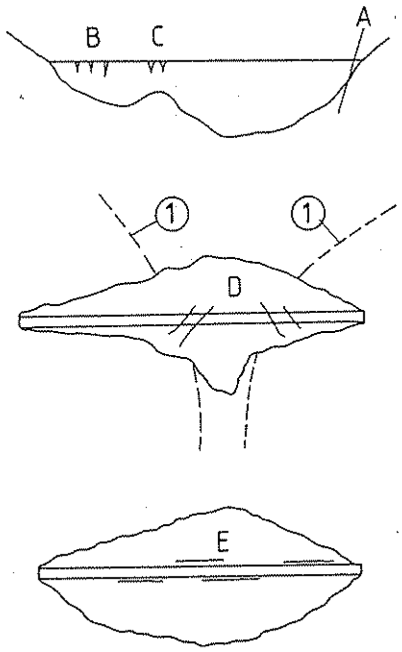
The existence of cracks due to arching was confirmed during the repair work in 1970 by respective drilling progress and loss of drilling mud. Subsequent grouting resulted in a reduction of the seepage from 10 l/s to zero.

Further confirmation was given by FE-computations (List & Sadgorski 1981). Computed settlements were in good agreement to measured settlements. This allows the assumption that the FE-input of stresses corresponds satisfactorily with the actual stresses. The computation results in tensile stresses at those places where cracks due to arching can be expected.

The following can be concluded: the events observed with the 180 m long dam, in a narrow valley with the extremely unfavourable rock contour and the mighty deposit of heterogeneous sediments, are connected to the phenomena of arching and hydraulic fracturing of the core. The measures taken to improve the core material and to place it most advantageously in the core have clearly contributed to limiting the dam's deterioration.

Sherard (1986) describes the risk of arching in the vicinity of irregularities in the foundation area, such as steps and ledges. He observed leaks up to a discharge of 50 l/s at the bottom of a 20 m high dam resting on quartzite. Steps of the rock surface up to 2 m in height had been detected. This is the reason to limit the permissible height of steps to a maximum of about 1.0 m (Fig. 9.40).

Typical cracks due to excessive deformations are demonstrated in Figure 9.45. These are shear cracks normal to the dam axis at steeply sloping abut-



a

b

c

Figure 9.45. Cracking patterns in embankment dams (adapted from Thomas 1976).

a Longitudinal section
b, c Plan
A Shear crack normal to dam axis
B Tension cracks over steep abutment, normal to dam axis
C Tension crack over saddle, normal to dam axis
D Cracking due to diverging rock contours, oblique to dam axis
E Cracking due to settlements, parallel to dam axis
1 Contour lines of the rock surface

ments (A). The crack at the right abutment in Figure 9.43 is an example. Such cracks are usually not open and hence not visible at the crest. The cracks may be widened by water entering and will then be dangerous.

Further, there are tension cracks over steep abutments (B) and over saddles (C). They develop due to differential settlements of adjacent sections. Such cracks are usually open and visible. They run normal to the dam axis or oblique at the condition of diverging contour lines (D). Measures to avoid such cracks are smoothing the foundation area and placing plastic material there and at the dam shoulders (Fig. 7.21). Existing cracks must be filled with plastic material or grout after cleaning.

Other than described for the Sylvenstein dam, arching and differential settlements may also occur without the development of cracks but with the development of a hazardous stress deficit. In such cases, during first impounding the water can penetrate into the zone of reduced stresses as soon as the hydrostatic pressure exceeds the total stresses. The tendency increases with decreasing cohesion and failure strain of the soil. So, the risk exists particularly with soils of low plasticity. In such soils also the pore-water pressure is reduced which is a part of the total stresses (Eq. 7.3).

In fact, experience with dam cores of low plastic moraine and similar material shows that high hydrostatic pressures occurred in the downstream portion of the core after first impounding. Open cracks and cracks filled with very wet soil have been found (Dascal 1984, Sherard 1986, Nilsson & Norstedt 1991). Such defects may have the following results:

– Abrupt rise of the seepage quantity up to a maximum which then remains constant. The water carries no suspended particles. There is no erosion at the flanks of cracks. The defect can be accepted if no erosion occurs over long periods and if the water loss is tolerable. Careful observation is required.

– Abrupt rise of the seepage quantity up to a maximum with further drop of the quantity down to a negligible rate. Initially the water may carry suspended particles. Erosion of fines has to be assumed which could not pass the filter. The leak was progressively clogged. The filter is effective. The defect is not deleterious. Careful observation is required.

– Abrupt rise of the seepage quantity with or without further progressive increase, the water carrying suspended particles. This case reflects the start of piping which cannot be accepted. It demonstrates an offence against proven rules, i.e. extreme arching and hydraulic fracturing due to inadequate design or work performance in connection with faulty filter design or segregation leading to the ineffective filter.

In most cases the above defects will develop simultaneously in several locations and at different intensity. This will aggravate the investigation of the causes.

9.9.4 *Cracks in dam shells*

Longitudinal cracks in dam shells in the vicinity of the crest as shown in Figure 9.45c are rather harmless. They may develop due to settlements of the shell. Sherard (1973) describes an example where a 90 m long crack was found some meters apart from the crest in the downstream shell. The crack was 40 to 60 cm wide, but only 2 m deep.

The respective dam is 30 m in height. It consists of the shells of well graded gravelly sand with a maximum of 30% silt and a central clay core. The shell material was placed at a water content of about 4% which is considerably below the optimum. The material was placed without addition of water and compacted by at least 6 passes of a 500 kN pneumatic tired roller. The shells will be stiffer than the core. The crack developed during first impounding. According to Sherard the upstream shell was wetted for the first time. The related settlement caused the core to deflect towards upstream. The stiff downstream shell did not follow this deflection, so the crack developed due to the separation of core and downstream shell.

In this context it is noted that dam cores frequently deflect towards upstream until the water level in the reservoir has reached about half the dam height. In the example of Sherard the rising of the water was slow. It remained about 6 months at 2/3 of the dam height. So, the core did not move towards downstream; the crack did not close.

9.9.5 *Defects due to ageing*

In practice, defects due to ageing, with subsequent deterioration of the material properties, rarely occur with earth and rockfill dams of natural materials. Combelles' General Report (1991) mentions a few cases of weathering and dissolution of rockfill materials. Such events are not discussed here because they can be avoided by respective material selection. Erosion processes and piping are discussed in other sections of this book. Erosion of soil-cement is explained in Section 9.6.1. Also, the well known, long-term process of filter clogging is not discussed here, since it is avoidable, or equipment for repair measures can be provided from the beginning.

An exceptional case of ageing is again the Sylvenstein dam with the core of processed moraine, reported by List & Strobl (1991). After the fist repair in 1970 new unexpected observations were made with the core in 1982. In the lower part permeabilities up to 10^{-5} m/s had been observed. Again the core was successfully grouted. Before grouting the pressure drop of the seeping water was found mainly in the downstream filter. After grouting the drop occurred mainly in the downstream part of the core.

List & Strobl explain that the increased permeability might be the effect of an ageing process of the Na-bentonite which had been added to the moraine material. The water absorption potential of the Na-bentonite of up to 700% might lead to unstable structures and related loss in volume as a long-term process. The reporters did not exclude a further ageing process with the same result. Further observations and the evaluation of all data of pore-water pressure did not indicate a change of the pressure conditions in the core of the Sylvenstein dam until 1995 (communication from Strobl).

Similar experiences from other dams with bentonite-improved materials did not come to the knowledge of the author. The question of ageing should be followed up. The case points to the requirement of permanent dam observation and monitoring. The process is not alarming. But the owners of the respective structures should be in a position to know that the need for repair measures might come up.

CHAPTER 10

Monitoring of the safety and the performance of dams

10.1 OVERVIEW

The safety and performance of embankment dams must be controlled during the construction, during the first impounding and during long-term operation. In the words of ICOLD (1989b): 'It is generally accepted that safety does not depend only on proper design and construction, but also on monitoring actual behaviour during the first few years of operation and over the service life of the structure'. For this purpose the dam and its vicinity and the foundation are equipped with measuring devices and controlling instruments. The total of these devices and instruments is called 'dam instrumentation'.

The data from the instruments may be made available continuously or at certain intervals. They must be supplemented by observations made during routine inspections of the dam and its vicinity and by reports on unusual events, such as floods, earthquakes, incidents on the crest and similar. The routine inspections are required in addition to the instrument readings since the instruments monitor the dam's performance only at defined locations. It is known that the first indications of irregularities such as cracks, wet spots or changes in the appearance of plants can be registered only by these inspections.

The monitoring and the inspections follow a schedule which defines the frequency of inspections, of instrument readings and of other checks. Understandably, the frequency of all control measures decreases from the construction and impounding periods towards the long-term operation. However, the control measures must be taken in a defined turn during the whole lifetime of the structure. Apart from the pre-designed schedule the frequency of readings and inspections should be increased at times of exceptional events, such as floods, storm-induced wave action, earthquakes and the like.

In some countries the owners are obliged to summarize annually all control measures and their evaluation, including the instrument readings, in a safety report. This report must reflect the correspondence of the reported data with the boundary conditions, such as precipitation, storage level, earthquakes and similar.

10.2 DAM INSTRUMENTATION

10.2.1 *Concept and location of instruments*

The main targets of monitoring during the lifetime of the structure are:

– The control of dam movements by geodetic survey of selected points of the dam crest and of the downstream slope, and

– the control of the quantity and quality of the water seeping through the dam, the abutments and the foundation.

This control is accompanied by measurements which commence during the construction period and continue during first impounding up to the end of the dam's life. These measurements provide the data for the evaluation of the dam's stability at all phases of construction and operation. Such monitoring covers:

– The control of settlements and horizontal displacements inside the dam and of settlements of the foundation,

– the control of earth pressures and pore-water pressures in the dam,

– the control of joint or pore-water pressure in the foundation,

– the control of the phreatic line in the dam and of ground water fluctuations in the abutments and in the vicinity of the downstream dam toe and

– the histograms of earthquakes and related dam performance.

All the instruments and devices required for such monitoring are installed progressively with the dam construction. Immediately after installation a zero-reading must be made which provides the reference data for the following monitoring process and its digital and graphical presentation. The zero-readings also serve to check the correct function of the instruments.

The instruments are concentrated in selected dam sections. This allows the correlation of the data of different types of instruments. As an example: the developments of earth pressures, of pore-water pressures and of settlements must correspond to each other and to the construction progress. Irregularities point to incorrect functioning of the respective instruments. The concentration of groups of instruments in measuring sections helps to reduce the unavoidable interference of instrument installation and construction progress. It also helps to reduce the risk of damage to the instruments by the construction machinery.

Figure 10.1 shows the typical location of instruments in a measuring section (example Kinda dam). Three sections of this dam have been equipped in this way. Many other dams show similar instrument arrangements.

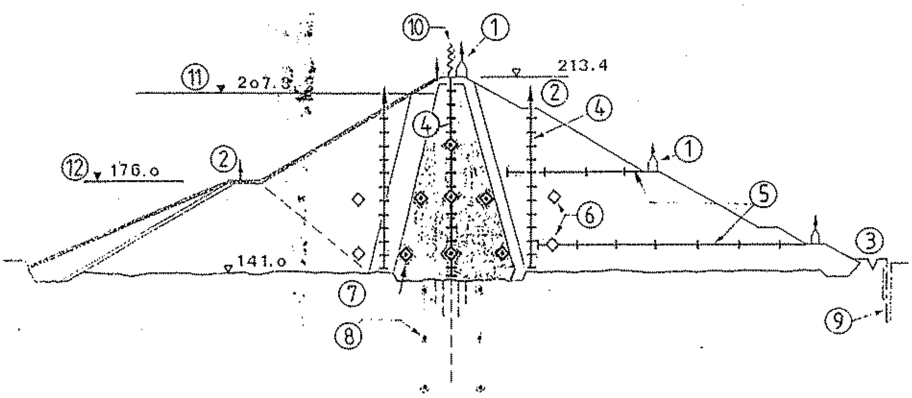


Figure 10.1. Typical instrumentation of a rockfill dam with earth core (example: Kinda dam, archives LI).

1 Surface survey point and measuring chamber
2 Surface survey point
3 Measuring weir for seepage control
4 Settlement gauges
5 Horizontal displacement gauges
6 Earth pressure cells
7 Pore-water pressure and earth pressure cells
8 Borehole piezometer for joint-water pressure
9 Ground water observation hole
10 Seismograph
11 Full supply level
12 Min. operating water level

Along both sides of the crest and on the downstream slope targets are installed for geodetic survey (survey points). The targets on both sides of the crest permit tracing the crest movement during impounding which frequently commences towards upstream, changing towards downstream after impounding to about half the dam height. Other survey points are located on the cofferdam crest and on the upstream slope close to the elevation of the full supply level. The three-dimensional movement of all survey points is controlled from outside survey stations at locations which are reliably firm.

Seepage control is carried out by a measuring weir located at the deepest point of the downstream dam toe. The example of Figure 10.1 does not show facilities to separate seepage through the dam from seepage through the foundation. In this case the seepage through the foundation was considered to be negligible. Examples for separate control are the Aabach dam (Fig. 5.2) and the Frauenau dam (Fig. 7.25). Seepage control means the control of the quantity and of the quality, i.e. existence of suspended particles and chemical composition. Such a quality check is indispensable to detect the development of erosion and dissolution processes. Suspended particles can be detected visually or with the help of filters.

The dam settlements are traced by settlement gauges in the core and in the shells. The markers at the bottom of the gauges are noted which reflect the foundation settlements. The gauges are placed at the location of maximum settlements. The readings can be evaluated with respect to differential movements of adjacent dam zones.

The horizontal displacements inside the dam are controlled by respective gauges in the downstream dam body. The upper gauge is located where relatively large displacements can be expected (Fig. 6.6). The maximum displacements will occur at the crest. They are indicated by the downstream crest survey points.

The earth pressures are monitored by pressure cells in the core and in the lower part of the shells. The instruments should be arranged to measure vertical and horizontal pressures. This is possible when two cells are normal to each other. It is common practice to embed additional pressure cells at selected places of different stresses σ_2 and σ_3 and at interfaces with rigid structures.

The pore-water pressures are monitored in the core. In the rockfill shells and in the generously dimensioned filters there will be no pore pressure. The pressure cells are located at typical elevations, in the center of the core as well as at the outer parts close to the filters. This allows the evaluation of the pore pressure development and subsequent dissipation. The outer instruments will reflect the draining effect of the filters.

The joint-water pressure in the foundation is measured at different elevations on both sides of the grout curtain. Such an arrangement of piezometers enables evaluation of the effectiveness of the curtain. Piezometers downstream of the curtain only would indicate the total pressure loss in the rock foundation upstream and in the curtain. The measurements at Kinda upstream and downstream – together with other indications – gave instructive information on the curtain's effectiveness which can be used for the design of the grout curtain of other projects in similar geological conditions (Kutzner 1996).

The phreatic line after long-term full storage relates to the data of pore pressures in the core. Ground water fluctuations downstream are controlled by ground water observation holes located along the downstream dam toe, from the left abutment across the valley to the right abutment. The Kinda dam is equipped with 19 such observation holes.

A seismograph is installed at the dam crest to register earthquakes there. It is recommended to locate at least one more seismograph at the bottom of the dam or – much better – to locate a number of seismographs in the vicinity of the dam to enable monitoring of the histograms of ground and crest accelerations. At Kinda two more instruments have been located at both the flanks of the Kinda geological fault (Fig. 3.5). The instrument west of the fault will register the ground acceleration which is representative for the dam to a sufficient degree of accuracy.

Simpler examples of dam instrumentations can be seen from Figures 5.1 and 5.3. The homogeneous Shen river dam is not equipped with pore pressure cells. The phreatic line is determined from readings of piezometers in the dam. With the Agus IV dam pore pressure gauges are arranged in the foundation of the downstream shell. They serve to evaluate the hydraulic conditions which relate to the concept of the upstream sealing blanket in connection with the semi-pervious foundation. The standpipe piezometers in the downstream shell serve the same purpose. More examples of dam instrumentation can be found, e.g. from Knight et al. (1985), Moreno & Alberro (1982), De Pablo & Cruz (1985), Fell et al. (1992) or ICOLD (1989b).

In addition to dam monitoring, the movement and the seepage conditions of selected places in the dam's vicinity should be observed. This may apply e.g. to slopes near the dam, to excavations near the power house and to the slopes along the spillway chute. An example of the controlled power house slope is described by the author (1988).

10.2.2 *Types of instruments and their installation*

10.2.2.1 *Survey points*

The targets for geodetic survey are plates or bolts of metal fixed on easily visible concrete structures. The structures must firmly rest on the dam body, free of settlements and turning, and frost-proof.

10.2.2.2 *Seepage*

Usually, the measuring weir at the dam toe is a concrete trough with a V-shaped outlet calibrated for direct reading of the discharge. Alternatively, measurements can be made with the aid of bucket and stop watch. Continuous recording of the discharge and telemetering of the data to a remote control station is practiced with important projects. It must be ensured that all seeping water is conducted to the measuring weir. With some dams the foundation area or the bottom drainage layer is subdivided by impervious barriers. In this case each section must be equipped with its own measuring weir.

Unavoidably, rain water percolating through the downstream dam body and the abutments is conducted to the measuring weir. The different sources of the total discharge, namely seepage through the dam and surface water of dam and abutments, can be identified by evaluating the seepage histogram. Figure 10.2 shows the development of seepage across the Kinda dam and its

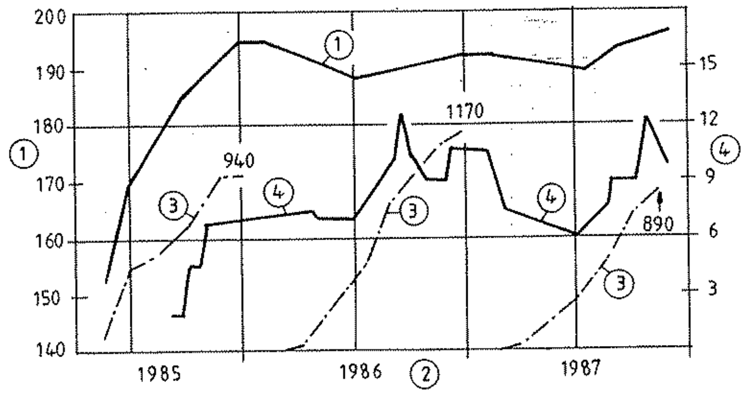


Figure 10.2. Typical seepage histogram (Kinda dam, archives LI).

1 Reservoir water level (m a.s.l.)
2 Years of operation
3 Precipitation (total in mm)
4 Seepage quantity (l/s)

foundation during the first years of operation. The seepage quantity increases with impounding and precipitation in 1985 to a rate of around 7 l/s. At approximately constant storage level this rate does not significantly change until June 1986, i.e. during the dry season of 1986 and the beginning of the next rainy season. With accumulating precipitation in 1986 the quantity increases to around 12 l/s, dropping again to 6 l/s in the following dry season and rising to about 12 l/s in the rainy season of 1987. The repeated change from about 7 to about 12 l/s and vice versa is plausible and reflects satisfactory performance of the dam with respect to seepage. The quantity is tolerable.

The water seeping across artificial internal sealings and across controlled face sealings of asphaltic concrete is usually conducted to an inspection gallery (Figs 7.58 and 7.60). The quantity is measured in one or more sumps. An additional weir downstream is required to collect water seeping through the foundation. The seepage across concrete faces and uncontrolled faces of asphaltic concrete percolates through the dam and must be conducted to a measuring weir at the downstream toe. The requirement of maintenance and cleaning of all measuring systems is noted.

Surface inspection to detect other sources of seepage and wet spots, as described in Section 10.1, is required.

10.2.2.3 *Settlements and horizontal displacements*

Internal deformations are controlled by devices developed particularly for this purpose. A proven system (Fig. 10.3) consists of a string of plastic tubes

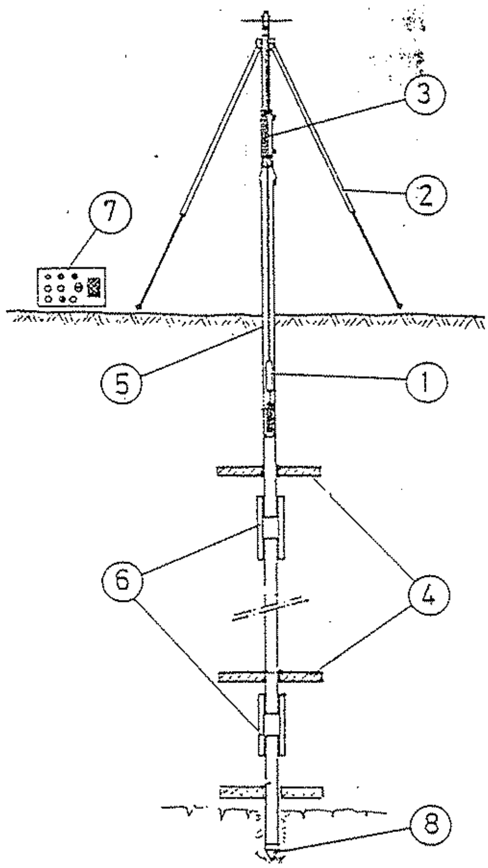


Figure 10.3 Settlement gauge, type Idel (courtesy of Interfels).

1 Radio probe with transmitter cable, measuring tape and weighting piece
2 Tripod with winch for cable and tape
3 Extension tube with scale
4 Steel plates
5 String of plastic tubes
6 Couplings to compensate settlements
7 Receiver and recorder
8 End piece, fixed in the foundation rock

with compensating couplings which are embedded in the dam in a vertical or horizontal position. As dam construction continues, metal plates are put on the string at selected intervals, e.g. 5 m. The plates move along the tube according to the settlement or displacement of the vicinity. The bottom end of vertical strings is fixed to the foundation.

For measuring purposes a torpedo-shaped, short radio probe is moved inside of the string of tubes (Fig. 10.4). It is connected to a transmitter cable and a measuring tape. In vertical strings the probe is moved with a winch on top. In horizontal strings it is moved with a winch at the surface and a guide

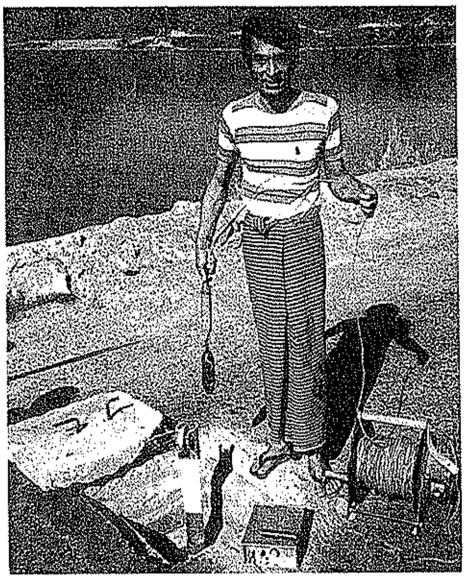


Figure 10.4. Idel radio probe in operation (courtesy of LI). The operator holds the probe above the upper end of the string of plastic tubes in the shaft. In front of him are the cable winch and the receiver.

pulley at the inner end. In most cases the winch is placed in a measuring chamber at the downstream slope of the dam (Fig. 10.1). As soon as the probe passes the metal plates its frequency is altered. The alteration is transmitted to the receiver on the surface. The signal can be made audible and visible. The exact location of the plate can be read from the measuring tape. Accuracy is ± 0.5 to 1.0 cm, which is satisfactory.

In the course of compacting the adjacent dam material the vertical string of tubes is slightly deformed like a corkscrew whereby the settlement readings are slightly falsified. It is recommended to measure the tube's inclination to enable a correction of the settlement readings. According to experience, the inclination measurement includes the risk of new inaccuracies. It is dispensable if the compaction around the tubes is done with light equipment. Then the deformation of the tubes is negligible.

The horizontal string of tubes is embedded in and surrounded by a sand-gravel fill. The string will be deformed due to the settlements of the dam and the foundation. In extreme cases further movement of the probe may be impossible. Then, other measuring systems must be applied, such as e.g. multiple rod extensometers. Given larger settlements, the free movement of the rods may be impeded by the deformed protective hose. It is recommended to combine the extensometers with settlement indicators. Electronic hose levelling instruments (real time electrolevels) are useful for this purpose.

Extensometers up to 120 m in length have also been used to measure lateral strain in dam cores parallel to the dam axis. Such an arrangement is to be seen in the context of dam cracking in Figure 9.45a. An example is the Svartevann dam (ICOLD 1989b, Kjærnsi et al. 1982).

10.2.2.4 *Stresses*

The most common systems of stress measurement are based on the deformation of a hydraulic or pneumatic pressure cell or the deformation of a vibrating wire.

With the hydraulic earth pressure cell (Fig. 10.5) the stress-dependent deformation of the oil filling of the pressure pad is transmitted to the valve diaphragm. The re-forming of the valve, via pump and oil circulating pipe, enables determination of the stress acting on the valve.

With the hydraulic pore-water pressure cell (Fig. 10.6) the water pressure in the ceramic filter – which is equal to the pore-water pressure to be measured – acts on the valve diaphragm. The pressure is determined the same way by re-forming of the valve via pump and oil circulating pipe.

With the vibrating wire system (Fig. 10.7) the vibrating wire in the pressure cell is deformed by stress. This deformation leads to a change of the natural frequency of the wire. The wire is plucked by an impulse; the electro-magnet in the pressure cell transmits the wire's frequency to the outside receiver and recorder. The change in frequency relates to the stress acting on the cell.

The pressure cells are progressively embedded in the dam (Fig. 10.8). The cells located in coarse materials must be surrounded by a protective cushion of sand-gravel. The measured value of stresses is equal to the actual stresses in the vicinity of the cell if the deformation modulus of the cushion is equal to the deformation modulus of the material in the vicinity. Stresses will be concentrated on the cell if the cushion is too stiff. The stress measured ex-

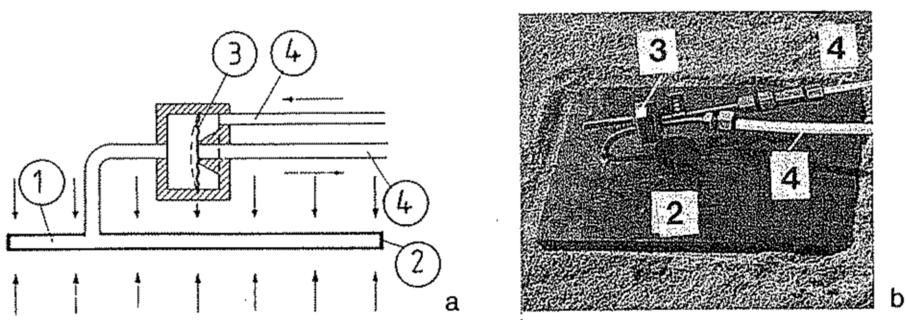


Figure 10.5 Hydraulic earth pressure cell (courtesy of Gloetzl).

a System
b Pressure cell
1 Oil filling
2 Pressure pad 200 × 300 mm
3 Valve diaphragm
4 Oil circulating pipe or hose

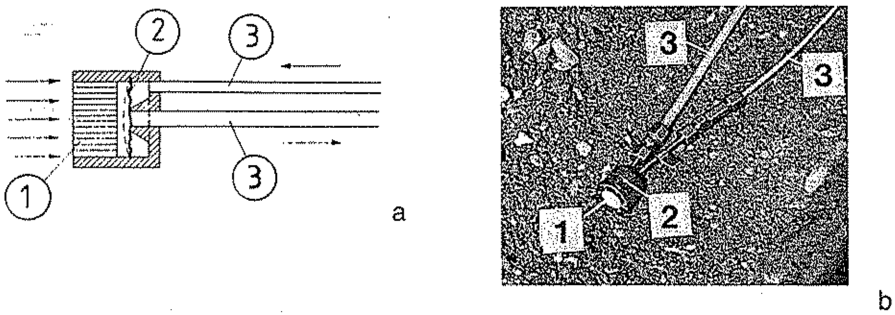


Figure 10.6. Hydraulic pore-water pressure cell (courtesy of Gloetzl).

a System
b Pressure cell
1 Ceramic filter
2 Valve diaphragm
3 Oil circulating pipe or hose

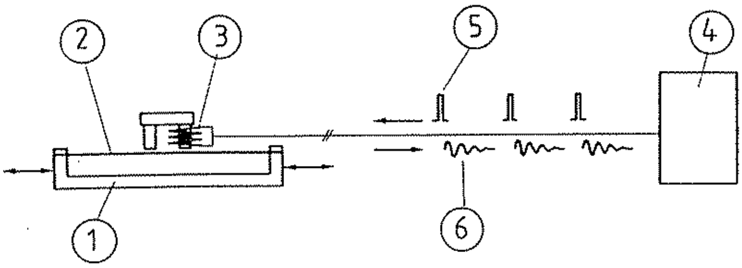


Figure 10.7. Vibrating wire measuring system (courtesy of Maihak).

1 Pressure cell
2 Vibrating wire
3 Electromagnet
4 Receiver and recorder
5 Exciting impulse
6 Resulting oscillation

ceeds the actual stress. In contrast, a supporting arch develops around the cell if the cushion is too weak. The stress measured is less than the actual stress. This is a handicap of the earth pressure measurements in dam materials. The pore-water pressure measurements are not or almost not sensitive to the method of instrument installation.

The connecting cables and pipes are also embedded in protective sand-gravel. Bunches go from the instruments to control stations or to one central station (Fig. 10.9). Protecting the cables, pipes and hoses from damage is a problem at each construction site. Goodwill and care is requested from all people involved. As is known, a part of the instruments will always be damaged during construction and hence never function correctly.

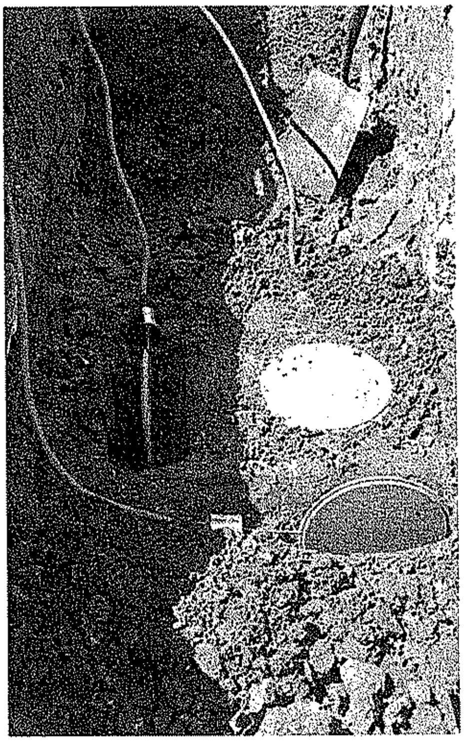


Figure 10.8. Set of hydraulic earth pressure cells for three dimensional measurement.

Center left: pressure cell for lateral earth pressure parallel to dam axis
Center right: pressure cell for vertical earth pressure
Bottom right: pressure cell for lateral earth pressure normal to dam axis

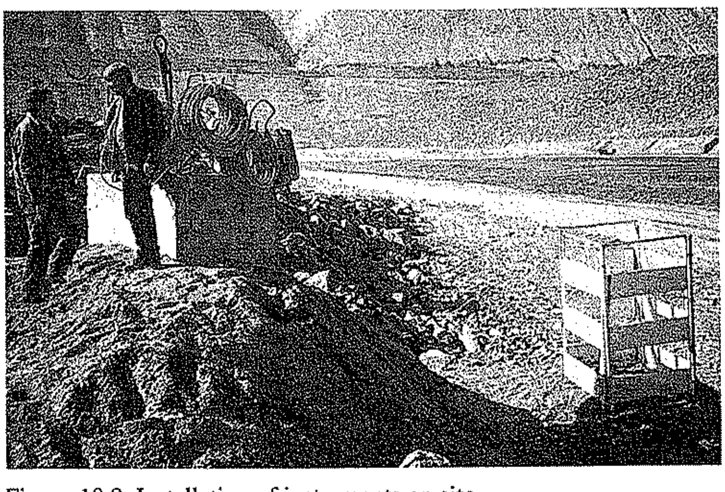


Figure 10.9. Installation of instruments on site.

Left: set of hoses for earth pressure cells to connect the cells with the measuring chamber, protected by concrete block
Right: protective fence for settlement gauge

10.2.2.5 *Joint water and ground water*
The joint and pore-water pressure in the foundation is usually controlled by hydraulic or vibrating wire piezometers in boreholes. Two or more instruments installed in one borehole (Fig. 10.1) must be isolated from each other by clay filling in the borehole sections between them. If there is an inspection gallery available it will be equipped with open boreholes to register the water level with an electrical device lowered into the hole or, if the water is pressurized, with a pressure gauge attached to the standpipe of the borehole. This method of control does not enable a centralized registration of water levels and pressures. The water level in ground water observation holes (Fig. 10.1) is controlled in the same way, namely individually, by lowering the electrical device into the boreholes, or is centralized by firmly installed borehole piezometers.

The measurements made during the impounding of the Kinda reservoir are an instructive example to study the hydraulic conditions of the foundation. The performance of the uppermost piezometers in the boreholes upstream and downstream of the grout curtain and the performance of the ground water in the observation hole in the main section of the dam are shown in Figure 10.10. At the beginning of impounding the foundation water level in the dam axis was approximately equal to the ground water level at the downstream dam toe.

With the rising water level in the reservoir the water level in the piezometers increased, more rapidly upstream of the curtain than downstream. In both the piezometers the water level reached its maximum with delays against the reservoir water level, upstream first and then downstream. The ground water level at the dam toe follows, again with increased delay. The total rise of the ground water level there is about 3 m.

Figure 10.10 reflects the low permeability of the foundation rock and related low effectiveness of the grout curtain. Given greater permeability of the rock and increased effectiveness of the curtain, the maximum water level in the upstream piezometer would be higher and the pressure loss between the upstream and the downstream piezometer would be more.

10.2.2.6 *Deformation control of artificial internal sealings*
The settlements and horizontal displacements of internal sealings of asphaltic concrete are usually controlled. In addition, the transversal strain rate of such membranes is of interest because it is related to the shear deformation, which must not exceed a tolerable limit. The settlements can be monitored with the aid of electronic hose levelling instruments or with targets which are attached to the membrane. Horizontal displacements can be controlled with extensometers. The transversal strain can be determined by measuring the membrane's thickness.

With the Finstertal and Grosse Dhünn dams these measurements are made

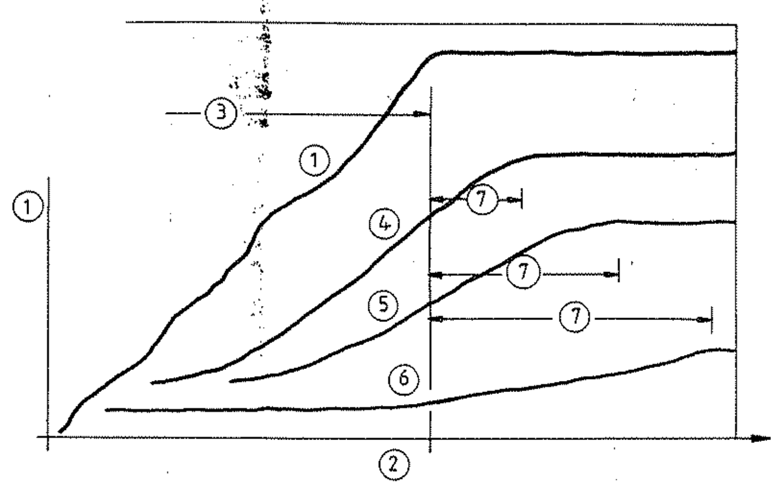


Figure 10.10. Development of joint and ground water tables during first reservoir impounding (example: Kinda dam, instrument locations as in Fig. 10.1, archives LI).

1 Water head
2 Time
3 Period of first impounding
4 Upper piezometer upstream of the grout curtain
5 Upper piezometer downstream of the grout curtain
6 Ground water observation hole at the dam toe
7 Delays

by use of a measuring shaft which is located some meters from the membrane in the downstream shell. All deflections must be correlated with the movements of the shaft which, therefore, must be measured as well, e.g. by geodetic survey, settlement gauges and extensometers. At the two dams, the measurement of the membrane's thickness was performed by different methods which are described and discussed by Breth & Arslan (1990).

At Finstertal magnetic plates were fixed opposite each other at both sides of the membrane. Transversal strain of the membrane causes a change in the magnetic field between the plates, which can be detected and recorded. According to Pircher & Schwab (1988) three arrangements of this type 'have registered all thickness variations during first impounding and 6 subsequent filling cycles without interruptions and with an accuracy of tenths of millimeters'.

At Grosse Dhünn, the measurement of the membrane's thickness was restricted to the downstream half of the membrane. Extension wires are attached to the center and to the downstream face of the membrane. The wires end in the measuring shaft. According to Breth & Arslan the transversal strain of the membrane represented by the movement of the two wires relative to each other could be measured to a satisfactory accuracy. At all measuring points the change in thickness of the half of the membrane was less than 4 mm.

With both methods, it cannot be excluded that the deformation of the adjacent transition zones affects the thickness measurement. Also, the transversal strain may not be equally distributed over the horizontal measuring plane of the membrane. Accordingly, the reporter's conclusions coincide in the statement of Pircher & Schwab: 'Under such circumstances only the boundary zones of the membrane will be affected by dilating, while the center will maintain its initial low porosity and required imperviousness. This should be taken into consideration when selecting the thickness of the membrane'.

A new technique of thickness measurement is described by Haas et al. (1993) and Haug & Wolff (1994). It was applied at the Schmalwasser dam. This technique is based on the physical phenomenon that deformations of a light conducting medium affect the loss of light in a light ray which passes through this medium. The modulation of the light ray is detected and registered by respective sensors and related components. The sensor technique enables continuous monitoring of deformations of asphaltic concrete, starting at a very early phase.

At the Schmalwasser dam the sensor technique has detected that asphaltic concrete is subjected to considerable extension and compression in the period after placement. The extension causes an initial increase of the volume which is followed by compaction due to the weight of the subsequent layers of the dam. During the period of observation – until the end of construction with no impounding – the membrane of Schmalwasser has not shown unacceptable deformations or volume changes.

10.3 CONCLUSIONS

The present experiences of dam monitoring clearly demonstrate:

– Dam instrumentation and monitoring enables the engineers responsible to evaluate the safety of the structure at any time and under any conditions, and

– dam instrumentation and monitoring enables the engineers responsible to widen their knowledge of dam performance.

Having this in mind it is worthwhile to support all efforts done:

– In placing instruments in the dams,

– in evaluating the complete set of data delivered by monitoring, and

– in making all results available to the profession.

In the last decades measuring techniques have been developed to high standard and have been applied increasingly, last but not least because of the increased and justified request for safety by the public. A considerable part of the present knowledge in dam engineering results from the use of such techniques. No doubt the development has been supported by the refined computational methods of finite elements, enabling the engineer to link pure computations with true dam performance – for the benefit of further design and construction of earth and rockfill dams.

Data of the dams described and pictured

Project	Country	Year	Type	Height (m)	Length (m)	Figure	Table
Aabach	Germany	1979	ECERD	45	400	5.2	7.4
Agus IV	Philippines	1985	ECERD	36	350	5.3	7.3
Alcova	Wyoming/USA		ED	68	75	7.16	
Alto Anchicaya	Columbia	1974	CFRD	140	285		7.1
Atatürk	Turkey	1990	ECRD	184	1820	*	
Bakun 1986	Sarawak/Malay-	Design	ECRD	200	900	7.13	
Bakun 1996	sia	Design	CFRD	200	900	7.49	
Balderhead	UK	1975	ED	50		7.34	
Bolgenach	Austria	1979	ECGD	100	220	7.41	
Castagnara	Italy	1992	ECRD	100	600	9.13	
Castello	Italy	1982	AFRD	50	780	*	
Chico	Philippines	Design	ECRD	160	620	7.14 8.4 8.8	4.10
Chicoasen	Mexico	1980	ECRD	261	485	3.3	
Colbun	Chile	1984	ECGD	116	560		7.4
Dartmouth	Australia	1979	ECRD	180	670		7.4
Eberlaste	Austria	1968	ACED	28	480	7.51 7.52	
Finstertal	Austria	1980	ACRD	149	652	7.42	4.10 7.1 7.10
Foz do Areia	Brazil	1980	CFRD	160	828	7.44	7.1
Frauenau	Germany	1983	ECRD	75	640	7.25	
Förmitz	Germany	1976	ECERD	33	800	7.24	

Project	Country	Year	Type	Height (m)	Length (m)	Figure	Table
Grosse Dhünn	Germany	1980	AFRD	58	400	5.6	4.10 7.10
Inamura	Japan	1982	ECRD	88	350	3.2	
Kinda	Burma	1986	ECRD	75	625	5.4 9.22	4.10 7.4 7.7
LG-4	Canada	1984	ECRD	128	3750		7.4
Manicouagan 3	Canada	1974	ECGD	107	885	7.64	
Mauthaus	Germany	1972	ECRD	61	290		9.2
Megget	UK	1982	ACGD	56	570		7.10
Mica	Canada	1973	ECGD	242	790	7.20	
Monasavu	Fiji	1983	ECRD	85	460		7.4 7.7
Mornos	Greece	1976	ECGD	126	815	6.8	
Nantahala	USA	1942	ED	80		7.22	
Nipawin							
West dam	Canada	1985	ED	40	170	7.43	
Main dam	Canada	1985	ED	54	400	7.43	
Nurek	USSR	1980	ECGD	300	704	*	
Obernau	Germany	1972	AFRD	60	300	5.7	
Oroville	California/USA	1968	ECGD	230	2120		4.10
Peruca	Yugoslavia	1959	ECGD	60			7.1
Poza Onda	Ecuador	1971	AFED	39	310		
Prims (Nonnweiler)	Germany	1982	AFRD	62	306	*	
Pueblo Viecho	Guatemala	1983	ECRD	110	250		7.4
Punchina Cofferdam	Columbia	1980	ED	45		7.11	7.3
Rogun	USSR	1991	ECGD	335	660	*	
Sadd-el-Kafara	Egypt	2600 BC	ECRD	14	110	2.1	
Salvajina	Columbia	1984	CFRD	148	360	7.62	
San Fernando							
Lower dam	California/USA	1930	ECSD	40		*	
Scammonden	UK	1969	ECRD	70			7.4
Schmalwasser	Germany	1992	ACRD	80	325	*	
Shen river	Nigeria	1979	ED	36	1400	5.1 7.8	7.3
Srinagarind	Thailand	1978	ECRD	140	610		7.4 7.7

Project	Country	Year	Type	Height (m)	Length (m)	Figure	Table
Sugar Pine	California/USA	1981	ECRD	57	180	*	
Svartevann	Sweden	1976	ECRD	129	420	*	
Sylvenstein	Germany	1958	ECGD	42	180	3.4 9.43	
Talbingo	Australia	1971	ECRD	162	700	7.12	
Tarbela	Pakistan	1974	ECRD	162	2740	*	
Tataragi	Japan	1973	AFRD	65	300	*	
Teton	Idaho/USA	1976	ED	120	950	8.3	
Wister	Oklahoma/USA	1948	ED	30	1800	9.42	
Wupper	Germany	1987	ACRD	40	280		7.1

ACED = Asphaltic core earth dam
ACGD = Asphaltic core gravel dam
ACRD = Asphaltic core rockfill dam
AFED = Asphaltic face earth dam
AFRD = Asphaltic face rockfill dam
CFRD = Concrete face rockfill dam
ECERD = Earth core earth and rockfill dam
ECGD = Earth core gravel dam
ECSD = Earth core sand dam
ECRD = Earth core rockfill dam
ED = Earth dam
*see subject index 'dams by name'

References

ABBREVIATIONS

ASCE: American Society of Civil Engineers, New York, NY, USA
BVFA: Bundesversuchs- und Forschungsanstalt Arsenal, Wien, Österreich (Federal Center for Research and Testing, Geotechnical Institute, Department of Soil Mechanics and Applied Geotechnics, Vienna, Austria)
DGEG: Deutsche Gesellschaft für Erd- und Grundbau e.V., since 1994: Deutsche Gesellschaft für Geotechnik e.V. (German Geotechnical Society), Essen, Germany
DIN: Deutsches Institut für Normung e.V., Berlin, Germany
DNK: Deutsches Nationales Komitee für Große Talsperren in der Bundesrepublik Deutschland, since 1991: Deutsches TalsperrenKomitee (German Committee on Large Dams), Düsseldorf, Germany
DTK: see DNK
DVWK: Deutscher Verband für Wasserwirtschaft und Kulturbau e.V. (German Association for Water Resources and Land Improvement), Bonn, Germany
EERC: Earthquake Engineering Research Center, University of California, Berkeley, Ca., USA
ENR: Engineering News Record
FGSV: Forschungsgesellschaft für das Straßen- und Verkehrswesen (Road and Transportation Research Association), Cologne, Germany
IBF: Institut für Boden- und Felsmechanik, Universität Karlsruhe (Institute of Soil Mechanics and Rock Mechanics, University of Karlsruhe, Germany)
ICOLD: International Commission on Large Dams, Paris, France
ISRM: International Society for Rock Mechanics, Lisbon, Portugal
ISSMFE: International Society for Soil Mechanics and Foundation Engineering, Cambridge, UK
LI: Lahmeyer International GmbH, Frankfurt, Germany
SAA: Standards Association of Australia
USBR: United States Bureau of Reclamation, Denver, Co., USA
USCE: United States Corps of Engineers, Vicksburg, Miss., USA
USCOLD: United States Committee on Large Dams, Denver, Co., USA

Ambraseys, N.N. & Sarma, S.K. (1967). The response of earth dams to strong earthquakes. *Géotechnique* 17: 181-213.

Arnevik, A., Kjærnsli, B. & Walb, S. (1988). The Storvatn dam. A rockfill dam with central core of asphaltic concrete. *16th ICOLD-Congress, San Francisco,* II: 141-158.

Arrau, L., Ibarra, I. & Noguera, G. (1985). Performance of Cogoti dam under seismic loading, in J.B.Cooke & J.L.Sherard (1985) (eds): 1-14.

Baldovin, G., Allaria, N. & Baldovin, E. (1991). The core of Castagnara dam on Metramo river. *17th ICOLD-Congress, Vienna,* IV: 179-205.

Balissat, M. & Wilhelm, R. (1985). Design and Construction of the Pueblo Viecho dam. *Water Power & Dam Construction* 38 (July): 65-70.

Bawag (1986). Schmaldichtwände am Lech. *Wasserwirtschaft* 76: 545-612.

Beier, H. & List, F. (1982). Trench diaphragms as sealing elements in earth dams. *14th ICOLD-Congress, Rio de Janeiro,* IV: 215-227.

Beier, H., List, F. & Lorenz, W. (1979). Subsequent sealing in the impervious core of the Sylvenstein dam. *13th ICOLD-Congress, New Delhi,* II: 103-116.

Beier, H. & Strobl, Th. (1985). Resistance against internal erosion of various types of cut off walls in dam construction. *15th ICOLD-Congress, Lausanne,* III: 417-428.

Bernell, L. (1982). Experiences of wet compacted dams in Sweden. *14th ICOLD-Congress, Rio de Janeiro,* IV: 421-431.

Bertram, G.E. (1967). Experience with seepage control measures in earth and rockfill dams. *9th ICOLD-Congress, Istanbul,* III: 91-109.

Bertram, G.E. (1973). Field tests for compacted rockfill, in *Embankment Dam Engineering,* Casagrande Volume, R.C.Hirschfeld & S.J.Poulos (eds), Wiley, 1-19.

Beyer, W. (1964). Zur Bestimmung der Wasserdurchlässigkeit von Kiesen und Sanden aus der Kornverteilungskurve. *Wasserwirtschaft und -technik* 14: 165-168.

Bigalli, F., Dolcimascolo, F., Japelli, R., Pezzini, R., Toti, M. & Valore, C. (1980). *Rassegna delle principali dighe in Sicilia,* Volume issued at the occasion of 48th ICOLD Executive Meeting, Roma.

Bishop, A.W. (1955). The use of slip circle in the stability analysis of slopes. *Géotechnique* 5: 7-17.

Brauns, J. (1978). Wirksamkeit unvollkommener Abdichtungswände unter Staubauwerken. *Veröffentlichungen des Institutes für Bodenmechanik und Felsmechanik der Technischen Hochschule Fridericiana in Karlsruhe* 80.

Brauns, J. (1985). Erosionsverhalten geschichteten Bodens bei horizontaler Durchströmung. *Wasserwirtschaft* 75: 448-453.

Brauns, J. (1987). Auswirkungen von Fehlstellen in Dichtwänden. *Mitteilungen des Institutes für Bodenmechanik und Grundbau Technische Universität Braunschweig* 23: 215-232.

Brauns, J. (1990). Filters and drains. *Proceedings of the NATO Advanced Study Institute on Advances in Rockfill Structures, Lisbon.* Kluwer Academic Publishers, Dordrecht (1991): 239-266.

Brauns, J., Degen, F.P. & Armbruster, H. (1988). The leakage problem with thin dam seals. *16th ICOLD-Congress, San Francisco,* II: 125-140.

Breth, H. (1972). Der derzeitige Stand des Staudammbaues. *Wasserwirtschaft* 62: 1-14.

Breth, H. (1976). Aktuelle baugrundtechnische Probleme für den Kraftwerksbau. *VGB Kraftwerkstechnik* 56: 407-414.

Izhar-ul-Haq (1991). Foundation problems of Tarbela dam project. *17th ICOLD-Congress, Vienna,* III: 1353-1369.

Jansen, R.B. (1988) (ed.). *Advanced dam engineering for design, construction and rehabilitation.* Van Nostrand Reinhold, New York.

Jenner, M. *Jemen neu entdeckt.* Longman, London.

Kanchanaphol, L., Mahasandana, T. & Bucharoen, V. (1982). Geology of Srinagarind dam, foundation treatment, and evaluation of grouting efficiency. *14th ICOLD-Congress, Rio de Janeiro,* II: 7-35.

Kawasaki, T., Niina, A., Saitoh, S., Suzuki, Y. & Honjyo, Y. (1981). Deep mixing method using cement hardening agent. *ISSMFE-Congress, Stockholm,* 3: 721-724.

Kjærnsli, B., Kvale, G. & Lunde, J. (1982). Design, construction control and performance of the Svartevann Earth-rockfill dam. *14th ICOLD-Congress, Rio de Janeiro,* IV: 319-337.

Knight, D.J., Naylor, D.J. & Davis, P.D. (1985). Stress-strain behaviour of the Monasavu soft core rockfill dam: Prediction, performance, and analysis. *15th ICOLD-Congress, Lausanne,* I: 1299-1326.

Knight, D.J., Worner, N.M. & McClung, J.E. (1982). Materials and construction methods for a very wet clay core rockfill dam at Monasavu Falls, Fiji. *14th ICOLD-Congress, Rio de Janeiro,* IV: 293-303.

Knodel, P.C. (1988). Dispersive clays, in Jansen, R.B. (1988) (ed.): 176-182.

Kretzke, W. (1969). *Einführung in die Geophysik, Bd. 1.* Bibliographisches Institut, Mannheim.

Kropatschek, H. & Rienössl, K. (1970). The vertical asphaltic concrete core of earthfill dam Eberlaste of the Zemm hydro-electric scheme. *10th ICOLD-Congress, Montreal,* I: 233-245.

Kuhlmann, W. (1989). Bituminous core sealing in dams - developping the placing technique. *Strabag Schriftenreihe Asphalt-Wasserbau* 11, Folge 1, 2. Ausgabe, Teil B: 81-87.

Kulesca, R.L. (1988): Design of upstream facing. *16th ICOLD-Congress, San Francisco,* V: 203-210.

Kutzner, C. (1962). Über die Vorgänge in körnigen Schüttungen bei der Rüttelverdichtung. *Veröffentlichungen des Institutes für Bodenmechanik und Grundbau der Technischen Hochschule Fridericiana in Karlsruhe* 9.

Kutzner, C. (1980). Entwicklung der erforderlichen Standsicherheitsbeiwerte für einen Erddamm und für die seitlichen Böschungen am Beispiel des Mornos-Dammes. Vorträge der Baugrundtagung Mainz, DGEG: 465-484, DGEG and *Die Bautechnik* 59: 24-27.

Kutzner, C. (1982a). Interaction of embankment dam control and first reservoir filling. *14th ICOLD-Congress, Rio de Janeiro,* I: 1-15.

Kutzner, C. (1982b). The value of inspection galleries. *Water Power & Dam Construction* 34 (May): 79-84.

Kutzner, C. (1985). Design and construction of earthquake-resistant earth and rockfill dams. *Geotechnical Engineering, Organ of the Southeast Asian Geotechnical Society* 16: 1-23 and Vorträge der Baugrundtagung 1984, Düsseldorf, DGEG: 609-638.

Kutzner, C. (1988). Impact of reservoir filling on the environment close downstream (Kinda dam Burma). *16th ICOLD-Congress, San Francisco,* I: 113-125.

Kutzner, C. (1996). *Grouting of Rock and Soil.* Balkema, Rotterdam.

Hollingworth, F. & Druyts, F.H.W.M. (1982). Filter cloth partially replaces and supplements filter materials for protection of poor quality core material in rockfill dam. *14th ICOLD-Congress, Rio de Janeiro*, IV: 709-725.
ICOLD (1974). *Lessons from Dam Incidents*. ICOLD, Paris.
ICOLD (1975). A review of earthquake resistant design of dams. *ICOLD Bulletin* 27.
ICOLD (1979). Interface problems of dams. *13th ICOLD-Congress, New Delhi, I, Q. 48.*
ICOLD (1982). Bituminous concrete facings for earth and rockfill dams. *ICOLD Bulletin* 32a.
ICOLD (1983). Seismicity and dam design. *ICOLD Bulletin* 46.
ICOLD (1985a). Dam and foundation monitoring. *15th ICOLD-Congress, Lausanne, I, Q. 56.*
ICOLD (1985b). Filling materials for watertight cut off walls. *ICOLD Bulletin* 51.
ICOLD (1986a). Static analysis of embankment dams. *ICOLD Bulletin* 53.
ICOLD (1986b). Earthquake analysis procedures for dams. *ICOLD Bulletin* 52.
ICOLD (1986c). Soil-cement for embankment dams. *ICOLD Bulletin* 54.
ICOLD (1986d). Quality control for fill dams. *ICOLD Bulletin* 56.
ICOLD (1989a). Rockfill dams with concrete facing. *ICOLD Bulletin* 70.
ICOLD (1989b). Monitoring of dams and their foundations. *ICOLD Bulletin* 68.
ICOLD (1992). Bituminous cores for fill dams. *ICOLD Bulletin* 84.
Idel, K.-H. (1960). Die Scherfestigkeit rolliger Erdstoffe. *Veröffentlichungen des Institutes für Bodenmechanik und Grundbau der Technischen Hochschule Fridericiana in Karlsruhe* 2.
Idel, K.-H. (1981). Aabachtalsperre – Erddamm mit Kerndichtung und horizontalem Dichtungsteppich. *Wasserwirtschaft* 71: 105-109.
Idel, K.-H. & Stöhr, E. (1982). Influence of geomechanical investigation on comparison of a rockfill dam with upstream membrane and earthfill dam with sealing core. *14th ICOLD-Congress, Rio de Janeiro*, II: 309-324.
Idriss, I.M. & Duncan, J.M. (1988). Earthquake response analysis of embankment dams, in Jansen, R.B. (1988) (ed.).
Innerhofer, G. (1980). Der 100 m hohe Kiesdamm Bolgenach mit Moränenkern. *Wasserwirtschaft* 70: 129-133.
Innerhofer, G. & Loacker, H. (1982). The connexion of the core of the Bolgenach dam to weathered marl. *14th ICOLD-Congress, Rio de Janeiro*, II: 275-287.
Ishihara, K. (1985). Stability of natural deposits during earthquakes. *ISSMFE-Conference, San Francisco*, I: 321-327.
ISRM (1977). Suggested methods for determining hardness and abrasiveness of rocks. ISRM-Committee on Laboratory Tests, Document no 5, in Brown, E.T. (1981) (ed.): 95-103.
ISRM (1979). Suggested methods for determining water content, porosity, density absorption and related properties and swelling and slake-durability index properties. ISRM-Committee on Laboratory tests, Document no 2, in Brown, E.T. (1981) (ed.): 79-94.
ISRM (1985). Suggested method for determining point load strength. *Int. Journ. Rock Mech. Min. Sci. & Geomech. Abstr.* 22: 51-60.
Iverson, L.N. & Ringheim, A.S. (1973). Upstream slope protection at Gardiner and Qu'Appelle river dams. *11th ICOLD-Congress, Madrid*, III: 655-681.

Izhar-ul-Haq (1991). Foundation problems of Tarbela dam project. *17th ICOLD-Congress, Vienna,* III: 1353-1369.

Jansen, R.B. (1988) (ed.). *Advanced dam engineering for design, construction and rehabilitation.* Van Nostrand Reinhold, New York.

Jenner, M. *Jemen neu entdeckt.* Longman, London.

Kanchanaphol, L., Mahasandana, T. & Bucharoen, V. (1982). Geology of Srinagarind dam, foundation treatment, and evaluation of grouting efficiency. *14th ICOLD-Congress, Rio de Janeiro,* II: 7-35.

Kawasaki, T., Niina, A., Saitoh, S., Suzuki, Y. & Honjyo, Y. (1981). Deep mixing method using cement hardening agent. *ISSMFE-Congress, Stockholm,* 3: 721-724.

Kjærnsli, B., Kvale, G. & Lunde, J. (1982). Design, construction control and performance of the Svartevann Earth-rockfill dam. *14th ICOLD-Congress, Rio de Janeiro,* IV: 319- 337.

Knight, D.J., Naylor, D.J. & Davis, P.D. (1985). Stress-strain behaviour of the Monasavu soft core rockfill dam: Prediction, performance, and analysis. *15th ICOLD-Congress, Lausanne,* I: 1299-1326.

Knight, D.J., Worner, N.M. & McClung, J.E. (1982). Materials and construction methods for a very wet clay core rockfill dam at Monasavu Falls, Fiji. *14th ICOLD-Congress, Rio de Janeiro,* IV: 293-303.

Knodel, P.C. (1988). Dispersive clays, in Jansen, R.B. (1988) (ed.): 176-182.

Kretzke, W. (1969). *Einführung in die Geophysik, Bd. 1.* Bibliographisches Institut, Mannheim.

Kropatschek, H. & Rienössl, K. (1970). The vertical asphaltic concrete core of earthfill dam Eberlaste of the Zemm hydro-electric scheme. *10th ICOLD-Congress, Montreal,* I: 233-245.

Kuhlmann, W. (1989). Bituminous core sealing in dams - developping the placing technique. *Strabag Schriftenreihe Asphalt-Wasserbau* 11, Folge 1, 2. Ausgabe, Teil B: 81-87.

Kulesca, R.L. (1988): Design of upstream facing. *16th ICOLD-Congress, San Francisco,* V: 203-210.

Kutzner, C. (1962). Über die Vorgänge in körnigen Schüttungen bei der Rüttelverdichtung. *Veröffentlichungen des Institutes für Bodenmechanik und Grundbau der Technischen Hochschule Fridericiana in Karlsruhe* 9.

Kutzner, C. (1980). Entwicklung der erforderlichen Standsicherheitsbeiwerte für einen Erddamm und für die seitlichen Böschungen am Beispiel des Mornos-Dammes. Vorträge der Baugrundtagung Mainz, DGEG: 465-484, DGEG and *Die Bautechnik* 59: 24- 27.

Kutzner, C. (1982a). Interaction of embankment dam control and first reservoir filling. *14th ICOLD-Congress, Rio de Janeiro,* I: 1-15.

Kutzner, C. (1982b). The value of inspection galleries. *Water Power & Dam Construction* 34 (May): 79-84.

Kutzner, C. (1985). Design and construction of earthquake-resistant earth and rockfill dams. *Geotechnical Engineering, Organ of the Southeast Asian Geotechnical Society* 16: 1- 23 and Vorträge der Baugrundtagung 1984, Düsseldorf, DGEG: 609-638.

Kutzner, C. (1988). Impact of reservoir filling on the environment close downstream (Kinda dam Burma). *16th ICOLD-Congress, San Francisco,* I: 113-125.

Kutzner, C. (1996). *Grouting of Rock and Soil.* Balkema, Rotterdam.

Kutzner, C. & Hönisch, K. (1985). Beispiel für die Berechnung und den Entwurf eines hohen Dammes unter Erdbebeneinfluß. Vortrag zur Jahrestagung der Deutschen Gesellschaft für Erdbeben-Ingenieurwesen und Baudynamik, Stuttgart.

Kutzner, C., Hönisch, K. & Hein, B.R. (1988). Computergestützter Entwurf und Rückrechnung für einen Steinschüttdamm mit Erdkern. Tagung Finite Elemente- Anwendungen in der Baupraxis, Ruhruniversität, Bochum, Ernst & Sohn: 255-266.

Lazanyi, I. (1986). The use of decomposed granite rock for dam construction. *Water Power & Dam Construction* 38 (July): 25-28.

Le Coroller, A., Bienaimé, C., Herment, R., Poupart, M., Huynh, P., Crepel, J.M., Moderassi, A. & Bard, E. (1988). Approach to bituminous concrete core behaviour by finite element model computation. *16th ICOLD-Congress, San Francisco,* II: 997- 1027.

Lehnert, J. & Geiseler, W.D. (1979). High Island water scheme Hong Kong. Bituminierte Kerndichtung für zwei 100 m hohe Dämme. *Strabag Schriftenreihe Asphalt- Wasserbau* 10, Folge 1: 111-131.

Leps, Th.M. (1988a). Failure of Teton dam, in Jansen, R.B. (1988) (ed.): 34-41.

Leps, Th.M. (1988b). Rockfill dam design and analysis, in Jansen, R.B. (1988) (ed.): 368- 387.

Leussink, H. (1960). Bau eines großen dreiaxialen Schergerätes zur Untersuchung grobkörniger Erdstoffe. *Veröffentlichungen des Institutes für Bodenmechanik und Grundbau der Technischen Hochschule Fridericiana in Karlsruhe* 1: 31-61.

Leussink, H. & Kutzner, C. (1962). Laboratoriumsversuch zur Feststellung der dichtesten Lagerung körniger Erdstoffe. *Veröffentlichungen des Institutes für Bodenmechanik und Grundbau der Technischen Hochschule Fridericiana in Karlsruhe* 8.

Leussink, H., Wisweswaraiya, T.G. & Brendlin, H. (1964). Beitrag zur Kenntnis der bodenphysikalischen Eigenschaften von Mischböden. *Veröffentlichungen des Institutes für Bodenmechanik und Grundbau der Technischen Hochschule Fridericiana in Karlsruhe* 15.

List, F. (1980). Erfahrungen beim Bau und Betrieb der Förmitztalsperre mit Dammdichtung und Untergrundabdichtung. *Wasserwirtschaft* 70: 124-128.

List, F. (1987). Geotextilien im Staudamm Frauenau als Sicherheitselement und für Meß- und Kontrollzwecke. *Wasserwirtschaft* 77: 361-365.

List, F. & Sadgorski, W. (1981). Die Risse im Kern des Sylvensteindammes. Sanierung und Entstehungsnachweis nach der FEM. *Geotechnik, Organd der DGEG* 4: 16-24.

List, F. & Strobl, Th. (1991). Changing of tightness in the grout curtain and core of the Sylvenstein dam by ageing. *17th ICOLD-Congress, Vienna,* II: 1295-1309.

Londe, P. (1983). 10-point plan for dam safety. *World Water,* January: 12.

Lorenz, W. (1966). Messungen und Beobachtungen am Sylvenstein-Staudamm. *Wasserwirtschaft* 56: 33-41.

Lorenz, W. (1973). The sealing element of the Mauthaus drinking water reservoir. *11th ICOLD-Congress, Madrid,* III: 81-99.

Lorenz, W. (1976). Plastische Dichtungswände bei Staudämmen. Vorträge der Baugrundtagung Nürnberg, DGEG: 389-407.

Lorenz, W. & List, F. (1976). Application of the trench diaphragm method in constructing the impervious core of dams consisting in part of low-grade fill material. *12th ICOLD- Congress, Mexico,* I: 93-103.

Lorson, H.-R., Keitel, H.-P. & Hermann, E. (1983). Primstalsperre Nonnweiler – ein neuer Abschnitt in der Wasserwirtschaft des Saarlandes. *Wasserwirtschaft* 73: 103-108.

Lowe III, J. (1988). Seepage analysis, in Jansen, R.B. (1988) (ed.): 270-275 and USCOLD (1993).

Luebke, T.A., McClain, J. & Farrar, J.A. (1991). Application of SMW method to Jackson Lake dam modifications. *17th ICOLD-Congress, Vienna,* III: 1159-1177.

Lugeon, M. (1933). *Barrages et Géologie.* Réimpression photostatique 1979, Poliografico Pedrazzini, Locarno.

Maksimovic, M. (1973). Optimum position of the central clay core in respect to arching and hydraulic fracture. *11th ICOLD-Congress, Madrid,* III: 789-800.

Mallet, C. & Pacquant, J. (1951). *Les barrages en terre.* Eyrolles, Paris.

Marachi, W.D., Clarence, K.C. & Seed, H.B. (1972). Evaluation of properties of rockfill materials. *Journal of the Soil Mechanics and Foundation Division* 98, no SM 1, ASCE: 95-114.

Marsal, R.J. (1973). Mechanical properties of rockfill, in *Embankment Dam Engineering,* Casagrande Volume, R.C.Hirschfeld & S.J.Poulos (eds), Wiley, 109-200.

McConnell, A.D., Seemel, R.N. & O'Beirne, H.J. (1973). Freeboard and slope protection of dykes for the Churchill Falls project. *11th ICOLD-Congress, Madrid,* III: 683-705.

Militzer, H. & Weber, F. (1987) (eds). *Angewandte Geophysik, Band 3, Seismik.* Springer, Wien.

Militzer, H., Schön, J., Stötzner, U. & Stoll, R. (1978). *Angewandte Geophysik im Ingenieur- und Bergbau.* VEB Deutscher Verlag für Grundstoffindustrie, Leipzig.

Moreno, A. & Alberro, J. (1982). Behaviour of Chicoasen dam: Construction and first filling. *14th ICOLD-Congress, Rio de Janeiro,* I: 155-182.

Morgenstern, N.R., Matheson, D.S. & MacPherson, J.G. (1991). Influence of foundation conditions on design and construction of the Nipawin hydroelectric project. *17th ICOLD-Congress, Vienna,* III: 1775-1802.

Morgenstern, N.R. & Price, V.E. (1965). The Analysis of the stability of general slip surfaces. *Géotechnique* 15: 79-93.

Morin, H.W. & Todor, P.C. (1976). *Laterite and Lateritic Soils and other Problem Soils in the Tropics.* Lyon Associates Inc., Baltimore.

Müller, L. (1963). *Der Felsbau, Band 1.* Enke, Stuttgart.

Müller-Salzburg, L. (1992). *Der Felsbau, Band 2, Teil A.* Enke, Stuttgart.

Murley, K.A. & Cummins, P.J. (1982). Design considerations of materials during construction of Dartmouth dam, Australia. *14th ICOLD-Congress, Rio de Janeiro,* IV: 627-641.

Nakayama, K., Itoga, F. & Inoue, Y. (1982). Selection and quality control of materials for rockfill dam of pumped storage project in phyllocristalline schistose area. *14th ICOLD-Congress, Rio de Janeiro,* IV: 23-46.

Neuhauser, E. (1970). Das bauliche Wagnis bei den Tiefbauarbeiten des Kaunertalkraftwerkes. *Österreichische Ingenieur-Zeitschrift* 115, Hefte 3, 4 und 5.

Newmark, N.M. (1965). Effects of earthquakes on dams and embankments. *Géotechnique* 15: 139-160.

Nilsson, Å. & Norstedt, U. (1991). Evaluation of ageing process on two Swedish dams. *17th ICOLD-Congress, Vienna,* II: 23-47.

Nonveiller, E. (1987). Discussion on Dascal, O. (1987).

Nonveiller, E. (1989). *Grouting Theory and Practice.* Elsevier, Amsterdam.
Penman, A.D.M. (1977). The failure of Teton dam. *Ground Engineering* 10: 18-27.
Penman, A.D.M. (1982). Materials and construction methods for embankment dams and cofferdams. General Report Q. 55. *14th ICOLD-Congress, Rio de Janeiro,* IV: 1105- 1228.
Penman, A.D.M. & Charles, J.A. (1973). Effect of the position of the core on the behaviour of two rockfill dams. *11th ICOLD-Congress, Madrid,* III: 315-339.
Penman, A.D.M. & Charles, J.A. (1985). A comparison of observed and predicted deformations of an embankment dam with a central asphaltic core. *15th ICOLD-Congress, Lausanne,* I: 1373-1389.
Pinto, Nelson L. de S., Materon, B. & Lagos Marques, P. (1982). Design and performance of Foz do Areia concrete membrane as related to basalt properties. *14th ICOLD- Congress, Rio de Janeiro,* IV: 873-906.
Pircher, W. & Schwab, H. (1988). Design, construction and behaviour of the asphaltic concrete core wall of the Finstertal dam. *16th ICOLD-Congress, San Francisco,* II: 901-924.
Poweleit, A. (1985). Bemessung des Freibords im Erddammbau. *Wasserwirtschaft* 75: 434- 439.
Proctor, R.R. (1933). Design and construction of rolled earth dams. *ENR* 111: 254.
Prusza V., Z. & Choudry, T. (1982). Moisture control of residual soils for large dams. *14th ICOLD-Congress, Rio de Janeiro,* IV: 167-183.
Rajcevic, B.M. (1970). Drainage core. *10th ICOLD-Congress, Montreal,* I: 423-435.
Regalado, G., Materon, B., Ortega, J.W. & Vargas, J. (1982). Alto Anchicaya concrete face rockfill dam. Behaviour of the concrete face membrane. *14th ICOLD-Congress, Rio de Janeiro,* IV: 517-535.
Rizzoli, I.-L., Le Moullec, E., Reverchon, B., Isambert, F. & Dupas, J.-M. (1991). Observations on compressible foundation behaviour and very compressible foundation design: Dams of Seine, Aube and Marne reservoirs. *17th ICOLD- Congress, Vienna,* III: 105-134.
Rogers, R.L. & Pearce, K.G. (1991). Effect of the foundations on design, construction and filling of Split-Yard Creek dam. *17th ICOLD-Congress, Vienna,* III: 1577-1599.
Rutledge, P.C. & Gould, J.P. (1973). Movements of articulated conduits under earth dams on compressible foundations, in *Embankment Dam Engineering,* Casagrande Volume, R.C.Hirschfeld & S.J.Poulos (eds), Wiley, 209-237.
SAA (1980). Determination of Emerson class number of a soil. AS 1289, C8.1.
Saitoh, S., Suzuki, Y. & Shirai, K. (1985). Hardening of soil improved by deep mixing method. *11th ISSMFE-Congress, San Francisco,* 3: 1745-1748.
Sarma, S.K. (1975). Seismic stability of earth dams and embankments. *Géotechnique* 25: 741-761.
Schmidt, M. (1989). Die Wasserwirtschaft des Oberharzer Bergbaues. *Schriftenreihe der Frontius-Gesellschaft* 13.
Schnitter, N.J. (1987). Verzeichnis geschichtlicher Talsperren bis Ende des 17. Jahrhunderts, in Historische Talsperren, DVWK: 9-20.
Schober, W. (1982). Concrete core diaphragm walls for high embankment dams. *14th ICOLD-Congress, Rio de Janeiro,* IV: 201-214.
Schober, W. (1988). A study on the behaviour of embankment dams with diaphragms. *16th ICOLD-Congress, San Francisco,* II: 973-996.

Schober, W. & Lercher, H. (1985). The concrete core diaphragm wall of the embankment dam Bockhartsee monitoring and interpretation. *15th ICOLD-Congress, Lausanne,* I: 1257-1272.

Schober, W. & Rostek, R. (1979). The shear resistance in the interface of the fill material or the rock overburden and the bedrock. *13th ICOLD-Congress, New Delhi,* I: 989-1004.

Schultze, E. & Muhs, H. (1967). *Bodenuntersuchungen für Ingenieurbauten.* 2. Auflage, Springer, Berlin.

Schwab, H. (1984). Analyse der Kontrollmessungen im Staudamm Finstertal. *Wasserwirtschaft* 74: 172-179.

Schwab, H. & Pircher, W. (1979). Roughening of smooth and steeply sloping rock surfaces in the foundation of Finstertal rockfill dam. *13th ICOLD-Congress, New Delhi,* I: 387- 404.

Seed, H.B. (1973). Stability of earth and rockfill dams during earthquakes, in *Embankment Dam Engineering,* Casagrande Volume, R.C.Hirschfeld & S.J.Poulos (eds), Wiley, 239-269.

Seed, H.B. (1979). Considerations in the earthquake resistant design of earth and rockfill dams. *Géotechnique* 29: 215-263.

Seed, H.B. (1981). Earthquake-resistant design of earth dams. *International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, University of Missouri-Rolla,* 3: 1157-1175.

Seed, H.B. & Idriss, I.M. (1970). Soil moduli and damping factors for dynamic response analyses. Report no. EERC 70-10, University of California.

Seed, H.B. & Idriss, I.M. (1971). Simplified procedure for soil liquefaction potential. *Journal of the Soil Mechanics and Foundation Division* 97, no SM9, ASCE.

Seed, H.B., Idriss, I.M., Makdisi, F. & Banerjee, M. (1975). Representation of irregular stress time histories by equivalent uniform stress series in liquefaction analyses. Report EERC 75-29, University of California.

Seed, H.B., Wong, R.T., Idriss, I.M. & Tokimatsu, K. (1984). Moduli and damping factors for dynamic analyses of cohesionless soils. Report EERC 84-14, University of California.

Sherard, J.L. (1973). Embankment dam cracking, in *Embankment Dam Engineering,* Casagrande Volume, R.C.Hirschfeld & S.J.Poulos (eds), Wiley, 271-353.

Sherard, J.L. (1979). Sinkholes in dams of coarse, broadly graded soils. *13th ICOLD-Congress, New Delhi,* II: 25-35 and in Brauns, J. (1991).

Sherard, J.L. (1986). Hydraulic fracturing in embenkment dams. *Journal of Geotechnical Engineering* 112, no 10, ASCE: 905-927.

Sherard, J.L. & Dunnigan, L.P. (1989). Critical filters for impervious soils. *Journal of Geotechnical Engineering* 115, no 7, ASCE: 927-947.

Sherard, J.L., Dunnigan, L.P. & Decker, R.S. (1976a). Identification and nature of dispersive soils. *Journal of the Geotechnical Engineering Division* 102, no GT4, ASCE: 287-301.

Sherard, J.L., Dunnigan, L.P., Decker, R.S. & Steele, E.F. (1976b). Pinhole test for identifying dispersive soils. *Journal of the Geotechnical Engineering Division* 102, no GT1, ASCE: 69-85.

Sherard, J.L., Dunnigan, L.P. & Talbot, G.R. (1984a). Basic properties of sand and gravel filters. *Journal of Geotechnical Engineering* 110, no 6, ASCE: 684-700.

Sherard, J.L., Dunnigan, L.P. & Talbot, G.R. (1984b). Filters for silts and clays. *Journal of Geotechnical Engineering* 110, no 6, ASCE: 701-718.
Sierra, J.M., Ramírez, C.A. & Hacelas, J.E. (1985). Design features of Salvajina dam, in Cooke, J.B. & Sherard, J.L. (1985) (eds): 266-315.
Strobl, Th. (1982). Ein Beitrag zur Erosionssicherheit von Einphasen-Dichtungswänden. *Wasserwirtschaft* 72: 269-272.
Strobl, Th. (1989). Erfahrungen über die Untergrundabdichtung von Talsperren, dargestellt am Beispiel des Brombachspeichersystems. *Wasserwirtschaft* 79: 372-377.
Sykes, A.W. & Goister, E.A.W. (1979). Grass protection of downstream earthfill slopes. *13th ICOLD-Congress, New Delhi,* II: 47-55.
Takahi, A., Kimoto, E. & Kanwo, S. (1991). Design and construction of an earthfill dam on sandy foundations. *17th ICOLD-Congress, Vienna,* III: 381-400.
Taylor, K.V. (1973). Slope protection on earth and rockfill dams. *11th ICOLD-Congress, Madrid,* III: 215-235,
Terzaghi, K. (1925). *Erdbaumechanik auf bodenphysikalischer Grundlage.* Deuticke, Wien. Unveränderter photomechanischer Offsetdruck bei Gistel, Wien, 1976.
Terzaghi, K. & Peck, R.B. (1948). *Soil Mechanics in Engineering Practice.* Wiley, New York.
Thomas, H.H. (1976). *The Engineering of Large Dams.* Wiley, New York.
Thurner, H.F. (1986). Neue Methoden der Verdichtung und Verdichtungskontrolle. *Symposium Verdichtung von Böden, Korngemischen und Asphalten, Verdichtungstechniken und Prüfmethoden, Wien,* BVFA: 43-46.
Tschernutter, P. (1988). Experience gained with asphaltic concrete facings on high-level embankment dams of the Fragant group of power schemes. *16th ICOLD-Congess, San Francisco,* II: 1105-1131.
Tsuchida (1970). Prediction and countermeasure against the liquefaction in sand deposits (in Japanese), in Ishihara, K. (1985).
Tykal, J. & Schwab, E. (1986). Erfahrungen mit dynamischen Verdichtungsmessungen auf Prüfstrecken. *Symposium Verdichtung von Böden, Korngemischen und Asphalten, Verdichtungstechniken und Prüfmethoden, Wien,* BVFA: 209-247.
USBR (1973). *Design of Small Dams.* 2nd edition, United States Government Printing Office, Washington.
USBR (1974). *Earth Manual.* 2nd edition, United States Government Printing Office, Washington.
USCE (1955). Drainage and erosion control – subsurface drainage facilities for airfields. Part XIII, Engineering Manual, Military Construction, Washington.
USCOLD (1993). Use of granular filters and drains in embankment dams. Report of the Committee on Materials for Embankment Dams, 3rd draft.
Varty, A., Boyle, R.J., Pritchard, E.D. & Gill, R.E. (1985). Construction of concrete face rockfill dams, in Cooke, R.B. & Sherard, J.L. (1985) (eds): 435-458.
Vaughan, P.R. & Soares, H.F. (1982). Design of filters for clay cores of dams. *Journal of Geotechnical Engineering* 108, no GT1, ASCE: 17-31.
Villegas, F. (1982). Difficulties during construction of the Punchina cofferdam. *14th ICOLD- Congress, Rio de Janeiro,* IV: 1081-1101.
Volk,G.M. (1937). Method of determination of the degree of dispersion of the clay fraction of soils. *Proceedings, Soil Science Society of America* 2: 561.

Wallace, B.J. & Hilton, J.I. (1972). Foundation practices for Talbingo dam, Australia. *Journal of the Soil Mechanics and Foundation Division* 98, no. SM10, ASCE: 1081- 1098.

Webster, J.L. (1970). Mica dam designed with special attention to control of cracking. *10th ICOLD-Congress, Montreal,* I: 487-507.

Whiteley, R.J. (1983). Recent developments in the application of geophysics to geotechnical investigations, in In Situ Testing for Geotechnical Investigations, M. Ervin (1983) (ed.), Balkema, Rotterdam.

Wilson, S.D. & Marsal, R.J. (1979). *Current Trends in Design and Construction of Embankment Dams.* ASCE, New York.

Witt, K.J. (1986). Filtrationsverhalten und Bemessung von Erdstoff-Filtern. *Veröffentlichungen des Institutes für Bodenmechanik und Felsmechanik der Universität Fridericiana in Karlsruhe* 104.

Wittke, W. (1990). *Rock Mechanics. Theory and Applications with Case Histories.* Springer, Berlin.

Wittmann, L. (1982). Sicherheitsaspekte bei der Filterbemessung. *Wasserwirtschaft* 72: 147- 151.

Working Group of the Italian National Committee on Large Dams (1988). Embankment dams with impervious upstream facings: An overview of Italian practice. *16th ICOLD- Congress, San Francisco,* II: 415-454.

Xanthakos, P.P. (1979). *Slurry Walls.* McGraw-Hill, London.

Zahaf, A., Claesson, A. & Holmquist, S. (1982). Preparation and compaction of impervious fill on the Bourguiba dam project at Sidi Saad. *14th ICOLD-Congress, Rio de Janeiro,* IV: 4333-452.

Ziems, J. (1968). Beitrag zur Kontakterosion nichtbindiger Erdstoffe, in Wittmann, L. (1982).

Zingg, Th. (1935). Beitrag zur Schotteranalyse. *Schweiz. Min.-Ptr. Mitt.* 15: 39-140.

Subject index