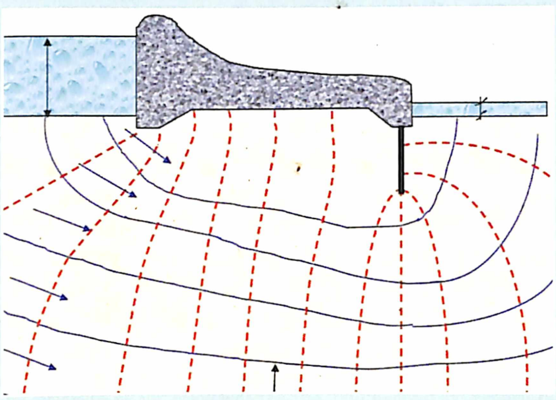

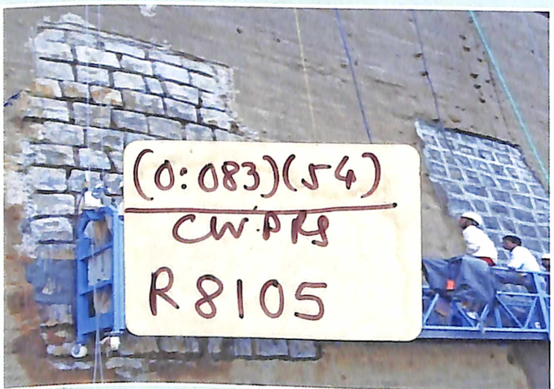

सत्यमेव जयते

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Government of India
जल संसाधन, नदी विकास और गंगा संरक्षण मंत्रालय
Ministry of Water Resources, River Development and Ganga Rejuvenation

Technical Memorandum
on

CONTROLLING SEEPAGE THROUGH HYDRAULIC STRUCTURES

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PREFACE

Excessive seepage in Hydraulic Structures may threaten their safety and stability, in addition to enormous loss of precious water resources. In spite of taking proper care in planning, design and execution stages, there are incidences of distress in dams due to seepage related problems. As such, it becomes essential to diagnose the complex problem of seepage using several possible techniques to arrive at the most suitable controlling measures. Regular monitoring of seepage helps in deciding appropriate and economical remedial measures by taking into account the type of structure, extent and quantity of seepage and feasibility and economy of the control measures.

Central Water & Power Research Station, Pune, is a premier hydraulic research institute offering wide range of R&D services. For the past few decades, CWPRS has developed expertise in providing cost effective and viable solutions for seepage control in hydraulic structures by conducting field and laboratory investigations. Mathematical modeling is being also used for the purpose in some situations. This document provides comprehensive information on various aspects of seepage and its control in Hydraulic Structures with specific reference to dams, canals and reservoirs. Several case studies are also provided for the purpose of illustration.

This document is organized into six chapters, with Chapter I giving a general introduction about causes of seepage, investigations for monitoring and detection of seepage and various remedial measures for seepage in hydraulic structures. Chapter II presents various methods for detection of Seepage in hydraulic structures, important among which are geophysical methods, borehole logging, tracer techniques, permeability test and measurement of pore pressure and uplift.

Seepage in various hydraulic structures and the remedial measures being adopted are discussed in chapters III, IV and V. Chapter III deals with seepage and control measures in Earth and Rockfill dams. Various aspects covered include mechanism of seepage, mathematical and analytical tools for seepage investigations, calculation of seepage forces, seepage monitoring and remedial measures. Chapter IV deals with phenomenon of seepage and its control in Masonry and Concrete dams, covering the effect of seepage on the structure and measures for its control. Chapter V describes the mechanism, monitoring and detection of seepage in canals and reservoirs. Various canal lining methods including application of geosynthetics are discussed. Chapter VI summarizes the important aspects like first detecting the seepage using different methods and then employing suitable remedial *measures to mitigate the seepage, highlighting importance of seepage investigation and remedial measures in hydraulic structures.*

The document is expected to be of great help to practicing engineers, researchers, scientists, consultants and managers of Water Resources projects to mitigate seepage related problems in Hydraulic Structures.

Editor

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CHAPTER I
INTRODUCTION

R.K. Kamble, Scientist-E

1.0 PREAMBLE

Huge amount of water is stored for longer time in Hydraulic Structures. However, significant amount of water is lost in the form of evaporation, seepage and leakages. Seepage is defined as interstitial movement of water through a structure, its foundation or abutments whereas leakage can be defined as flow of water through cavities or cracks. Both seepage and leakage are matters of concern for safety of the structure and pose a serious water management problem. In spite of taking due care in planning, design and execution stages, many of such structures have shown signs of distress due to occurrence of excessive seepage or leakage. Uncontrolled seepage can lead to dire consequences such as complete failure of the hydraulic structure. This can result into severe floods on the downstream leading to loss of life and property. The phenomenon of occurrence of seepage is evident in earth and rockfill dams, masonry and concrete dams, canals and reservoir as well.

In embankment dams, uncontrolled seepage can saturate and weaken portions of the embankment and foundation, making the embankment susceptible to earth slides. If the seepage forces are large enough, soil will be eroded from the foundation causing "sand boils". Seepage flow which is muddy and carrying sediment (soil particles) is evidence of "piping" and is a serious condition. If left untreated, piping can cause failure of the dam. Piping can most often occur along a spillway or other conduit through the embankment. These areas should be closely inspected. Sinkholes may develop on the surface of the embankment as internal erosion takes place. A whirlpool in the lake surface may follow and then a rapid and complete failure of the dam is likely to occur.

Seepage in concrete and masonry dams occurs due to a number of causes such as constructional deficiencies, defects in structural members, effects of environmental changes on concrete / material used for construction, excessive loading on the structure etc.. Mitigation measures for seepage depend upon conditions, quantum and causes of seepage. If seepage exceeds beyond limit, the structure may become structurally weak and may not be suitable for the intended purpose. The incidences are rare in case of concrete dams as compared to masonry dams, where seepage is mainly because of cracking due to thermal effects, alkali aggregate reaction or constructional deficiencies. If the concrete structure does not have measures such as weep holes or relief drains to relieve water pressure, the concrete structure may heave, rotate, or crack. It should be noted that water pressure behind or beneath structures may also be due to infiltration of surface water or spillway discharge, but should still be addressed.

Seepage losses in canals lead to a significant loss of usable water apart from other disadvantages such as water logging of adjacent areas, increased maintenance costs, etc. Seepage through canals can be reduced by lining of the canals. Suitable type of lining such as boulder lining, brick lining, asphalt lining, concrete lining, geomembrane lining, etc can be selected considering aspects such as economy, ease of implementation, availability of the material etc..

Seepage losses through reservoir are important and need to account during planning and design stage of project. Seepage losses are mainly due to unfavorable geological conditions and related to the permeability of surrounding rock. The control of seepage through reservoir is necessary to drain seepage from foundation so that stability and functioning is not affected. Control measures like foundation grouting, providing cutoff trenches and upstream impervious blankets are adopted for reducing / stopping seepage.

The development of seepage through body and subsoil of a dam provides basic information on the state of a hydraulic structure and on the possibilities of its safe operation. Therefore, seepage through or under a hydraulic structure can be considered as one of the most important objects in structural safety. To arrive at an optimum solution, every problem, involving occurrence of seepage or leakage, needs specific attention owing to its uniqueness. Very often costly repair works for addressing seepage problems are undertaken using conventional methods which are deficient in mitigating the problem. Hence, it is imperative to study the complex problem of seepage through hydraulic structures, systematically. Estimation of seepage and evaluation of seepage parameters will serve as inputs to repair or remedial measures applied for reducing seepage through hydraulic structures.

1.1 CAUSES OF SEEPAGE IN HYDRAULIC STRUCTURES

The main causes of seepage and leakage in different types of hydraulic structures in general can be listed as follows:

(1) Aging of structure,

(2) Fracture, fault or shear zones in the pervious or permeable foundation,

(3) Construction deficiencies

(4) Uneven settlement of structure due to faulty design, stress state and dimensions.

However, the causes in particular for different hydraulic structures are complex in nature and site specific.

1.1.1 Earth Dams

The main causes of occurrence of seepage in earth dams are piping / erosion and pore pressure development. Seepage through earthen dams mainly occurs due to lack of filter protection and improper filter design, washing away or particles or clogging of drains, poor compaction, open seams, cracks caused by earth movement, buildup of excess pore pressure, etc. The principal failure modes in earth dams are internal erosion / piping, overtopping, structural issues and slides on either upstream or downstream face. Internal erosion of the foundation or embankment caused by seepage is known as piping. Transition between masonry/concrete dams and earth dams constitutes an area of discontinuity in the material properties and if not taken care of may lead to failure.

1.1.2 Masonry Dams

Seepage path through body mass when exposed to water are permeable and start seeping water by lower pressure inside the body of dam or through heterogeneous, pervious zone where seepage pressure in pervious layer exerts an excessive force on overlying confining layer. Moisture absorption, leaching, excessive uplift pressure, construction defects, construction joints etc. are the main causes of seepage in masonry dams. Seepage through masonry structures is mainly attributed to improper cement mortar ratio, type of cement, poor quality of stones, stiffness in joints, lack of expertise of mason in packing the rubble gaps and low degree of quality control exercised. Due to the technique used for construction, likelihood of seepages in masonry dams is more than that in concrete dams. The construction quality of masonry dam solely depends upon the skillfulness of the mason doing the jointing work of stones. The procedure of construction therefore, is liable to involve numerous human errors affecting quality. The art of placing of mortar in joints and packing joints is most important factor governing quality of joints with respect to seepage in masonry dams.

1.1.3 Concrete Dams

The main causes of seepage in concrete dams are (i) porosity of construction material, (ii) construction joints (provided as part of structural requirement) ,(iii) cracks induced due to various causes, (iv) constructional deficiencies, (v) defects in structural members, (vi) effects of environmental changes on concrete or other material used for construction ,(vii) excessive loading on the structure etc.. Seepage in concrete dams occurs due to improper mix design, inferior quality of construction material, and poor quality control during construction. Development of cracks due to various reasons like shrinkage, thermal loading and other structural problems are also responsible for seepage in concrete dams. Some of the other major causes for seepage in concrete dams include disintegration and scaling, efflorescence, erosion, spalling, popouts, cracks, etc.. The effects of the freezing and thawing can amplify these problems.

1.1.4 Canals

Seepage in canals refers to the water that percolates into the soil strata through wetted perimeter of a canal. Seepage losses affect the operation and maintenance of canals by piping and eroding the bank of canals. Generally earthen canals are mostly constructed using local materials, often with high permeable characteristics. Despite attempts to reduce permeability, construction methods have often failed to achieve a watertight barrier, particularly in older canals. Importing of better quality soils is often limited by availability or cost. Seepage from open canals especially with high embankment is therefore of great concern.

The consequence of seepage losses results not only in depleted freshwater resources but also cause water logging, salinization, and ground-water contamination. Further, seepage losses produce excessive saturation, uplift pressure, which might produce failures of the canal and other structures (Rushton and Redshaw 1979). Even concrete lined canals also have seepage if the lined areas consist of cracks (Merkley 2007).

Seepage loss from unlined canals is governed by factors such as canal geometry, hydraulic conductivity of the sub-soils, hydraulic gradient between the canal , the aquifer beneath and initial boundary conditions. The two most effective solutions to combat canal seepage include either lining of the canals or replacing them with pipes. However, targeted reduction of canal seepage through lining is more cost-effective than piping an entire irrigation system to reduce the evaporation and seepage losses.

1.1.5 Reservoirs

Seepage in reservoirs occurs due to unfavorable geologic conditions such as fractures, faults, open jointed bed rock, solution cavities, sinkholes in carbonate rocks, unconsolidated and /or permeable sedimentary rocks, lava tunnels etc. The magnitude of seepage losses in reservoirs is related to the permeability of the enclosing bed rocks.

Severe reservoir leakages have resulted in reservoir abandonment and even dam failure, in several extreme cases. Filling of reservoirs is followed by water seepage at the foundation and around the flanks of dam. Water seepage from the reservoir through the dam foundation beyond acceptable limits is a serious concern as it creates uplift pressures beneath the dam body. Filtration water losses from reservoir causes swamping in the surrounding areas. The roundabout seepage gives rise to springs on the downstream slope of dam and within its surroundings.

Few of the geologic conditions contributing to seepage are listed below:

- Loose, saturated, non-plastic soil deposits liquefying under earthquake
- Weak and sensitive clay
- Dispersive, organic, expansive, collapsible soils or clay
- Shales, limestone or calcareous deposits with solution channels
- Clay seams and shear zones
- Rock formations with low RQD (< 50%)
- Certain evaporites like gypsum etc.
- Buried palaeochannels
- Unconformities or discontinuities in the rock formation

1.2 METHODS FOR MONITORING AND DETECTION OF SEEPAGE

It is important that an early detection of occurrence of seepage in hydraulic structures is carried out. This can be achieved by regular inspection and monitoring. Monitoring by visual inspection or instrumentation is essential to detect seepage and prevent failure of the structure due to seepage. It is important to keep written records of points of seepage exit, quantity and content of flow, size of wet area, and type of vegetation. Photographs provide invaluable records of seepage. Instrumentation can also be used to monitor seepage. V-notch weirs can be used to measure flow rates easily and inexpensively and piezometers may be used to determine the saturation level (phreatic surface) within the embankment.

Regular observation and maintenance of the internal embankment and foundation drainage outlets is also required. The rate and content of flow from each pipe outlet for toe drains, relief wells, weep holes, and relief drains should be monitored and documented regularly. Normal maintenance consists of removing all obstructions from the pipe to allow for free drainage of water from the pipe. Typical obstructions include debris, gravel, sediment, mineral deposits, calcification of concrete, rodent nests, etc. Water should not be permitted to submerge the pipe outlets for extended periods of time. This will inhibit inspection and maintenance of the drains and may cause them to clog.

Measurements of seepage are indicators of the functioning and safety of a hydraulic structure which can be compared with the permissible seepage values. Inferences on the safe magnitude of seepage for the structure as a whole cannot be worked out based on the permeability values of the constituent materials. As such, in-situ measurements are required to be carried out and permissible seepage values can be derived based on mathematical calculations.

If occurrence of seepage is noticed, measures should be taken to identify the source of leakage. The detection of seepage in hydraulic structures can be done by adopting one or more techniques from the following:

1.2.1 Geophysical Methods

Geophysical methods in general address three objectives namely mapping of geologic features, monitoring of seepage and in-situ determination of engineering properties. Monitoring includes long time observations of intensity of seepage and engineering properties that can be determined in-situ include deformation modulus, electrical resistivity including magnetic and density properties to a lesser extent.

Generally two types of electrical methods a) self-potential and b) electrical resistivity methods are employed for seepage investigation and monitoring. Application of resistivity method to dam seepage investigations is two-fold. The method may be used to monitor spatial and/or temporal variations in electrical resistivity in response to changing soil conditions caused by internal erosion and anomalous seepage. For seepage investigations, resistivity targets generally include fracture zones and solution features created through preferred seepage paths. Resistivity profiling is a primary method used in seepage investigations and was successfully delineated seepage paths in past studies (Butler and Llopis, 1990, Karastathis et al, 2002, Panthulu et al, 2001). In recent times, applications of techniques such as Ground Penetrating Radar (GPR) and Seismic Refraction Method have resulted in fast and accurate analysis of geological parameters. Refraction seismic method is utilized to delineate weak zones in the bedrock and in the foundation of the dam which may be the prospective zones for the seepage. In seismic refraction method, compression wave (P-wave) is generated using a near-surface impulsive energy which propagates through the subsurface media and is refracted along stratigraphic boundaries. The impulsive energy source is selected based upon the length of the seismic line, the resolution desired and the environmental suitability of the seismic source.

1.2.2 Tracer Techniques

Tracer techniques are widely used in different fields as advanced techniques to solve different kinds of problems. In the civil engineering field, tracer techniques are mainly used to solve seepage related problems through hydraulic structures viz. dams, reservoirs and canals. A tracer is a certain substance added to a material in a chemical, biological, or physical system to mark that material for study to observe its progress through the system, or to determine its final distribution.

Tracer techniques are adopted by injecting a pre determined quantity of tracer into a borehole or suspected seepage entry points and monitoring the dilution of tracers at the places of leakage points. The technique provides inter connection between reservoir and seepage points and in turn gives path and source of seepage.

1.2.3 Nuclear Borehole Logging

Borehole geophysical logging investigations represent an economic, non-invasive alternative and can provide in-situ assessment of the engineering properties of the subsurface, potential seepage pathways, lithological variations and solution activity (e.g. Black and Corwin, 1984, Al-Saigh et al., 1994). Application of these methods has demonstrated cost savings through reduced design uncertainty and lower investigation costs.

A borehole log is a continuous record of measurement made in bore hole that responds to variation in some physical properties of rocks through which the bore hole is drilled. The subsurface geologic conditions and engineering characteristics can be determined directly or indirectly from the properties measured by these techniques. Further, borehole logs can be run in cased/uncased and fluid filled boreholes and can be repeated a number of times. The different logging tools are named either on the basis of the parameter measured or according to the principle by which the measurement is made. Different logging techniques such as electrical logging, nuclear logging and acoustic logging techniques can be utilized effectively for detection of seepage in hydraulic structures.

1.2.4 Mathematical Modelling

The flow of water through soil obeys Darcy's law. For a given soil type and for a given boundary conditions of water heads, the movement of water in the soil is governed by Laplace's equation. Solution to this equation, gives the assessment of seepage force, seepage quantity, hydraulic gradient etc. in the flow region. There are many methods to solve this equation e.g. i) Physical seepage models such as electrical method, sand models etc, ii) Mathematical seepage modelling by analytical methods and numerical methods. In the earlier days, before the age of computers, the analytical methods were routinely used for seepage analysis. However, with the advent of computers and software, the numerical modelling has gained popularity because of its ease of usage in multi-layered soil strata and zoned earth structures.

1.2.5 Permeability Measurements

Rock as a material can be permeable to the passage of fluids by virtue of their porosity which is defined as the ratio of the volume of internal open spaces i.e. pores, interstices or voids to the bulk volume of the rock. Permeability is the intrinsic property of the rock material. The foundation rock mass with its system of discontinuities in form of persistent joints, bedding planes, weak seam, shear zones, fault planes etc. including stratification and weathering helps to aggravate seepage or passage of water under hydraulic gradient through foundation.

For any dam foundation, assessment of water tightness of the foundation is of utmost importance since seepage may attribute to the loss of reservoir storage, weakening of foundation by weathering and piping action over a long period of time causing subsidence/settlement as well as adding of considerable uplift pressure at the toe portion of the dam causing instability to the structure. In most of the cases, failure in concrete dams occur more due to foundation failure with or without influence of seepage whereas earth fill dams suffer from seepage and piping to a large extent. Measurement of permeability of foundation rock mass helps to ascertain the nature of flow and quantum of seepage through the foundation rock mass which further acts as guide to undertake various foundation improvement measures for controlling seepage flow through foundation.

1.2.6 Uplift and Pore Pressure Measurements

Effective instrumentation and monitoring combined with regular inspection are the key features of a good dam safety programme. Among other parameters, seepage or leakage through the dam and foundation is one of the major parameters required to be measured in Dam Safety Monitoring (ICOLD, Bulletin 60, 1988). Seepage has both a physical and a chemical influence on concrete and plays a noticeable role on the state of stresses and the stability of the dam. In concrete and masonry gravity dams, seepage occurs at the dam-foundation interface as well as through the body of the dam because of differential pressure gradient from upstream to downstream. Seeping water through the interface creates uplift pressure at the base of the dam is measured by foundation piezometers installed at dam base. Further, depending on the composition and grade of concrete and quality of construction dam body may contain voids, cracks and cavities and the water seeping through these will develop pore pressure in all directions resulting further widening of the seepage path. The pressures developed inside dam body are measured by pore pressure cells embedded in the dam body during construction. Both uplift and pore pressures are destabilizing forces and are required to be measured and monitored. Continuous measurement of these parameters assist, in a way, to assess health of the dam and any anomalous reading can be associated with water seepage.

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CHAPTER II
SEEPAGE DETECTION METHODS

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2.0 INTRODUCTION

Investigations related to dam seepage include study of the proposed site geology using various geophysical and geotechnical methods, assessment of seepage potential of the foundation and analysis of dam instrumentation data. Integration of different methods enables a better understanding of the problem and offer cost effective solutions. Other non-conventional methods like tracer techniques, borehole logging etc. are often used in conjunction with other techniques to have a better understanding of the sub-surface properties. The chapter describes the details of the methods and their potential in assessing seepage through hydraulic structures. Seepage through hydraulic structures are detected and measured by employing different methods such as i) Geophysical ii) Well logging iii) Tracer Technique iv) Permeability tests v) Measurement of Uplift and Pore pressure

2.1 GEOPHYSICAL METHODS

Geophysical methods in general address three objectives namely mapping of geologic features, monitoring of seepage, and in-situ determination of engineering properties. Typical geological features may include faults, bedrock profile, discontinuities, voids and groundwater. Monitoring includes long time observations of intensity of seepage and engineering properties that can be determined in-situ include deformation modulus, electrical resistivity including magnetic and density properties to a lesser extent. Monitoring as well as in situ testing to determine various parameters applying geophysical methods need to be conducted keeping in view of the seasonal variations.

2.1.1 Electrical Methods

Generally two types of electrical methods i.e. a) self-potential and b) electrical resistivity are employed for seepage investigation and monitoring.

Self Potential Method

The self-potential (SP) method is a passive technique used to measure small naturally occurring electrical potentials generated by fluid flow, mineralization, and geothermal gradients within the earth. This is the only one of the geophysical techniques that responds directly to fluid flow (Brosten et al 2005). Water flowing through the pore space of soil generates electrical current flow. This electro kinetic phenomenon is called streaming potential and gives rise to SP signals that are of primary interest in dam seepage studies.

Resistivity Method

The resistivity method is used to measure the electrical resistivity of the geological section in both the lateral and vertical sense. In general, most soil and rock types are considered electrically resistive and the flow of current is influenced by moisture filled pore (or fracture) spaces within the subsurface. Zones of low resistivity values are predominantly controlled by the porosity and permeability of the system, the degree of saturation of the subsurface, and the concentration of dissolved solids within the saturated subsurface. Both "profiling" and "sounding" resistivity techniques are applied.

Profiling provides a means of delineating lateral resistivity contrast within the subsurface electrical properties. Resistivity sounding yields characterization of vertical resistivity contrasts and provides an estimate of the "depth layering" within the subsurface. The purpose of the resistivity survey is to delineate zones of suspected increased permeability and groundwater flow paths as potential sites for borehole drilling and subsequent monitoring well installation. The correlation of known seepage areas with resistivity anomalies is extrapolated to defining areas of suspected increased subsurface flow.

Soundings provide a 1-dimensional (1-D) model of true layer resistivity and thickness beneath the center of the electrode array. In this method, the centre point of the electrode array remains fixed, but the spacing between the electrodes is increased to obtain more information about the deeper sections of the subsurface.

In recent times, applications of techniques such as Ground Penetrating Radar (GPR) and Seismic Refraction Method have resulted in fast and accurate analysis of geological parameters.

2.1.2 Ground-Penetrating Radar (GPR)

GPR, in principle, identify seepage either by detecting underground voids created by the seeping water as it erodes the material or by detecting anomalous change in the properties of the material due to saturation. Advantages include good spatial resolution and high acquisition speed.

The penetration depth depends not only on soil properties but also on the emitted frequency. Different antennas are therefore used for various applications. A long frequency gives a high penetration depth but a low resolution due to the low wavelength. Internal erosion affects the porosity of material in the core and increases the water content. Radar measurements can detect these changes since they influence the radio wave velocity.

2.1.3 Seismic Refraction Method

Refraction seismic method is utilized to delineate the contact between the unconsolidated material and underlying bedrock. In seismic refraction method, compression wave (P-wave) is generated using a near-surface impulsive energy which propagates through the subsurface media and is refracted along stratigraphic boundaries. The impulsive energy source is selected based upon the length of the seismic line, the resolution desired and the environmental suitability of the seismic source. The following case study illustrates the use of the electrical methods in the delineation of seepage paths.

2.1.4 Case Studies on Geophysical Methods

(a) ***Electrical survey for delineation of seepage at Som – Kamla – Amba Dam, Rajastan***

A 1205 m long and 23 m high earth-masonry dam was constructed across river Som near the village Kamla-Amba, Dungarpur District, Rajastan in the year 1993. In order to raise the reservoir level without extending reservoir area, four saddle dams were constructed. On partial impoundment of reservoir, seepage was observed at Saddle dam no. 1 and 3 on their downstream central and right side portions respectively. The project is located on Basal Aravalli formations, consisting of thinly foliated phyllitic quartzites and quartzites with inter layered Quartz-mica-schist bands. Due to folding and faulting the strata shows large variations in strike. A number shear zones most of which are parallel to the foliation, range in thickness from 10 to 30 m traversed the foundation rock. During the foundation investigation stage, weathered, jointed and bedding planes were encountered at Saddle dam No.1. At Saddle dam no.3, highly pervious open joints were encountered. Foundation geological map of main dam reveals that the entire dam is not permeable but some weak zones like cross shear zones and shear joints are susceptible to seepage.

Electrical Resistivity and SP methods were used to delineate the seepage paths (CWPRS Technical Report no. 3479). Parallel profiles were taken on both the saddles on their upstream, downstream and on top. These surveys revealed that in saddle no.3, a zone between Ch. 3170 m and Ch. 3370 m at the upstream toe with depths varying from 14.0 m to 56.3 m is prone for seepage. Two seepage zones were delineated between Ch. 890 m to Ch 1010 m and Ch. 1080 m to Ch. 1130 m with depths varying from 3.0m to 16.5 m at the upstream toe at saddle No. 1 (Fig. 2.1).

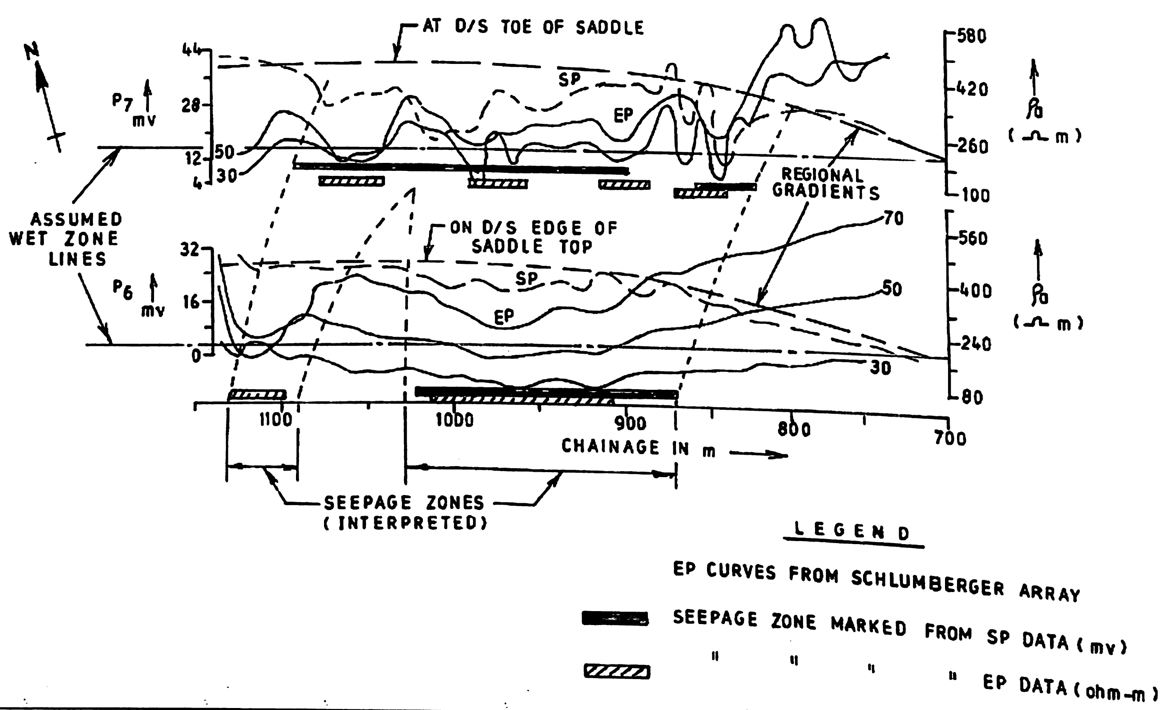


Fig. 2.1: SP and EP Curves at Saddle Dam No. 1 Showing the Seepage Path

It was suggested to keep monitoring the seepage with reservoir levels, by periodical SP measurements for few more wet seasons. If the seepage is found to be alarming, the delineated zones are required to be treated either by curtain grouting or by providing upstream blankets.

In another case study, a combination of GPR and Electrical Resistivity (Conventional and Multi-electrode Imaging) survey was used effectively to map the seepage zones of left bank canal, Tungabhadra Karnataka.

(b) *Ground Penetrating Radar & Electrical Resistivity Imaging to delineate seepage zones at Canal of Tungabhadra Dam, Karnataka*

Tungabhadra is a multipurpose reservoir located at Munirabad about 5 km from Hospet town in Karnataka State regulates water for irrigation through its left and right bank canals to Andhra and Karnataka States. The left branch canal developed breaches near Somnal village from mileages 39 to 41 between years 2000 and 2004. Electrical Resistivity (conventional and multi-electrode imaging system) and Ground Penetrating Radar (GPR) Surveys were conducted at vulnerable high embankment reaches to delineate seepage zones locally known as "Bongas"

(CWPRS Technical Report no. 4552 and 4775). The survey was conducted in two seasons when the canal was full as well as dries.

When the embankment was totally dry, it can be assumed that the bongas or voids are devoid of moisture giving rise to higher resistivty values than the surrounding undisturbed formations. Hence, the higher resistivty values obtained by the spreads of AB/2 =10 and 15m may be attributed to the weak zones susceptible for erosion. Multi-Electrode imaging survey has been conducted at few selected places based on the visual observations of the site when canal was dry. When compared, the resistivity ranges obtained from multi-spacing survey conforms to multi electrode imaging system.

The locations for the second phase of the survey were selected based on the results of first phase and visual observations made at the site. Since the canal was full, the embankment was fully saturated resulting lower resistivity values. From the images it can be seen that the resistivity values in the saturated state reduced considerably. The dark blue colour may indicate the existence of bongas (Fig 2.2).

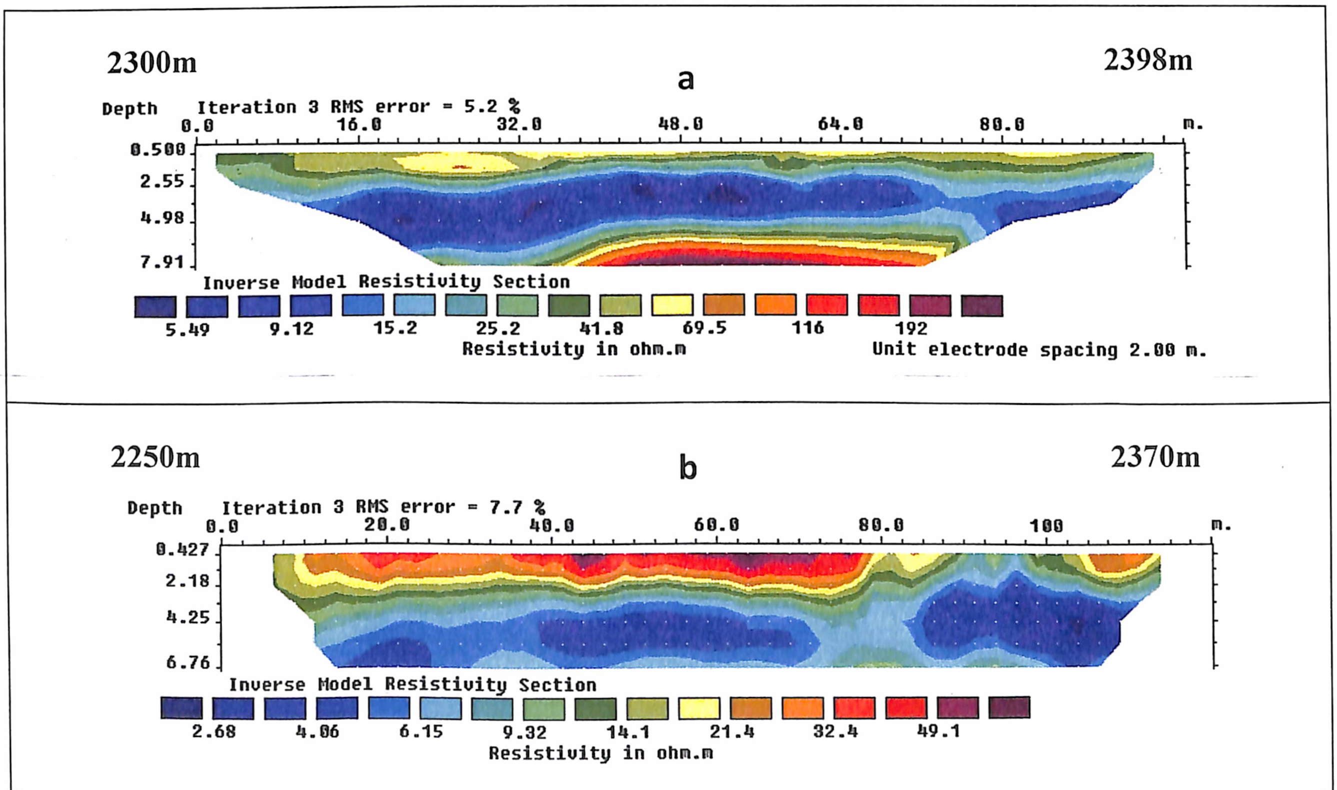


Fig. 2.2: Electric Image of Canal when it is (a) Dry and (b) Full

These locations were verified by the Ground Probing Radar survey. The features observed in the GPR survey may be contributing to the seepage of the canal. Most of the places, the disturbed zones within and below the concrete as observed by GPR (Fig 2.3) match well with the low resistivity zones of Multi-electrode results. It is recommended to inspect the stretches of the canal lining where deterioration/disturbance in the concrete as well as erosion of the material has taken place below the concrete layer and required strengthening works may be undertaken.

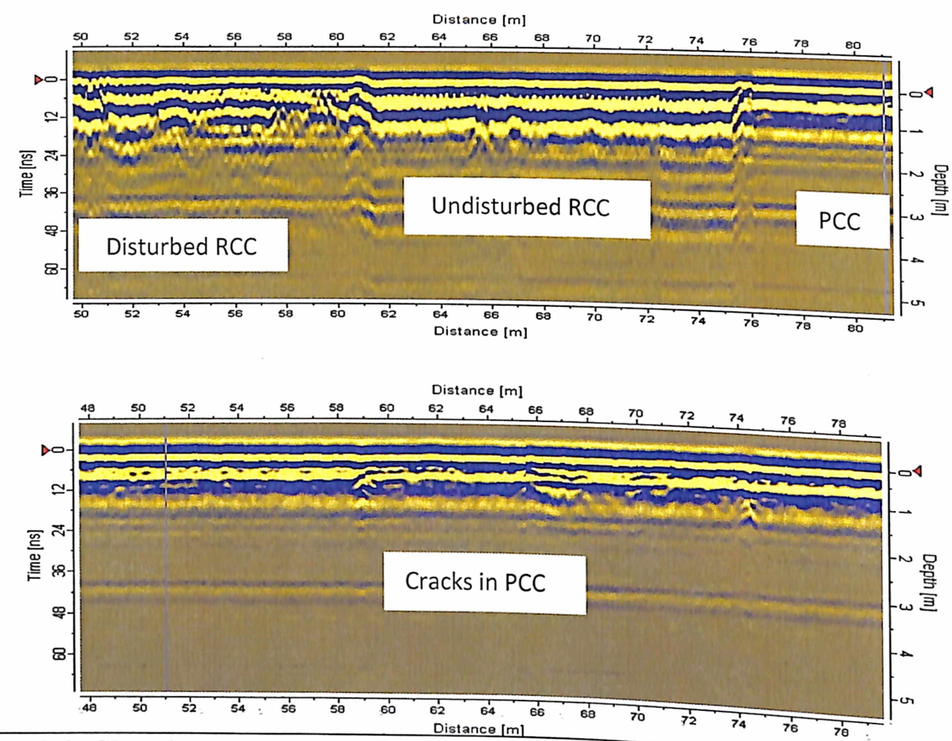


Fig. 2.3: Showing the Typical Images obtained from GPR on Canal Lining

2.2 BOREHOLE LOGGING TECHNIQUES

A borehole log is a continuous record of measurement made in bore hole that responds to variation in some physical properties of rocks through which the bore hole is drilled. The different logging techniques like electrical logging, nuclear logging (gamma-gamma density and neutron) that are most important for detection of seepage in hydraulic structure are discussed.

Electrical resistivity logging is carried out in uncased fluid filled boreholes to determine the electrical resistivity of the rocks, which together with other physical parameters can be used to identify borehole lithology (Maute 1992, Spies 1996). In E-Log, different electrical properties like spontaneous potential, single point resistance, short normal, long normal are measured simultaneously by combining several electrode configurations in the same tool.

2.2.1 Nuclear Logs

Nuclear or radiation logs are related to the measurement of radiations from the nucleus of an atom. The radioactivity measured can be either due to the natural radioisotopes within the formation or from transient response of radioactive sources kept in a probe. These nuclear radiations are in the form of alpha, beta, gamma rays or neutrons. The commonly used nuclear logs are Natural gamma, Gamma-gamma and Neutron. Nuclear logs have a fundamental advantage over most other logs; they may be run in either cased or open holes that are filled with any type of fluid.

Natural Gamma Logs

Natural gamma logs measure the amount of natural gamma radiation that is emitted by all rocks. The gamma ray log is primarily used for identification of lithology and stratigraphic correlation. The natural gamma probe employs thallium activated sodium iodide crystals to detect gamma radiation.

Gamma-gamma (Density) Log

The gamma-gamma or density logs measures the radiation from a gamma source in the probe, after it is attenuated and backscattered in the borehole and surrounding rocks. The main use of gamma logs is for the measurement of bulk density of rocks.

A radioactive source contained in this logging probe emits medium energy gamma rays into the formations which collide with the electrons in the formation. At each collision, they loose some of its energy to the electron and then continues with diminished energy. This type of interaction is known as Compton scattering. Density probe is so designed that the tool response is predominantly due to this phenomenon. Gamma radiation attenuation is assumed to be proportional to bulk density of material it passes through (Keys, 1990). Probes used for density logging contains a concentrated source of mono-energetic gamma rays, a ^{137}Caesium or ^{60}Cobalt and the detector is usually a scintillation counter.

Neutron - Neutron (Porosity) Log

Neutron logs are used principally for delineation of porous formations and determination of their porosity. In neutron logging, neutrons are artificially introduced into the formation and the effect of the environment on the neutrons is measured. The neutron interaction with the subsurface material measures the amount of hydrogen present, which is a direct indication of water content (Keys, 1990). Fast neutrons continuously emitted from a radioactive source such as ^{241}Americium-Beryllium collide with nuclei of the formation material and loose some of its energy and are slowed down by successive collisions to thermal velocities, corresponding to energies of around 0.025 eV.

The neutron log thus measures porosity by determining the amount of hydrogen, hence the amount of fluid filling the pore spaces. When the hydrogen concentration of the zone surrounding the borehole is large, most of the neutrons are slowed down and captured close to the borehole. This results in a low count rate and is interpreted as an indication of high porosity and vice versa. Modern neutron tools most commonly use ^{241}Americium-Beryllium source and count thermal neutrons with a ^{3}He detector.

2.2.2 Caliper Log

The caliper log provides a continuous record of changes in borehole diameter determined by a probe equipped with tensioned mechanical arms or an acoustic transducer. This log is essential in interpreting other logs that are affected by changes in borehole diameter (Keys, 1990). Caliper logs should be run in all boreholes in which other logging is anticipated. They provide indirect information on subsurface lithology and rock quality

One of the major uses of borehole caliper logs is to correct for borehole diameter effects. Multiple-arm calipers (one-, two-, three-, four-, or six-arm) convert the position of feelers or bow springs to electrical signals in the probe.

2.2.3 Borehole Logging Equipment

The well logging unit consists of three parts namely the down hole probe or sonde, cable and winch, and surface system for signal processing and recording. The probes contain sensors to enable specific properties to be measured such as bulk density, porosity/moisture content, formation resistivity, natural gamma radiation, borehole diameter etc. The output electronic signal of the probe either in the analog or digital form is transmitted to the surface instruments via cable and winch. The cable serves the dual purpose of supporting the sonde and conveying power and signals to and from the sonde to the surface unit. The surface unit consists of two sections to provide power and processing the signal from the sonde for recording. The data-recording units are either analog or digital such as laptop PC encoding the signal data from the sonde or surface modules formatting them and storing on magnetic media.

The Robertson Geologging well logging unit (Fig. 2.4), manufactured by M/s Robertson Geologging, UK consists of a winch with a 200 m long multi-core cable, a Micro logger data acquisition system with high-speed data link to connect to a laptop and various probes like Electrical (consisting of Single Point Resistance, 16 inch Short Normal, 64inch Long Normal, Self Potential), Nuclear (consisting of Natural Gamma, Gamma-Gamma Density, Neutron), Caliper, Sonic, Temperature and Conductivity.

Fig. 2.4 Set up for RG Well Logging Equipment

2.2.4 Case Study of Nuclear Logging Study

(a) *Nuclear Logging for Seepage Studies at Indirasagar Project, M.P.*

The Indirasagar Project is a multipurpose river valley project on River Narmada near Punasa village in Khandwa district, of Madhya Pradesh state. The 653 m long and 90 m high solid concrete gravity dam is mildly curved with a radius of 380 m. In addition to irrigation, the project has a powerhouse with an installed capacity of 1000 MW consisting of 8 units of 125 MW each. The dam instrumentation data from the piezometers installed in Block No. 25 at EL 208.85 m indicated high values of uplift pressure. On the basis of interpretation of data acquired from various dam instruments, it was felt that this could possibly be due to entry of seepage water and may pose problem to the structural stability. It was therefore decided to study the likely cause of seepage and to adapt suitable remedial measures for reducing the seepage. Hence, nuclear logging was undertaken to identify permeable zones in the boreholes drilled at selected locations in the body of dam and tracer studies were undertaken to delineate the path of seepage in the abutment, drainage gallery and downstream face of dam (CWPRS Technical Report no. 4679). Geologically, the Upper Vindhyan group of thickly bedded Gr. I, II Quartzite and ferruginous sand stones inner layered with occasional lenses of GR. II/III silt/ clay stone provided the foundation for the 92 m high concrete gravity dam.

TABLE - I

RESULTS OF NUCLEAR DENSITY LOGGING

Borehole Location	R.L.(m)	Drilled Depth of Borehole (m)	Weak Zones observed at RL (m)
Dam-top Block 28	267	60	238 – 241, 212 – 214
Dam-top Block 27	267	30	249.5 – 251
Drainage Gallery-Block 25	235	60	225 – 232, 221 – 22.5, 217 – 218, 213, 210 – 211
Drainage Gallery Block - 26, 48m Depth	245.4	60	221 –230, 198 – 207
Drift Gallery Block- 33, 47m Depth	256	60	235 – 237
Drift Gallery Block - 32, 17m Depth	253	17	250.5

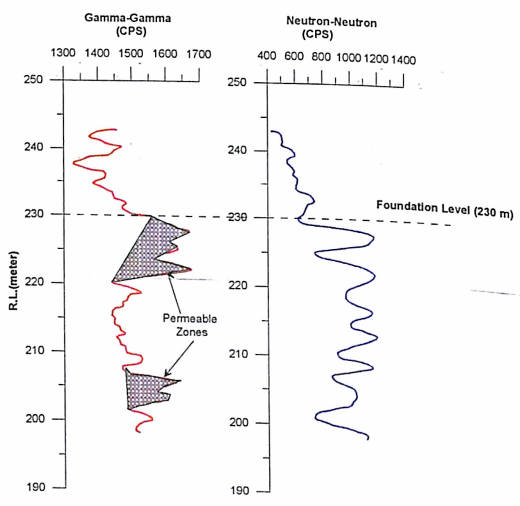


Fig. 2.5 Typical plot of nuclear logging showing permeable zones at Borehole in the body of dam

Nuclear logging comprising gamma-gamma density log, neutron log and caliper log were carried out using portable Well Logging Unit manufactured by M/s. Robertson Geologging Ltd., U.K.in six boreholes drilled in the body of dam at Monolith Nos. 25, 26 and 27, abutment at Monolith No. 28 and in the intake gallery at Monolith Nos. 32 and 33. The results of nuclear logging indicated the presence of weak and permeable zones (R.L. 225 – 232, 221 – 222.5,217 – 218, 213, 210 – 211) in the boreholes drilled in Block No. 25 and the borehole in the abutment at Block No. 28

(R.L. 238 - 241, 212 – 214). They are tabulated in Table – I. Fig. 2.5 shows a typical nuclear log of permeable zones in the borehole. Tracer techniques (Case Study 2.3.4 (a)) were also employed at Indirasagar Project, M.P.

(b) *Nuclear Logging for Detection of Seepage Zones at Pawana Dam*

Pawana Dam, constructed in the year 1972 across River Pawana, a tributary of River Bhima in Pune District, Maharashtra, India, is a 38.1 m high composite dam, comprising 414 m long masonry dam with overflow and non-overflow portion and a 903 m long earthen portion. Excessive seepage was observed in the drainage gallery and body of dam which increased with the rise in reservoir level. Although, guniting of the entire upstream face was undertaken during the period 1975-78 and grouting during 1982-83 noticeable seepage was observed at downstream slope of the non-overflow section and Left Hand Side of the gallery of the dam and the stability of the dam was doubted. It was also proposed to strengthen the dam and raise the Full Reservoir Level by 0.5m, for which in situ bulk densities of masonry of dam were required at different locations. Accordingly, nuclear logging was conducted in 10 numbers of Nx size boreholes drilled on either side of the overflow section at depths varying between 21.2 m to 30.5 m. On the basis of properties determined by nuclear logging(CWPRS Technical Report no. 4673), weak, permeable zones in the vicinity of the boreholes were identified. From the nuclear and caliper logging of boreholes conducted at Pawna Dam, it was observed that, in general, density of masonry varied from 2.3 gm/cm^3 to 2.58 gm/cm^3 and presence of voids/cracks were located in the boreholes. The results are shown in Fig. 2.6. Tracer techniques (Case Study 2.3.4 (b)) were also employed for detecting seepage path.

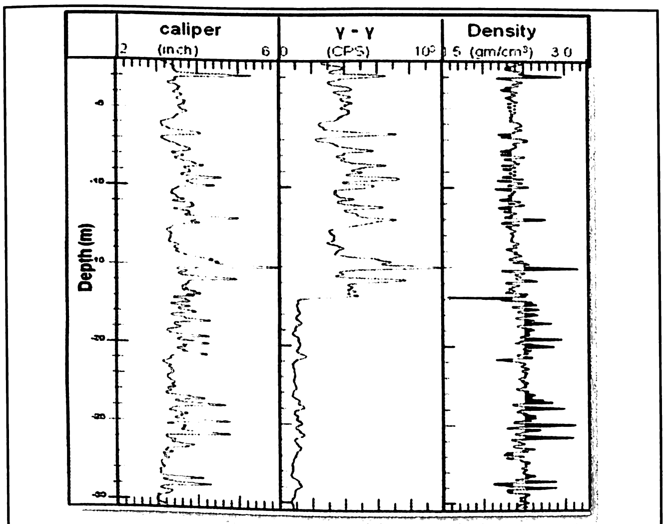


Fig. 2.6 A Typical Caliper & Density Log

(c) Nuclear Logging at *Bhama – Askhed Irrigation Project, Maharashtra*

Bhama - Askhed Irrigation Project envisages the construction of a dam on Bhama river, a right bank tributary of Bhima River in Pune district, Maharashtra with a side spillway, 14km long left bank and 105km long right bank canal. During the monsoon of 2005, 13 RCC panels (7 m x 11 m x 0.30 m) between RD 58 m and RD 91 m in EDA, were lifted up and displaced and the

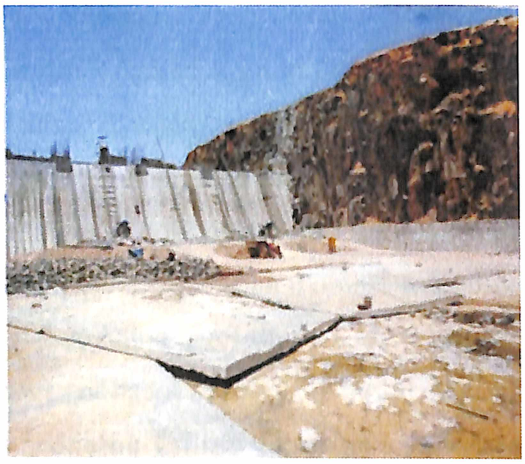


Fig.2.7a. Dislocation of RCC panels in EDA

Fig. 2.7b. Oozing of water at RD 83m in EDA

underlying rock exposed (Fig. 2.7a). Continuous oozing of water was also observed in the chute channel at RD 83 m (Fig. 2.7b). To ascertain the cause of the dislocation of panels and to suggest suitable remedial measures, nuclear logging and tracer studies were conducted in three boreholes drilled up to 6 m below the foundation level located in the area at RD 0 m, RD 40 m downstream on RHS and RD 80 m downstream on the LHS.

Nuclear Density and Caliper logging were conducted to determine the bulk density of formation and also to identify joints/fractures, which were susceptible for leakage (CWPRS Technical Report no. 4540). Nuclear density logging indicated, a weak and permeable zone at depth of 34 m in the borehole corresponding to the red breccia zone in two boreholes located at right hand side of spillway. Tracer techniques (Case Study 2.3.4 (c)) were also employed.

(d) Nuclear Logging at Nagarjunasagar Dam, A.P

Nagarjunasagar dam constructed in the post independence era across the river Krishna in A.P is the largest (4868 m) and highest (1246 m) rubble masonry dam in the world. The masonry dam (1450 m in length) in the centre of the gorge is flanked by earth dams (3418 m in length). The reservoir formed upstream of the dam is the largest man-made lake in the country and third largest in the world. Geologically, the area lies in the sedimentary terrain of Cuddapah group with quartzite as a predominant rock type. Fig. 2.10 shows plan and section showing borehole locations for tracer studies.

In 1989, a settlement was observed at Ch. 142.5 upstream of the right earth dam resulting in the formation of a cavity at RL 182.88 m. To examine the cause of the formation of the cavity and suggest suitable remedial measures, studies were conducted. Nuclear logging comprising gamma-gamma, neutron, caliper and electrical logging was also conducted at Nx-1, Nx-2, Nx-3 and Nx-4 in Nx boreholes to study the lithology (CWPRS Technical Report no. 3153). Tracer techniques (Case Study 2.3.4 (d)) were also employed after Nuclear logging.

2.3 TRACER TECHNIQUES

In the civil engineering field, tracer techniques are mainly used to solve seepage related problems through hydraulic structures viz. dams, reservoirs and canals. A predetermined tracer is injected in water within borehole or reservoir and the tracer dilution is monitored regularly at the place of injection and / or arrival of tracer, if any, is detected at different locations to establish the connectivity.

In seepage studies, Tracers are innocuous (nonhazardous and nontoxic) chemical compounds, salts and dyes that behave exactly similar to the materials to be traced but differ from them by a particular property that may be physical, chemical including radioactive (Moser. H, 1995). An ideal tracer is nontoxic, inexpensive, unique passive-type, easily soluble in cold water, moves with the fluid in contact, easily detectable in trace elements at low concentrations, does not alter the natural flow direction, is chemically stable and sensitive for the desired length of time and for most purposes is neither filtered nor absorbed by the solid medium through which the fluid moves (Flury and Wai, 2003).

2.3.1 Different types of tracers

Different types of tracers are: natural tracers (temperature, conductivity, chemical composition, environmental or stable isotopes of water etc.), environmental isotope tracers (^{16}O, ^{17}O, ^{18}O, ^{1}H (Protium) and ^{2}H (Deuterium) etc.), artificial tracers (NaCl, NH_4Cl and organic dyes like Sodium Fluoroscene, Rhodamine-B, Rhodamine-WT, etc.) and radioactive isotope tracers (Bromine-82, Iodine-131, Cobalt-60, Rubidium, Hydrogen-3 (Tritium) etc.) as well.

2.3.2 Detection of Dam and Foundation Seepage

Tracer techniques are adapted by injecting a predetermined quantity of tracer into a borehole or suspected seepage entry points and monitoring the dilution of tracers at the places of leakage points. Interconnection tests provide proof of hydraulic connection between the point of injection and the measuring point(s). The test consists of the injection of a given tracer amount in the reservoir or boreholes and its subsequent measurement of arrival time and concentrations at boreholes or downstream leaks. In addition to providing the arrival time the interconnection tests allow determination of the passing curve, transit time, amount of tracer recovered, characteristics of the preferential flow path(s), and an estimate of the volume of fissures or cavities along the flow path. The tracer techniques can be employed by utilizing two methods.

i. Single Well or Point Dilution Technique

ii. Multi-well Techniques

Single Well Technique (Point Dilution Technique)

The point dilution technique is designed to determine the Darcy velocity of the formation using single borehole. The aim of the technique is to obtain a direct measurement of filtration velocity i.e. the amount of subsurface water flowing per unit area per unit time in a water bearing formation under natural or induced hydraulic gradient (Halevy et. al. 1967).

The change in concentration of tracer, injected within borehole and monitored at regular interval of time, is caused either by flow or by diffusion (Fig. 2.8). The interconnected fissures/cracks can be located by tracer dilution and filtration velocity can be determined which in turn would give quantity of flow and permeability of masonry / concrete / formations.

This method is widely used to measure filtration velocity, seepage losses as well as permeability using the following equations:

$$\mathbf{V_f = \pi d / 4\phi t \ \ln (C_0 / C)} \qquad (1)$$

Where V_f = filtration velocity

d = borehole diameter

ϕ = hydrodynamic distortion coefficient

and t = time taken for tracer conc. To fall from C_0 to C

The permeability of the strata is computed using the filtration velocity (V_f) and the hydraulic gradient (i) :

$$\mathbf{k = V_f / i} \qquad (2)$$

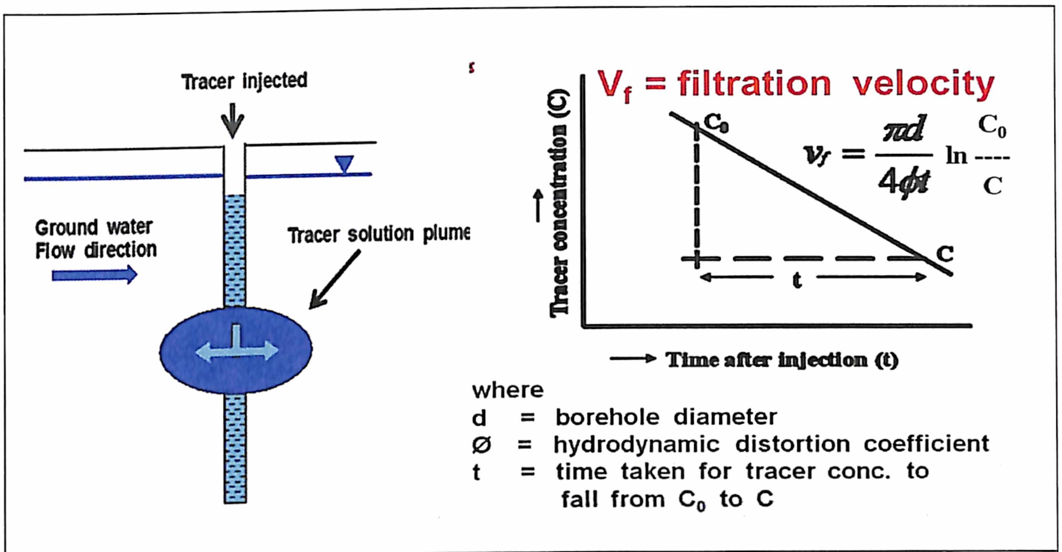


Fig. 2.8 Schematic diagram of Single Well or Point Dilution Technique

Multi-well Technique

The method involves injecting a predetermined quantity of tracer in one of the boreholes and monitoring its appearance in a number of boreholes located at the downstream, in the anticipated direction of flow. The method is used to determine direction of flow and seepage velocity through porous medium. Thus hydraulic interconnection between two water bodies, if any, can be established (Fig. 2.9).

2.3.3 Detection of Canal Seepage

The seepage occurring from canal can be estimated by following methods;

a. Inflow-Outflow method

b. Ponding method

c. Seepage meter method

d. Tracer Technique

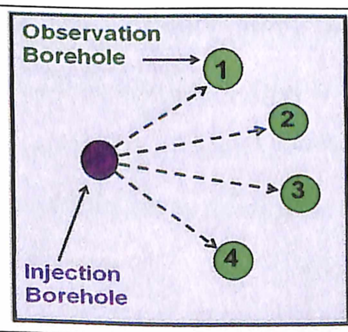


Fig. 2.9 Schematic diagram of Multi Well Technique

Short description of each method is given below:

Inflow-Outflow method

In this method the difference in the quantities of water flowing in and out of the reach is taken as the seepage loss taking into account the evaporation losses. This method is suitable for the main canals and branches where discharges and losses are high.

Ponding method

This method is the most accurate method for determination of the seepage losses from the canals, but it is expensive and time consuming. In this method, temporary water tight dikes or bunds are required to be made in isolated reaches of a canal. Water is impounded between the bunds and with time, rate of drop in water surface is measured. The rate of drop and physical dimensions of the ponded reach provide the data necessary to compute the seepage losses per unit of wetted area per 24 hours.

Seepage meter method

This method consists of a seepage cup connected to a plastic bag by means of a plastic tube and held by a rod. The seepage cup is placed at the bed of the canal, while the plastic bag is held at the water surface. The bag is filled with water and feeds the seepage cup. The amount of seepage loss is derived by weighing the plastic bag at the beginning and end of the experiment. This method has some limitations and generally gives losses more than the actual.

Tracer Technique

In the tracer technique, a tracer is injected in one or more bore holes located near the banks of canal and observing either the dilution of the tracer in the injection bore holes itself or by detecting its arrival in the observation bore holes located near the injection boreholes in the probable direction of seepage.

In canal seepage studies also, Point Dilution and Multiwell techniques are used. The formulae and filtration and seepage velocity equations used in dam seepage investigations are also used in assessment of canal seepage losses.

The seepage losses from the canal (Kaufman et al, 1969) can be calculated using

$$\mathbf{q = 2.V_f\, D.\, \theta.\, Cosec\,\theta / P} \qquad (3)$$

Where V_f = filtration velocity

q = Seepage losses per unit length of the canal,

D = Half bed width of canal,

θ = Angle in radians which the phreatic line makes with the normal at the Injection borehole.

P = fractional porosity

2.3.4 Case Study on Tracer Study

(a) *Tracer Studies for Seepage at Indirasagar Project, M.P.*

The Indirasagar Project is a multipurpose river valley project on River Narmada in Khandwa district, of Madhya Pradesh state. The dam instrumentation data from the piezometers installed in Block No. 25 at EL 208.85 m indicated high values of uplift pressure. On the basis of interpretation of various dam instrumentation data, it was felt that this could possibly be due to entry of seepage water and may pose problem to the structural stability. As such, it was decided to study the likely cause of seepage and to adapt suitable remedial measures for reducing the seepage. Tracer studies were conducted at Indirasagar project, M.P after delineating weak zones by Nuclear logging (Case Study 2.2.4 (a)) (CWPRS Technical Report no. 4679). Sodium fluoroscein tracer was injected at different depths corresponding to the weak and permeable zones in the boreholes viz. (i) in the reservoir at 40 m depth from dam top opposite block 25, (ii) in the reservoir at 25 m depth from dam top opposite block 26, (iii) in the borehole at block 27 from dam top, (iv) in the borehole at block 28 from dam top, (v) in the reservoir at joint of block 32-32, (vi) in the reservoir at joint of block 25-26, and (vii) in the head race. These studies were carried out in two phases, at maximum reservoir level and at minimum available reservoir level. The arrival of the tracer was monitored by taking samples at hourly intervals round the clock from the seepage points in the drainage gallery, in the drilled holes, and the porous block holes (Fig. 2.10). The result of tracer studies revealed that the borehole at Block No.25 was not directly interconnected with the reservoir, rather the path of seepage was from the hillock abutting Block No. 28. The tracer injected in the Head Race Channel near the abutment and its arrival at the seepage points in the adit/intake gallery indicated that the path of seepage could be through the abutment. It may be presumed that at higher reservoir levels, the abutment might be getting recharged and becoming the possible source of seepage at Block No. 25 and in the drift. As the studies indicated that the likely path of seepage could be through the hillock suitable treatment such as silt grouting/ (Bentonite grouting) of abutment is necessary for reducing/stopping the seepage. The providing proper drainage/ discharge of excess seepage in the drift is also suggested as a remedial measure.

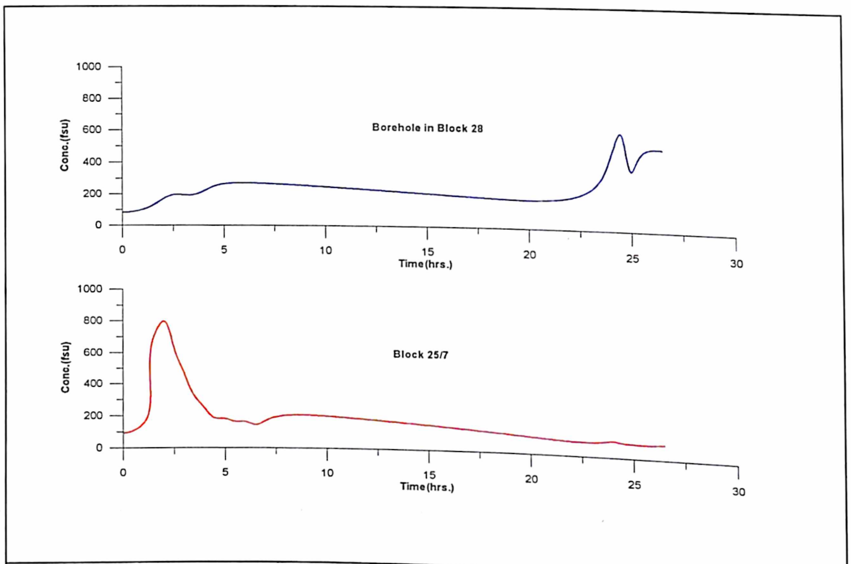


Fig. 2.10 A typical tracer curve at Indirasagar Project, MP

(b) *Tracer Studies for Detection of Seepage Zones at Pawana Dam*

Pawana Dam, was constructed in the year 1972 across River Pawana, a tributary of River Bhima in Pune District, Maharashtra, India. Excessive seepage was observed in the drainage gallery and body of dam which increased with the rise in reservoir level. Although, guniting of the entire upstream face was undertaken during the period 1975-78 and grouting during 1982-83 noticeable seepage was observed at downstream slope of the non-overflow section and Left Hand Side of the gallery of the dam and the stability of the dam was doubted. Tracer studies were conducted at Pawna dam, Maharashtra after delineation of weak zone by Nuclear logging (Case Study 2.2.4 (b)) (CWPRS Technical Report no. 4673). A predetermined quantity of tracer was injected into a borehole at weak zones determined by nuclear logging and the arrival of tracers monitored at leakage points. These studies established direct connection with water jet in LHS drainage gallery from the reservoir at Ch. 735ft. (224.0 m), RL 1939ft (591.0 m). It further revealed that dye injected in the boreholes, where prominent cracks/voids were detected by nuclear logging, arrived at leakage points at downstream slope of the dam. To reduce/stop seepage, it was therefore recommended to undertake controlled grouting in the body of dam where prominent cracks/voids were located, probably due to leaching.

(c) *Tracer Studies for Delineating Path of Seepage to the Damaged Portion of Tail Channel at Bhama – Askhed Irrigation Project, Maharashtra*

Bhama - Askhed Irrigation Project envisages the construction of a dam on Bhama river, a right bank tributary of Bhima River in Pune district, Maharashtra. During the monsoon of 2005, 13

RCC panels (7 m x 11 m x 0.30 m) between RD 58 m and RD 91 m in EDA, were lifted up and displaced and the underlying rock exposed Continuous oozing of water was also observed in the chute channel at RD 83 m. To ascertain the cause of the dislocation of panels and to suggest suitable remedial measures, tracer studies were conducted in three boreholes drilled up to 6m below the foundation level located in the area at RD 0 m, RD 40 m downstream on RHS and RD 80 m downstream on the LHS.

Tracer studies were conducted at Bhama – Askhed Irrigation Project, Maharashtra after delineation of weak zones by Nuclear Logging studies (Case Study 2.2.4 (c)) to confirm or rule out the possibility of interconnection with oozing of water at tail channel from the reservoir through spillway or foundation (Fig. 2.11) (CWPRS Technical Report no. 4540). Tracer techniques involve injecting a predetermined quantity of Sodium fluoroscein tracer into a borehole and monitor the arrival of tracers at leakage points. Tracer was injected in the boreholes at RD 0 m at the foundation level (34 m) RL 639.4 and RD 80 m downstream on the LHS at 16 m (RL 650.4) depths and also opposite to the spillway at different grid points. The samples were collected at the adjacent borehole (RD 40 m downstream on RHS) and at RD 83 m in tail channel where oozing of water was observed.

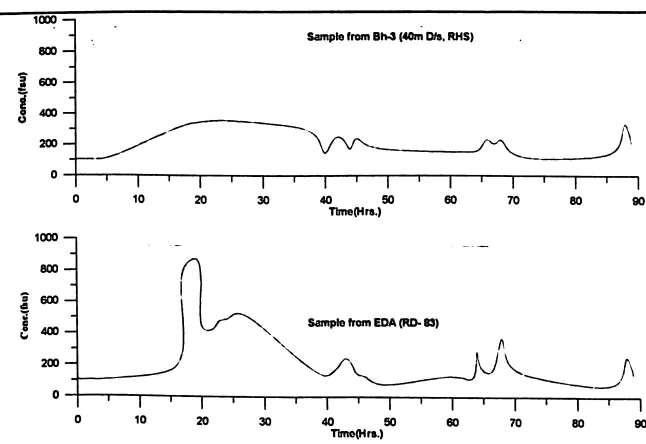


Fig. 2.11 A typical tracer curve at Bhama – Askhed Irrigation Project, Maharashtra

The tracer studies established interconnection between oozing of water in the EDA, and the reservoir through the permeable zone formed by weak red breccia in the foundation of the dam. Further, it was also established that there was no contribution of groundwater from left hand side of the spillway to the oozing of water at RD 83m in the tail channel. From the result of both Nuclear logging and Tracer studies it was concluded that the suitable remedial measures of proper treatment to red breccia encountered in the foundation were suggested to stop oozing of water in the EDA. Adequate drainage holes in EDA were also recommended to release the uplift pressure.

(d) *Tracer Studies at Nagarjunasagar Dam, A.P*

Tracer studies were conducted at Nagarjunasagar dam, A.P after delineation of weak and permeable zones by Nuclear Logging studies (Case Study 2.2.4 (d)).

Accordingly, tracer studies were conducted by injecting the dye in the cavity itself and monitoring in the boreholes drilled at the toe (Fig.2.12) (CWPRS Technical Report no. 3153). However, these studies were discontinued as they did not yield any appreciable results and the dye injected in the cavity did not show any appreciable dilution. Subsequently, CWPRS suggested drilling of three boreholes in the upstream portion near the cavity up to a depth of 10 m into the foundation to facilitate injection of the tracer and monitor its arrival in the downstream boreholes.

Three Nx size boreholes viz. Nx-1, Nx-2 and Nx-3 were drilled on the upstream side of the cavity area to facilitate injection of tracer. For monitoring the arrival of the tracers, eleven shallow boreholes (B1to B11), three deep boreholes (B-I, B-II and B-III) and one Nx-1 borehole used for logging were used for sample collection at the toe of the right earth dam. In order to ascertain the direction of seepage towards the canal located 350 m downstream of the right earth dam , another borehole, Nx-5 was drilled to monitor the flow of the injected tracer. B12 and B13 were drilled at

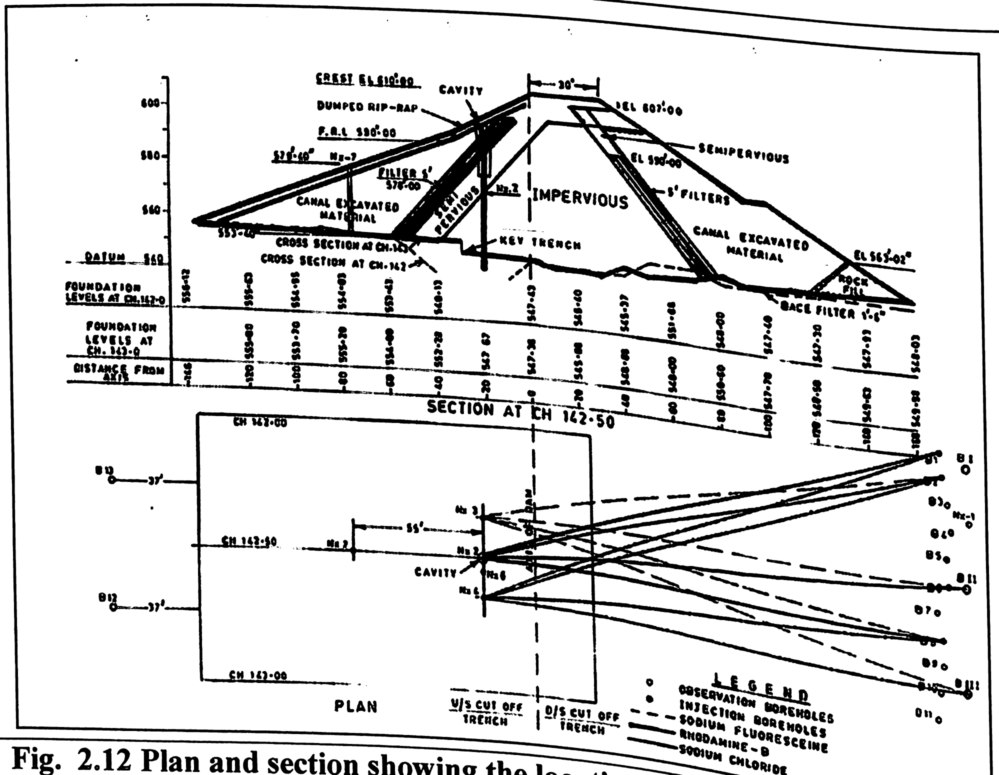


Fig. 2.12 Plan and section showing the location of boreholes for tracer studies at right earth dam, Nagarjunasagar project, A.P.

The tracer studies revealed that there was a hydraulic interconnection between the foundation rock and the toe viz. boreholes (a) Nx-2 and B1, B2, B8, B-II, (b) Nx-3 and B2, B8, B-II, B-III and (c) Nx-4 and B1, B2, B8, B-III. There was no interconnection between cavity and B12 and B13. The permeability of the foundation rock varying from 4.93×10^{-4} cm/sec to 7.75×10^{-4} cm/sec indicated that the rock permeability for rock in the cavity was higher than that for boreholes on either side of the cavity. High seepage velocity ranging between 3m/day to 7.8 m/day has also been observed. Borehole logging also revealed that the foundation rock below cavity (RL 544 – 539ft.) was prone to the excessive seepage. So it was recommended that a suitable treatment should be done for the foundation at cavities zone to reduce the seepage through them.

(e) *Tracer Studies for Leakage in the Approach Tunnel of Underground Power House at Ghatghar H.E.Project*

Ghatghar Hydroelectric (HE) Project, near Chonde village, Thane, comprises of two reservoirs; one at an elevation of 757.50 m MSL (Full Reservoir Level, FRL), formed due to an upper dam at Ghatghar and the other at an elevation of 348 m MSL (FRL), formed due to a lower dam at Chonde village (Fig.2.13).

During 2005 monsoon, due to a heavy natural landslide, floodwater and debris entered the underground powerhouse. All erected equipments were submerged in the powerhouse. During 2006 monsoon, when storage reached the FRL, heavy seepage was noticed at a location inside the Approach Tunnel (AT), and profuse leakage at many places along the length of approach tunnel. There was an apprehension that the leakage water could originate from the lower reservoir, which in turn would lead to head loss for generation of electricity.

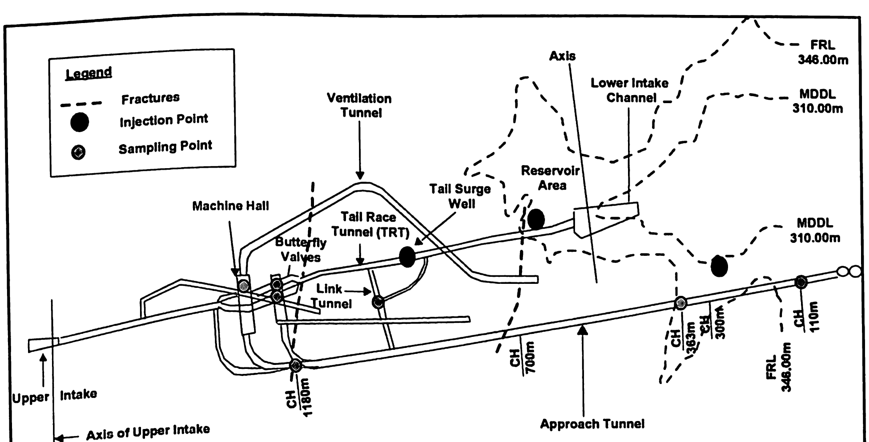


Fig.2.13 Layout of Ghatghar Hydroelectric Project

Tracer studies were undertaken for establishing interconnection, if any, between (i) Tail Race Tunnel (TRT) and seepage in AT, (ii) reservoir through fracture at 700 RD (Ch 1700 of TRT) and seepage in AT, (iii) reservoir at RD 300 (around Ch.300) and seepage in AT by injecting sodium fluoroscein in TRT and reservoir, and monitoring its arrival at leakage points (CWPRS Technical Report no. 4748).

Leakage was observed in the underground power house and suspected source of leakage, i.e. reservoir was about 1 Km away from the leakage point. As such, for conducting the studies, the approach was modified to suit the present case, and sodium fluoroscein dye was injected in the surge well / shaft at a depth of 120 m from the top and round the clock sampling undertaken in the approach tunnel at different locations such as Ch 1180, link tunnels, machine room and butterfly valves at hourly intervals (Fig.2.14).

Tracer studies revealed that leakage emerging from Ch 1180 in the power house was from the reservoir, and its path was through TRT. The interconnection between reservoir and seepage in the approach tunnel could not be established. It was recommended to divert the flow at Ch. 1180 away from the power house and carry out suitable repair work of TRT during lean period of power generation.

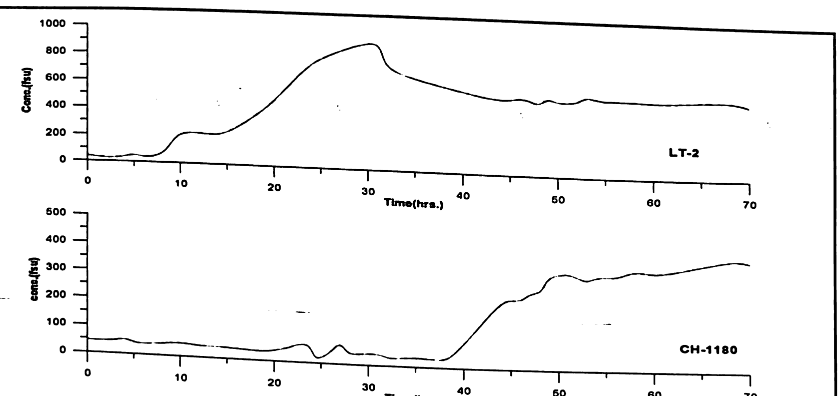


Fig. 2.14 A Typical Tracer Dilution Curve

(f) *Assessment of Seepage losses from Indira Gandhi Main Canal, Rajasthan, (IGMC)*

Indira Gandhi Nahar Pariyojana (IGNP) is the largest irrigation and drinking water project in India to cater to five districts of North-Western Rajasthan with a command area of 15.4 lakh hectares. It comprises a main canal (195 km), nine branches, three lift irrigation and 21 distributaries. The main canal that passes through the arid North Rajasthan region gets water from the River Sutlej in Punjab through a feeder canal which takes from Harike barrage constructed downstream of the confluence of Rivers Beas and Sutlej. Indira Gandhi Main Canal (IGMC) is lined with clay tiles, passes through arid zone and has rolling topography with shifting dunes and interdunal flats.

Water logging has been observed on both banks of IGMC between RD 0 and 100 covering a length of about 30 km. In addition, seepage losses through the canal also add to the water logged area. Tracer studies were therefore conducted to assess the seepage losses from the canal (CWPRS Technical Report no. 3686). The location selected for estimation of seepage losses from canal and for measurement of subsurface seepage at the water logged area are shown in Fig.2.15.

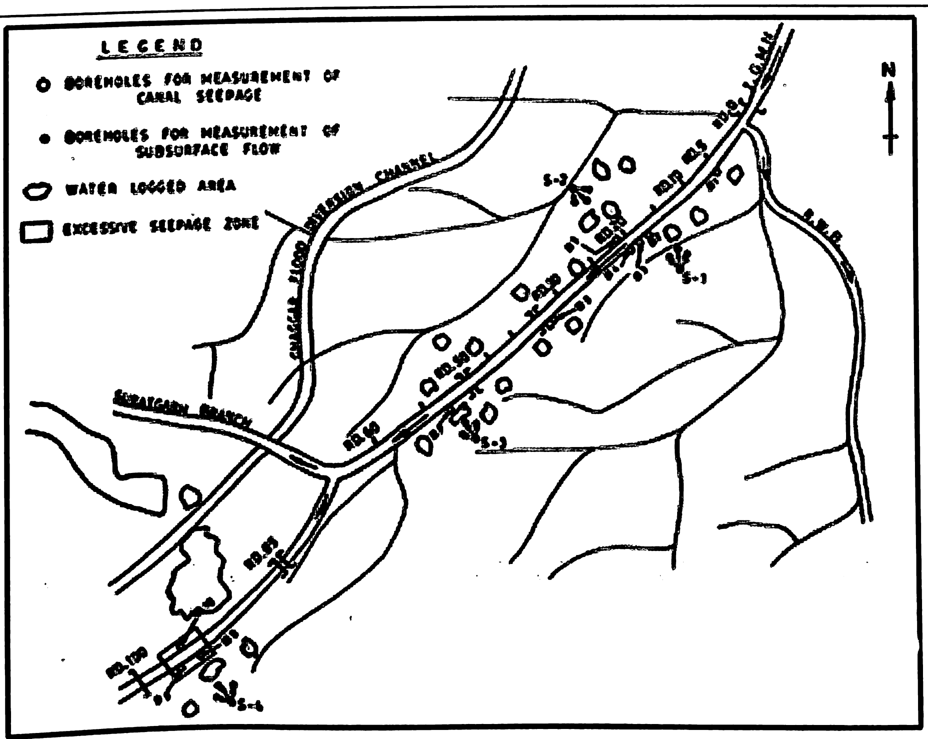


Fig 2.15 Plan of Indira Gandhi Main Canal, Rajasthan, showing locations of boreholes for canal seepage studies.

2.4 FOUNDATION PERMEABILITY

The permeability of dam foundation refers to the overall permeability characteristics of foundation rock mass which is in true sense heterogeneous and anisotropic. Almost all rock foundations are not watertight and some are highly pervious. The foundation rock mass with its system of discontinuities in form of persistent joints, bedding planes, weak seam, shear zones, fault planes etc. including stratification and weathering of rock strata, helps to aggravate seepage or passage of water under hydraulic gradient through foundation. Measurement of permeability of foundation rock mass helps to ascertain the nature of flow and quantum of seepage through the foundation rock mass which further acts as guide to undertake various foundation improvement measures for controlling seepage flow through foundation. Although permeability value for rock specimens of different porosities can be determined in laboratory, permeability of rock mass is governed exclusively by the geological discontinuities Thus laboratory permeability value of rock material do not reflect the overall value for rock mass after accounting for the losses into system of discontinuities present in rock mass and accordingly, in-situ values are much higher than the laboratory values.

2.4.1 Field Permeability Test

Water percolation tests or pumping in test was first introduced by French Engineer Maurice Lugeon in 1933 using single borehole to determine groutability of rock. At present the test is conducted using single or double packers in bedrock as per Indian Standard Code of Practice BIS: 5529- Part II (1973, 1985). The test is conducted either in a completed borehole or as the hole advances during drilling and can be carried out by using either one or two packers. The test is a constant head type test, which take place in an isolated uncased and ungrouted section of the bedrock in bore hole separated by packer/packers. The length of the test section usually varies between 1.5m to 3m. Water is pumped under constant pressure into the test section for determining the permeability of bedrock. This test can be made both above and below the water table provided, the hole through the rock formation stands intact and in no case the applied pressure shall exceed the overburden pressure above the length of the test section. The test is conducted in five stages, with a particular water pressure magnitude associated with each stage. During execution of test at each stage, both water pressure and flow rate values are recorded for a particular time interval. The tests are also of significance in interpreting the drilling data and in supplementing the information obtained from visual examination of the cores. The value of coefficient of permeability obtained from such test, which is the overall value for rock mass including loss into cracks, fissures, joints etc. is often much higher and is fairly acceptable to provide an approximate estimate for seepage loss after reservoir impoundment. In certain formations, it may not be possible to use the packer or there is a danger of the packer being stuck in the hole. In such cases, a better method will be to grout the earlier stage, extend the bore hole and carry out the test. The studies conducted by single or double packer method are detailed below and layout of equipment for permeability test in a drill hole using single packer is shown in Fig.2.16 whereas the arrangements of positioning packers in test hole using single as well as double packers is shown vide Fig.2.17.

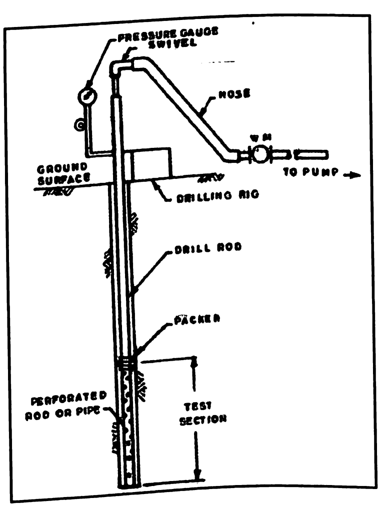
a. *Single Packer Method*

The method is best suited for tests during drilling, on completion of each drill run and is essential when the rock mass is weak or intensely jointed and the hole is likely to collapse if it is kept uncased/ungrouted. In this test, the hole is first drilled to a particular depth and after removal of core barrel, the hole is thoroughly cleaned with fresh water until clear water returns. The packer is then fixed at the desired level above the bottom of the hole and the test is performed.

After completion of test, the entire assembly is removed and the drilling operation is proceeded till the next section has been drilled for performing the next test. In this manner, entire depth of the borehole is tested side by side with the drilling.

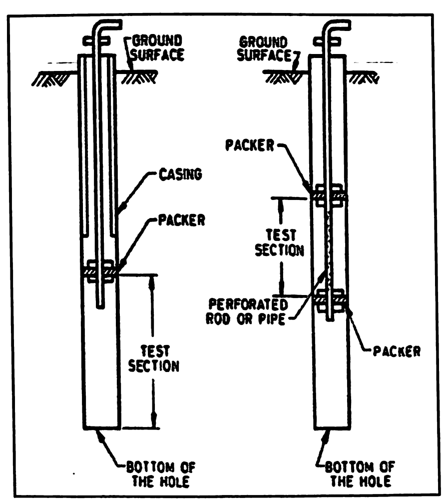
b. *Double Packer Method*

In cases, where rocks are sound and the entire length of the hole can stand without casing/grouting, double packer method is adopted. The specific advantage of double packer method is that, critical rock zones can be tested by confining them alone with packers and the disadvantage is that leakage through the lower packer can go unnoticed and lead to overestimation of water loss. In double packer method, the hole is first drilled up to the final depth desired and is thoroughly cleaned with fresh water until clear water returns. Two packers connected to the ends of a perforated drill rod of a length equivalent to the test section is then fixed in the drill hole. The bottom of the perforated rod needs to be plugged before the tests are proceeded with. The test may be conducted from bottom upwards or from top downwards. However it is usually convenient to start the test from the bottom of the hole and then gradually working upwards. A typical field permeability test setup for double packer test is shown in Fig.2.18.



Single Packer

Fig.2.16 Layout of equipment for Permeability Test in drill hole (Single Packer)



Double Packer

Fig.2.17 Field Permeability test using Single or Double Packers

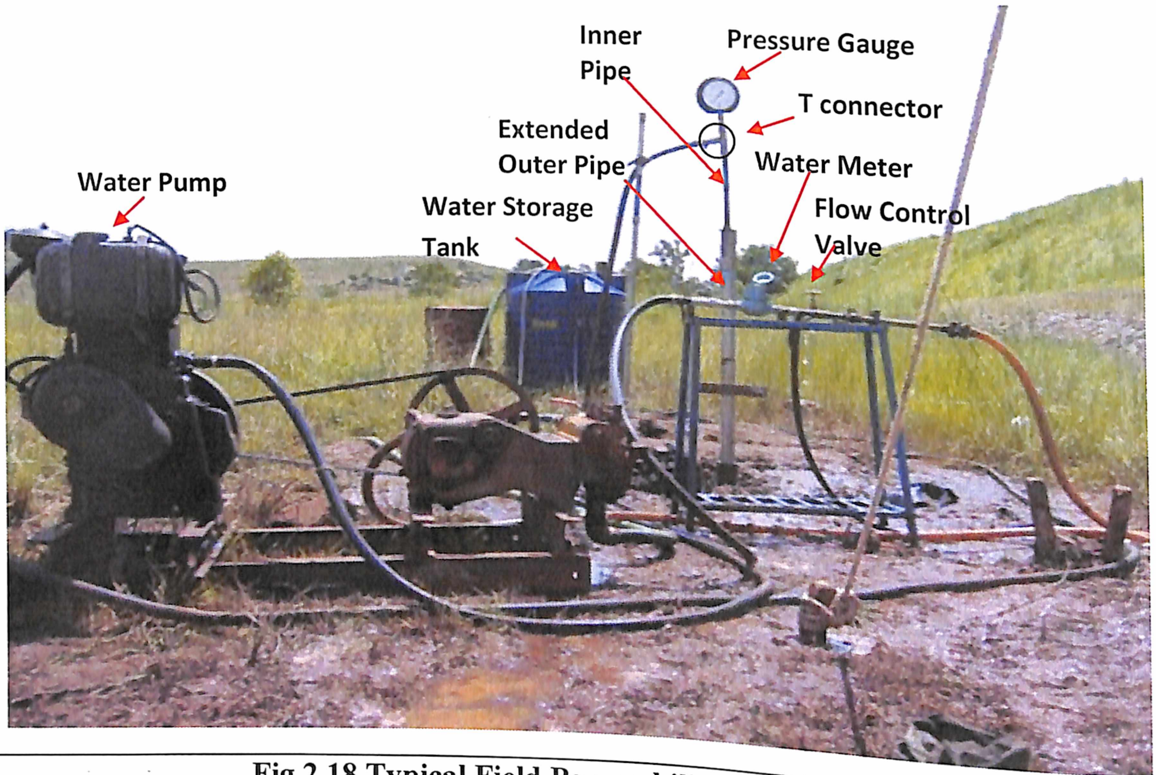


Fig.2.18 Typical Field Permeability Test Setup

2.4.2 Test Procedure

For single/double packer test in bore holes, it has to be ensured that the borehole under test has been drilled using diamond core rotary drilling and properly capped immediately after drilling to protect the same from entry of dirt, muck or other objects. Before commencing the packer test, the groundwater level (G.W.L.) in the borehole needs to be accurately measured since it is required to determine the hydrostatic pressure in the test zone. Packer tests are recommended to 1.5 m to 3 m sections so that the entire length of the hole is covered depending upon local geological setup. The test length should not be less than 5 times the diameter of the bore hole. The water swivel in the test assembly shall have a small uniform inside diameter to minimize loss of head and the pressure gauge is to be located between the swivel and the packer. Unnecessary bends in the pipe line from the pump to the swivel needs to be avoided and all joints and connections between the water meter and the packer should be watertight so that no water loss occur between the water meter and the test section. Water is then pumped into the section under pressure and each pressure is maintained till the readings of water intake at that particular pressure for a fixed time interval of 5 to 10 minutes remain constant. Applied pressure for a particular section depends mainly on the amount of rock cover available for that section to prevent any upheaval and based on rock types of the bed rock strata(Fig.2.19) and is termed as H(pressure) or H_P. Suggested pressures based on type of rock strata as per IS:5529- Part II are shown vide Table-2.

Table-2

Rock Type	Limit of test pressures in kg/cm^2 per metre of rock load (**P** $_{\textbf{suggested}}$)
Unconsolidated or poorly consolidated sedimentary formations	**0.12** kg/cm^2
Consolidated horizontally bedded sedimentary formations	**0.18** kg/cm^2
Hard igneous and metamorphic rocks	**0.24** kg/cm^2

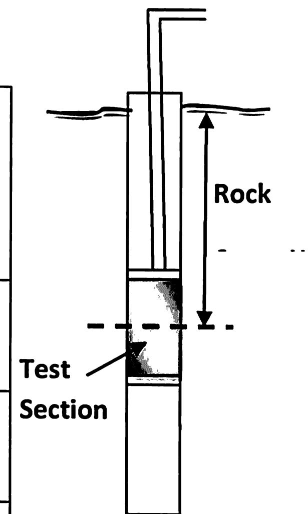


Fig.2.19 Estimation of Applied pressure

Now, Applied pressure $H_p = H \times P_{suggested}$

Water column pressure due to gravity H(gravity) or H_g is determined separately for single as well as double packer as

1) For test section below G.W.L., H_g = Height of swivel from G.W.L and
2) For test section above G.W.L., H_g =Height of swivel from middle of test section

The value of H(gravity) or H_g to be taken for single and double packer test for different ground water level condition is detailed in Fig.2.20(Lama R.D. and Vutukuri V.S., 1978). Maximum allowable pressure is then computed as $P = H_P + H_g$. Mostly, cyclic tests are performed to evaluate the permeability since these are useful in interpreting test results and computing lugeon values. Based on the computed maximum allowable pressure P for a test section, actual pressures are applied in sequence such as P/3 or (1st Stage, Low Pressure), 2P/3 (2nd Stage Medium Pressure),P(3rd Stage, Maximum Pressure),2P/3 (4th Stage , Medium Pressure) and finally P/3 (5th Stage, Low Pressure) and each for time interval of 5 to10 minutes. Thus starting at the lowest pressure, the maximum applicable pressure is built by increments and decreased in the same order till the original pressure is reached and the amount of water intake is recorded. During the test, the packers should not be leaking since, this may cause rise of water level in the borehole or even the water may start flowing out from the nipple.

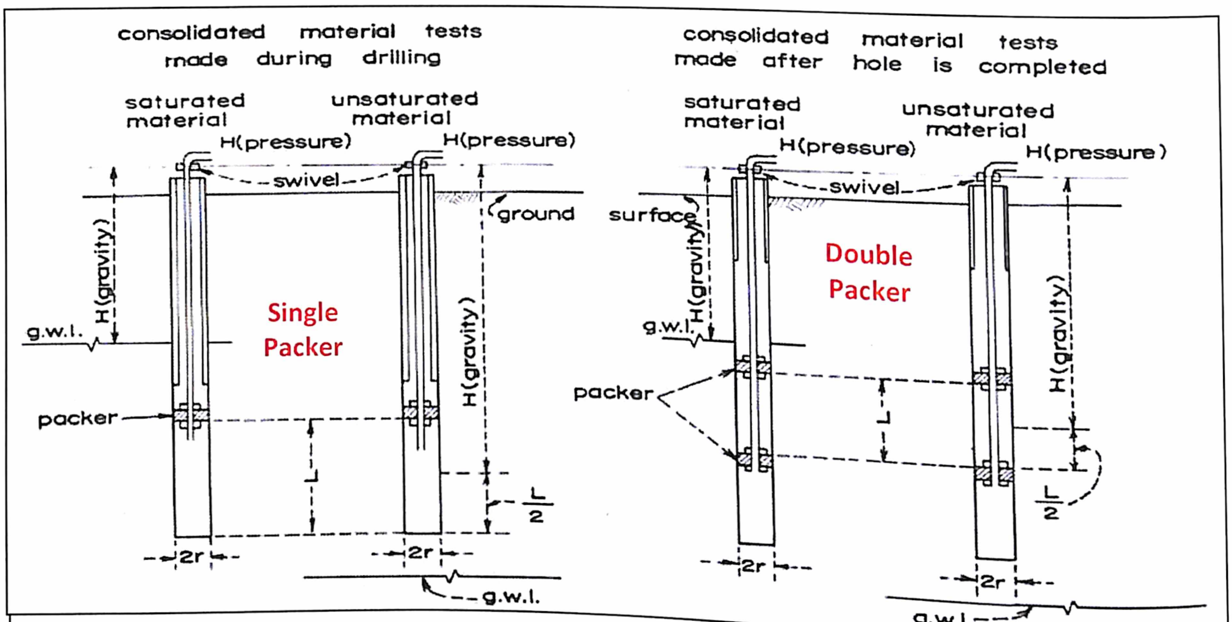


Fig.2.20 Water Column Pressure estimation for Test Section above and below G.W.L.

Lugeon Unit: General geotechnical practice is to define permeability as a type of velocity unit, measuring the rate of flow of water under a standard pressure and total seepage is then calculated by multiplying this by the cross sectional area. However this type of permeability unit is generally not as useful in grouting work as compared to one that indicates the flow radially from the test hole. A test of this radial type, which is used extensively in grouting, is the *pumping in* test where the volume of water taken in a section of test hole is measured during time intervals and the coefficient of permeability(k) determined from the test is commonly expressed in Lugeon units(after Maurice Lugeon,1933) .

A rock mass is considered to accept grout if it has permeability of 1 lugeon.

1 lugeon = flow of 1 litre of water per minute through a borehole of 1 metre length at a pressure of or 10 kg/cm^2 or 10 bars or 150 psi or 0.98 MPa.

In terms of velocity type permeability units, very approximate relationships are

1 lugeon unit = 10 ft/year = 1.3×10^{-5} cm/sec

However, the pressure of 10 bars or 1 MPa suggested by Lugeon has been based on the tests conducted by him at Swiss Alps and is usually too high for routine grouting work and accordingly much lower pressures are therefore used. A correction has therefore been applied to relate the lower pressures to that in the definition and the coefficient of permeability **k** is determined from the following equation.

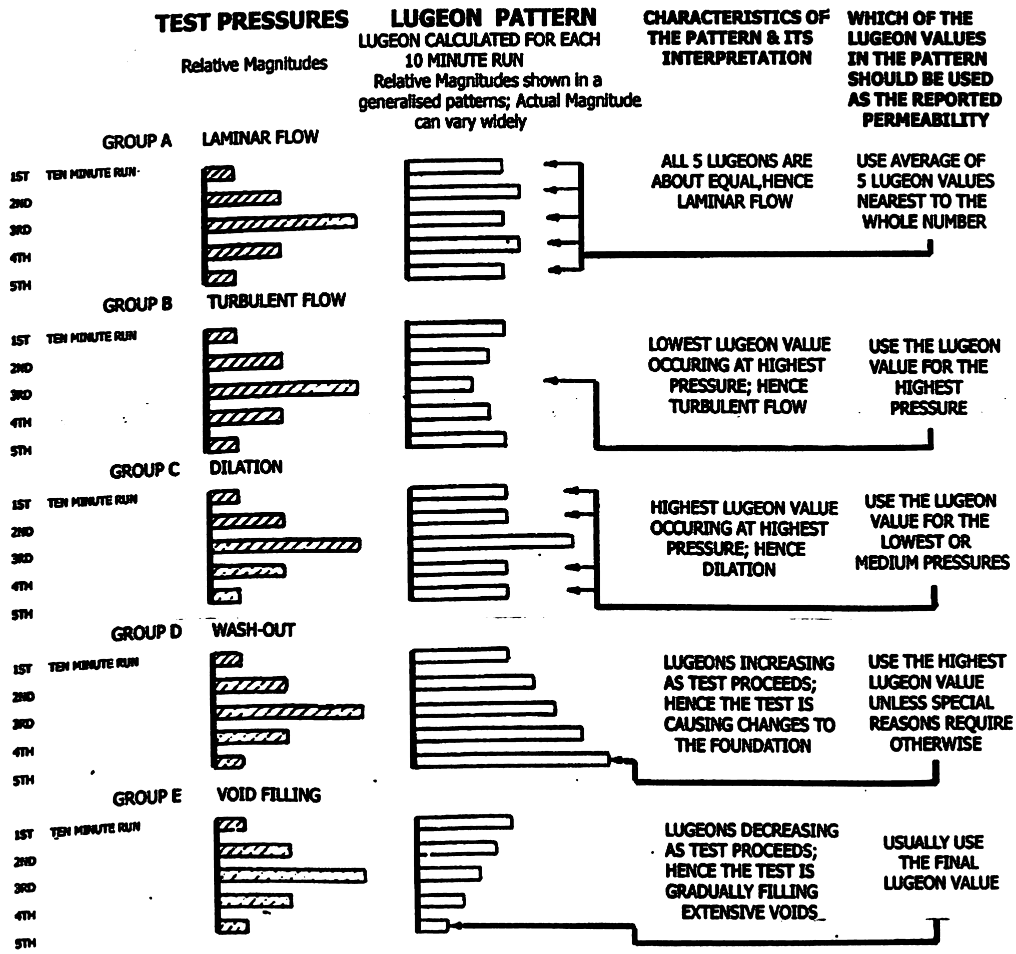
$$k = \frac{\text{Water taken in test(litre/meter/ minute)} \times 1.0 \text{ MPa}}{\text{Test Pressure (Mpa)}} = \frac{Q \times 10}{L \times t \times P} \text{ (lugeons)}$$

where k = coefficient of permeability(lugeons),Q= water intake / discharge (litre),

L = length of test section (metre), t = time interval (minutes)

P = Test pressure (kg/cm^2) = H_P (Applied Pressure)+ H_g (Water Column Pressure)

Summary of interpretation of Lugeon values from field permeability test data depicting various flow conditions is shown below.



One more condition in addition to those listed above can be recognized where initiation of flow occurs as peak pressure is approached but no flow and hence a zero lugeon value at low and moderate pressures. This situation is termed as Hydrofracture and it requires halting the test to avoid further damage once hydrofracture is recognized. A guideline describing condition of rock mass discontinuities associated with different Lugeon values as well as the typical precision used to report these values is shown vide Table-3(Camilo Quinones-Rozo, P.E (2010)).

Table-3

Lugeon Range	Classification	Hydraulic Conductivity Range (cm/sec)	Condition of Rock Mass Discontinuities	Reporting Precision (Lugeon)
<1	Very Low	$< 1 \times 10^{-5}$	Very tight	<1
1-5	Low	1×10^{-5} - 6×10^{-5}	Tight	± 0
5-15	Moderate	6×10^{-5} - 2×10^{-4}	Few partly open	± 1
15-50	Medium	2×10^{-4} - 6×10^{-4}	Some open	± 5
50-100	High	6×10^{-4} - 1×10^{-3}	Many open	± 10
>100	Very High	$>1 \times 10^{-3}$	Open closely spaced or voids	>100

2.4.3 Case Studies on Field Permeability Test

(a) *In-situ Permeability Studies For Hidkal Dam, Karnataka*

Reservoir seepage in terms of water pool formation at downstream side of earthen dyke 1 of reservoir of Hidkal Dam in Belgaum, Karnataka has been reported to be persistent ever since the impoundment of reservoir. Hydrological studies conducted earlier by CWPRS delineated the stretch of the seepage zone based on which four NX size trial bore holes of depth 15m each on either side of dyke1 between Ch. 4700 m to 5100 m have been prepared and geologically logged by the project authority. CWPRS has carried out field permeability studies to determine quantum of seepage for designing effective treatment measures (CWPRS Technical Report no. 4357). The principal rock type met with in the foundation has been Deccan Trap basalt with various degrees of weathering ranging from highly weathered to disintegrated variety including occasional presence of quartzites. Basalt as outcrop at both upstream and downstream side of the delineated seepage zone of the dyke has undergone spheroidal weathering (Fig.2.21) which continued further up to substratum levels and has been confirmed from the drilling logs and as well as inspection of rock cores from the trial bore holes. Water has been found to be overflowing from almost all boreholes except a few (Fig 2.22).

Fig. 2.21 Spheroidal weathering of Basalt as outcrop

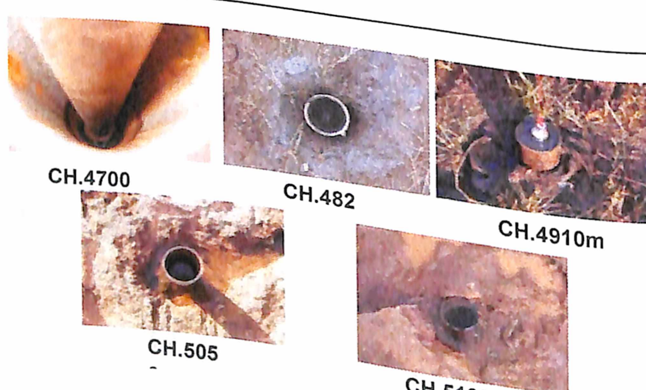


Fig.2.22 View of water table at the test boreholes

In-situ permeability tests have been carried out in five bore holes at the downstream toe of dyke1.Due to weathered state of surface strata each borehole has been provided with a collar and after thorough washing and cleaning, the boreholes have been capped. Before testing, the water level in each borehole has been recorded. The tests have been conducted in each borehole starting from the bottom most section and gradually shifting the packer to higher elevations till it reaches the section just below the collar pipe. Double packer test assembly of 54mm diameter has been used with length of test section as 1.5 m between packers and with a maximum pressure of 0.25 kg/cm^2 per meter of rock cover above the centre of each stage. Heavy water losses even in low pressures have been observed for most test sections and permeability values have been found to be greater than 5 lugeons at those locations. Due to weathered and fractured nature of the rock, for some of the test sections within top 10m from ground level, it has not been possible to isolate the test section properly from the remaining part of the bore hole even after full tightening of the packer which in turn has resulted in loosening of top strata (Fig.2.23 and Fig 2.24) and heavy leakages in certain cases have also been observed after application of test pressure.

Fig.2.23 Losening of top weathered strata during test for borehole at Ch. 4820 m

Fig.2.24 Water gushing out due to collapse of side wall boundary at depth for borehole at Ch. 5050 m

From average permeability values of each test section, it was observed that except a few locations beyond 8 m depth of borehole at chainage 4700 m, almost every section for other boreholes has shown very high permeability values mostly greater than 100 lugeons, suggesting the need for curtain grouting at the upstream as a preventive measure for seepage.

(b) *In-situ Permeability Studies For Maskinala Dam, Karnataka*

23.74 m high Maskinala dam is an earthen dam with central concrete spillway with gross storage of 0.5 TMC over river Maski Nala near Maraladinni village of Lingsugur Taluka of Raichur,Karnataka.The 57m long Ogee type spillway with 4nos of radial gates of sizes 12m×8.5m is designed for a max. discharge capacity of 2590 cumecs. The purpose of the dam is to irrigate an area of about 4793 acres and 2693 acres of land through left bank and right bank canal respectively. Foundation rock of all 5 blocks of spillway is of massive and fine to coarse grained, pink to grey color variety of granite gneiss without any major joints except a few sparsely spaced minor tight joints. In order to decide the water-tightness of foundation rock mass at the spillway section, before taking up the final grouting work in spillway block no.4 and 5,four NX size boreholes has been drilled at the upstream portion of the monolith 4 and 5 of the spillway portion(since the concrete was already laid in all 5 blocks of spillway). Borehole nos. 1 and 2 at spillway block no.4 has been drilled to 11m depth whereas borehole nos.3 and 4 has been drilled up to depths 10.9m and 8.7m respectively (Fig.2.25).

Percentage core recovery in the first three borehole has been found to be varying between 70% to 90% whereas for the fourth borehole, the recovery varies between 44% to 80% till depth 13m but reduces to 2% to 14% till depth 21m indicating the presence of weak geological features.

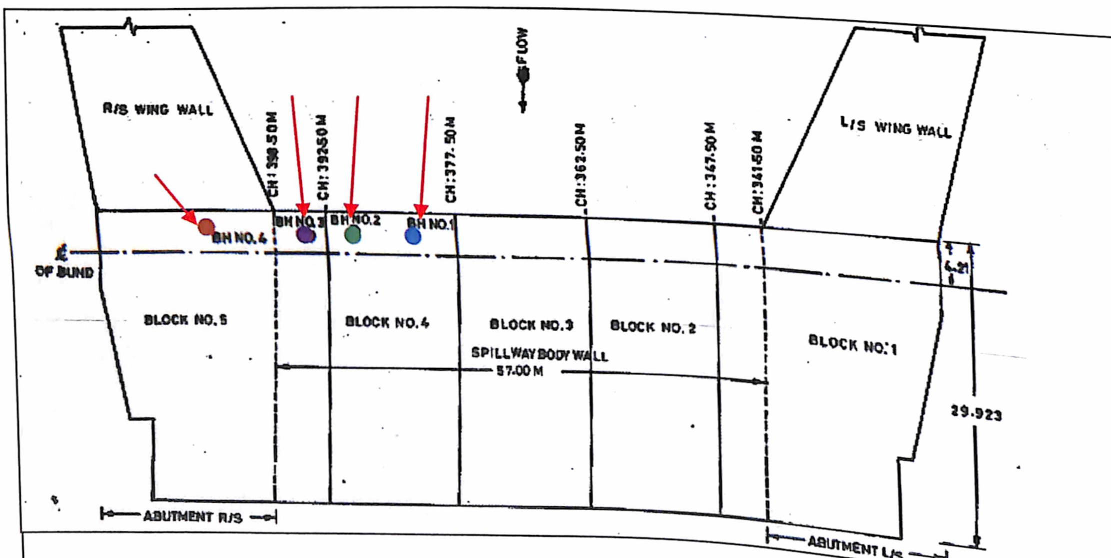


Fig.2.25 Sketch showing layout of boreholes

Before taking up the foundation grouting works, in-stu permeability studies (pumping in test) have been conducted (CWPRS Technical Report no. 3591) in uncased and ungrouted sections of these boreholes in a stage wise fashion using both single and double packers as per IS 5529(Part II). A typical permeability test in progress for one of the boreholes is shown vide Fig.2.26. For each of the borehole, the location of the groundwater table has been taken as that of the foundation for computational purpose. For each section, test pressure is maintained until the readings of water intake at interval of 5 minutes show a nearly constant reading of water intake for one particular pressure at the collar.

Fig. 2.26 Field Permeability test in progress

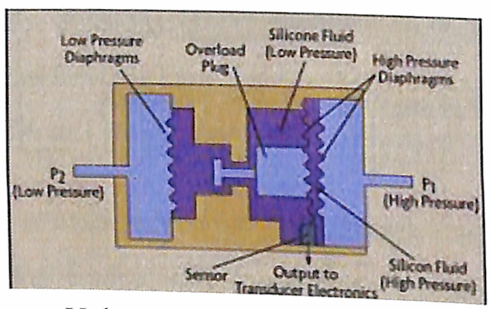
The average permeability values has been found to be varying between 0.03 lugeon to 28.6 lugeons. In block no.4 in both the boreholes, the permeability values are less than 1 lugeon suggesting very tight rockmass conditions. In block no.5, though at borehole no.3, the rock mass conditions are watertight (0.78 lugeon), at borehole no.4 near to the right bank wing wall and present water channel of Maskinala, the permeability values have been found to be much higher(28.6 lugeons) indicating presence of weak geological features such as shear zones/open joints etc. warranting grouting as a preventive seepage control measures.

2.5 EVALUATION OF UPLIFT AND PORE PRESSURE

An effective instrumentation and monitoring programme combined with regular inspection are the key features of a good dam safety programme. Among other parameters, Seepage or leakage through the dam and foundation is one of the major parameters required to be measured in Dam Safety Monitoring (ICOLD, Bulletin 60,1988). Seepage has both a physical and a chemical influence on the concrete and plays a noticeable role on the state of stresses and the stability of the dam. In concrete dams, the seepage occurs at the bottom of the dam as well as through any plane because of differential pressure gradient from upstream to downstream and is known as Uplift pressure. Further, depending on the composition and grade of concrete used for the construction, dams contain a certain percentage of pores. The seeping water through dam body in due course, may fill in these pores and exerts pressure known as pore pressure. Both uplift and pore pressures are destabilizing forces are required to be measured and monitored.

2.5.1 Instruments for Pore Pressure & Uplift Pressure

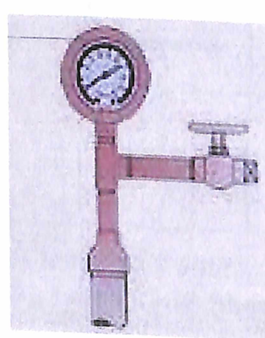
Various types of piezometrers (BIS: 7436,Part I and II(1976),BIS 8282,Part II(1976)) are installed in the body of dam for pore pressure measurement and also in the body of the dam and foundation for uplift pressure measurement. Different types of pore and uplift pressure measuring instruments available in the market are shown vide Figs.2.27 and Figs 2.28 respectively.



Unbonded strain gauges with Diaphragm

Bonded strain gauges / Semiconductor with Diaphragm

Fig 2.27 Typical Pore Pressure Measuring Instruments



Mechanical Uplift Pressure Meter

Vibrating Uplift Pressure Meter

Fig.2.28 Uplift Pressure Measuring Instruments

2.5.2 Monitoring

The purpose of instrumentation and monitoring is to maintain and improve dam safety. If threshold levels are reached within a short period of time, investigations and remedial actions need to be implemented. Variations from expected uplift and pore pressure data may suggest development of adverse conditions in dam. For example, in a concrete gravity dam, increasing uplift pressure, or decreasing drain flow, may indicate foundation drains are chocked and need to be cleaned. A routine regular maintenance of instruments, readout devices, and field terminals need to be established. Normally, Pore and Uplift pressure data need to be recorded on fortnightly basis; however, in case of sudden change in water levels, frequency of measurement can be increased. Data collected manually/ automatically including complementary data, such as air temperature, reservoir level, recent precipitation, and other information or observations need to be analysed. Data is required to be within the limits of the instrument and to be compared against previous measurements and threshold limits in the field, to identify erroneous measurements. After initial scrutinizing of the data, careful analysis is to be carried out and is required to be reviewed for reasonableness, evidence of incorrectly functioning instruments and transposed data. The results can be shown in the form of plots of instrument data with respect to time period and reservoir level, which are self-explanatory. Scales should be consistent to allow comparison of data between plots and labeled. Trends of measurements toward threshold levels need to be identified and evaluated. Results are required to be compared with expected behavior based on the basic engineering concepts. A case study explaining the understanding of dam behavior from instrumentation data analysis is given below.

2.5.3 Case Studies on Evaluation of Uplift and Pore Pressure

Analysis of Dam Instrumentation Data of Indira Sagar Dam

To study the post construction structural behaviour of the dam, various frequency based vibrating wire type instruments have been installed in block 25 (Fig. 2.29) of the dam during construction. Data from all the installed instruments are recorded by project officials every fortnightly. Analysis and interpretation of this data is being carried out by CWPRS since 2003-04 (CWPRS Technical Report No.4820).

Analysis of uplift data based on foundation piezometers has shown that the measured uplift is more than the theoretically-calculated value (Fig 2.30) in dam base towards d/s side. To confirm the same, further tracer studies have been carried out and findings from the study indicated that water is being percolated from surrounding right abutment and enriching the flow at lower part of dam base. During the first site visit it was found most of the relief holes were chocked and requested to clean the same. It was observed that uplift pressure reduced after cleaning the relief holes.

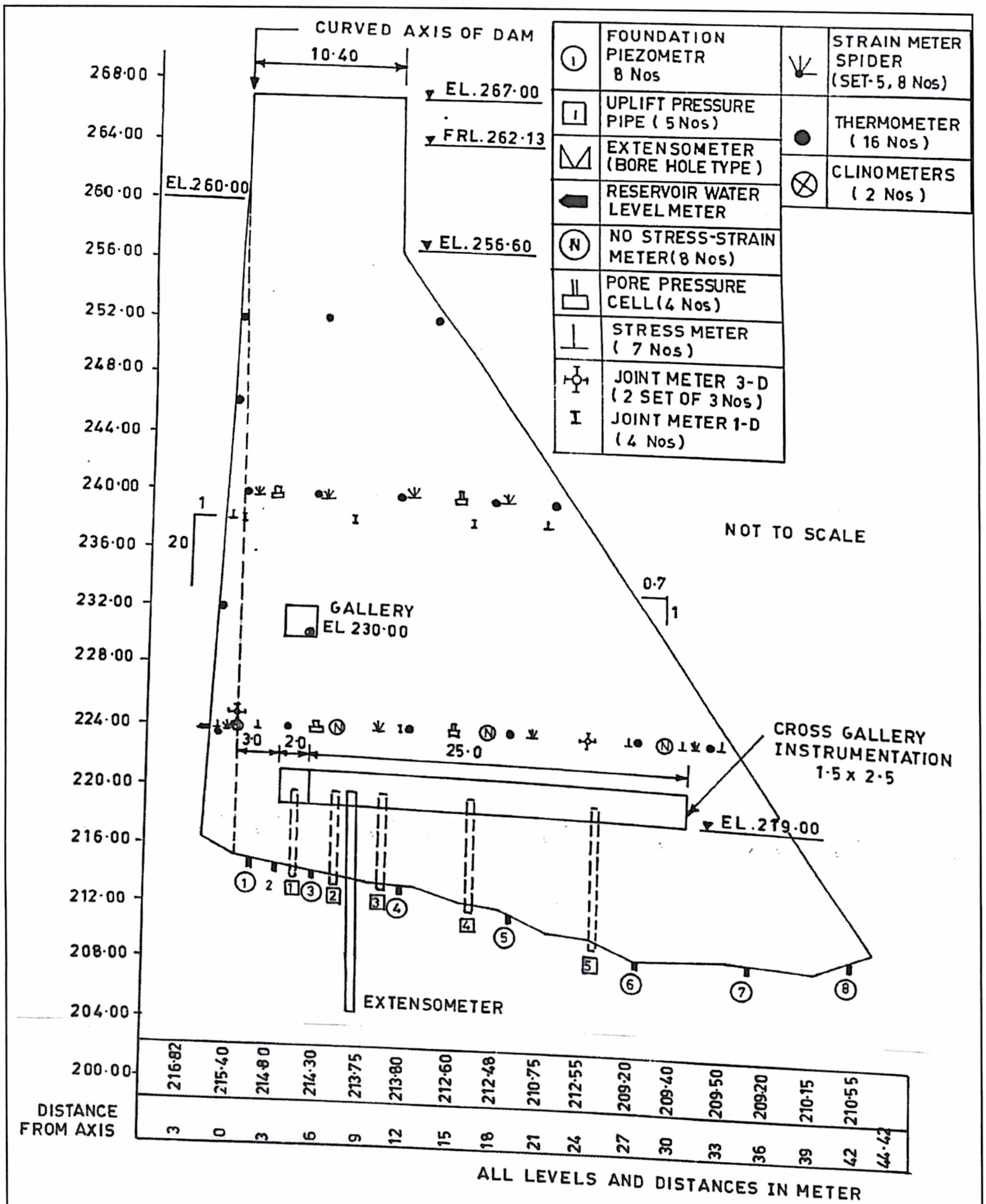


Fig.2.29 Location for Instruments in Block No 25

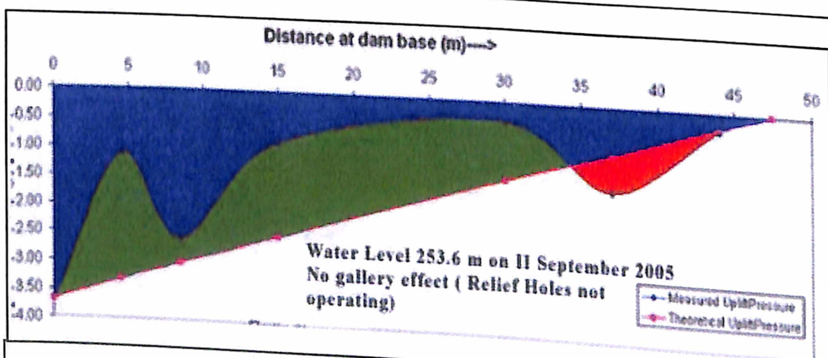


Fig.2.30 Higher Uplift pressure near D/s side

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CHAPTER III
EARTH AND ROCKFILL DAMS

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3.0 INTRODUCTION

The basic requirements for design of earth dam are to ensure a) safety against overtopping, b) stability and c) safety against internal erosion due to seepage. These requirements are often interrelated in a complex manner. For example, uncontrolled seepage may weaken the soil and lead to a structural failure. A structural failure may shorten the seepage path and lead to a piping failure. Surface erosion may result in structural failure. Minor defects such as cracks in the embankment may be the first visual sign of a major problem which could lead to failure of the structure.

Generally, in embankment dam's water passage is through body and foundation since all earth materials are porous. An uncontrolled and excessive seepage progressively erodes soil from the embankment or foundation, resulting in rapid piping, which may lead to failure of the dam. Slope failures are also caused by creating high water pressures in the soil pores or by saturating the slope. Assessment of seepage and early detection of piping is essential to avoid catastrophic incidences of dam failures. The mathematical modeling is routinely used for assessment of seepage through embankment dams.

3.1 MECHANISM AND MONITORING OF SEEPAGE

The main causes of occurrence of seepage through earthen dams are i) piping/ erosion and ii) pore pressure developed.

3.1.1 Piping/ Erosion

The flow of water through a pervious soil produces seepage forces as a result of the friction between the percolating water and the walls of the pores of the soil through which it flows. Fig 3.1 shows the flow path of water through the pervious foundation of a dam. The water percolating downward at the upstream toe of the dam adds the initial seepage force, F_1, to the submerged weight of the soil, W_s, to produce the resultant body force, R_1. As the water percolates upward at the downstream toe of the dam, the seepage force tends to lift the soil, reducing the effective weight to R_4. If exit seepage force,F_4 exceeds W_s, the resultant would be acting upward and the soil is carried out / eroded / "piped out." If the foundation materials are similar throughout, the erosion could progress backwards along the flow line until a "pipe" is formed to the reservoir, allowing rapid escape of reservoir storage and subsequent failure of the dam. This action can occur rapidly or can be slow.

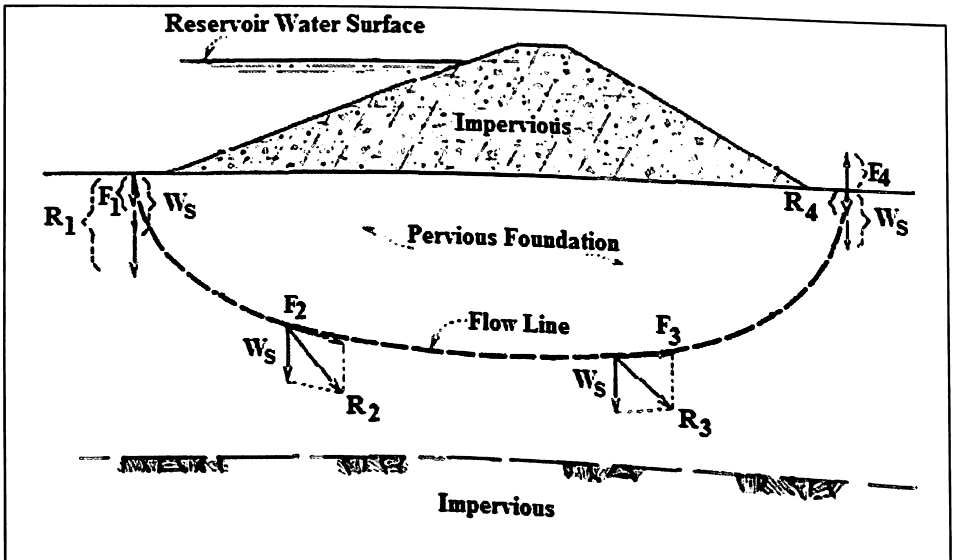

Fig 3.1: Seepage Forces

The Teton earth Dam of 100 m high which failed completely by piping during the first filling of the reservoir in June,1976, was a significant event. If a more impervious layer at the surface overlies a pervious foundation, sudden upheaval of the foundation at the downstream toe of the dam can occur, called "blowout". Relatively impervious foundations are not usually susceptible to piping because impervious soil offers a greater resistance to seepage forces and, consequently, to displacement.

3.1.2 Pore Pressure

In the seepage flow region of the soil, the fluid pressure is developed which depends on the permeability of the soil, head difference on u/s and d/s ends, length of seepage path etc. In hydraulic structures, such as barrages, the 'pore pressure' acting on the bottom of the floor of the structure exerts an upward pressure, called 'uplift pressure' which is detrimental to the safety of the structure. In such cases, the uplift pressure at the bottom floor of the structure is reduced by constructing cutoff piles on d/s and u/s ends.

Regular monitoring is essential to detect seepage and prevent dam failure. Instrumentation should be used to monitor seepage. The two instruments that are essential to monitor and measure seepage are – i) V-notch weir and ii) Piezometers.

3.1.3 V-notch weirs : It should be used to measure seepage flow rates. The measured values can be compared with computed seepage rates from mathematical modeling. Any increase in the flow rate and also the quality of water seeping out should be monitored thoroughly. If flow rate changes unusually or water with soil particles emerge from the downstream, should be immediately reported.

3.1.4 Piezometers : Simple Casagrande type piezometers or Electronic vibrating wire type piezometers, should be used to determine the pore pressure within the embankment and foundation. The piezometers reading on the d/s slope should be minimum and should not increase with reservoir water level. This indicates the filter is functioning as desired. Similarly, piezometer reading on d/s foundation give the effectiveness of cut-off trench and grouting.

Regular maintenance of the internal embankment and foundation drainage outlets is required. The rate and content of flow from each pipe outlet for toe drains, relief wells, weep holes, and relief drains should be monitored and documented regularly. Normal maintenance consists of removing all obstructions, such as sediment, mineral deposits, calcification of concrete, and rodent nests etc, from the pipe to allow for free drainage of water from the pipe.

The performance of the earth dam can be predicted by conducting mathematical analysis. The relevant input parameters of soil such as permeability, density are determined from laboratory and field tests.

3.2 SEEPAGE ANALYSIS

The flow of water through soil obeys Darcy's law. For a given soil type and for a given boundary conditions of water heads, the movement of water in the soil is governed by Laplace's equation. Solution to this equation, gives the assessment of seepage force, seepage quantity, hydraulic gradient etc. in the flow region. There are many methods to solve this equation e.g. i) Physical seepage models such as electrical method, sand models, etc, ii) Mathematical seepage modeling by Analytical methods and Numerical methods. In the earlier days, before the commencement of computers, the analytical methods were routinely used for seepage analysis. However, with the advent of computers and software, the numerical modeling has gained popularity because of its ease of usage in multi-layered soil strata and zoned earth structures.

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3.2.1 Seepage through Porous Media with boundary conditions

The saturated soil which is considered for analysis must be defined by boundaries, permeability of the soil, and heads imposed upon the water. The nature and location of these boundaries are determined by a soils exploration program, assumptions based on engineering judgment and conditions imposed by the proposed design. There are two general cases of seepages through soil: i) confined and ii) unconfined flow. In confined flow all boundaries are defined and soil mass does not have a line of seepage boundary. In unconfined flow, surface of seepage and the line of seepage must be defined in the analysis. Generally, seepage analysis problems associated with dams have four possible types of boundaries. [Fig. 3.2] (EM 1110-2-1901, 1986)

a) Impervious Boundary

The interface between the saturated, pervious soil mass and adjacent materials such as a very low permeability soil or concrete is approximated as an impervious boundary. It is assumed that no flow takes place across this interface, thus flow in the pervious soil next to the impervious boundary is parallel to that boundary.

b) Entrances and Exits

The lines defining the area where water enters or leaves the pervious soil mass are known as entrances or exits, respectively. Flow is perpendicular to an entrance or exit.

c) Surface of Seepage (seepage face)

The saturated pervious soil mass may have a boundary exposed to the atmosphere and allow water to escape along this boundary, line GE.

d) Line of Seepage (free surface)

This boundary is located within the pervious soil where water is at atmospheric pressure, line DG. The line of seepage is not known until the flow distribution within the pervious soil is known. The flow in the soil is parallel to the boundary.

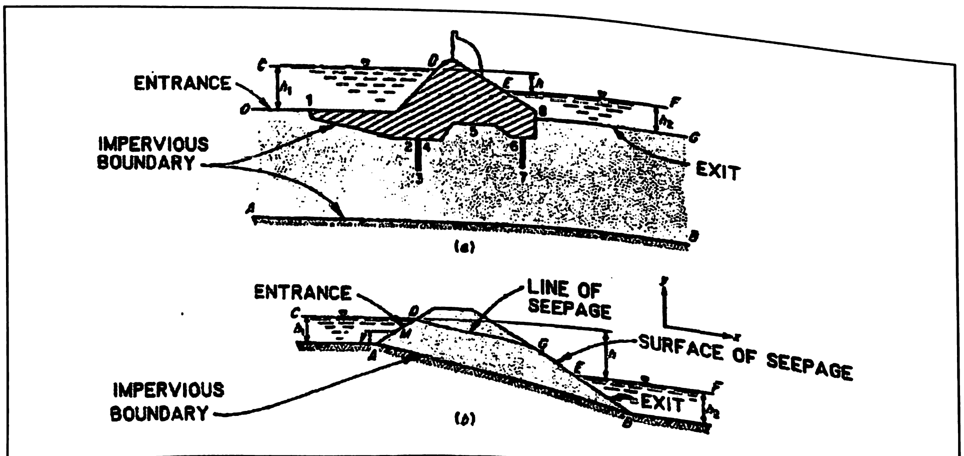


Fig. 3.2 : Boundary Conditions for Confined & Unconfined Flow

3.2.2 Laplace Equation

The Laplace equation is the mathematical basis for several models used in seepage analysis. Following assumptions are made to develop it:

a) The soil is homogenous
b) The voids are completely filled with water
c) No consolidation or expansion of the soil takes place
d) The soil and water are incompressible
e) Flow is laminar and Darcy's law is valid.

The equation of continuity state that the quantity of water entering and leaving an element of soil is equal., Eq. 3.1

$$\frac{\partial u}{\partial x}+\frac{\partial v}{\partial y}+\frac{\partial w}{\partial z}=0 \qquad \text{... Eq. (3.1)}$$

Where u, v, w are the components of discharge velocity in x, y and z directions respectively.

Darcy's law (Eq.3.2), which relates discharge velocity to variation of water head (h) and permeability of the soil medium (k), is

$$u=-k\frac{\partial h}{\partial x} \qquad v=-k\frac{\partial h}{\partial y} \qquad w=-k\frac{\partial h}{\partial z} \qquad \text{... Eq. (3.2)}$$

Laplace equation in three dimensional flow of water through porous media is obtained from above relations as,

$$\frac{\partial^2 h}{\partial x^2}+\frac{\partial^2 h}{\partial y^2}+\frac{\partial^2 h}{\partial z^2}=0 \qquad \text{... Eq. (3.3)}$$

This equation is used to solve steady-state flow through earth dam and foundations. There are different types of models available to solve this equation. (M.E.Harr,1981)

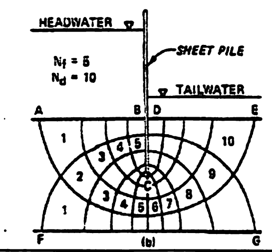
There are two main approaches for solving Laplace's equation for seepage through hydraulic structures. They are:

i. Analytical method and
ii. Numerical method.

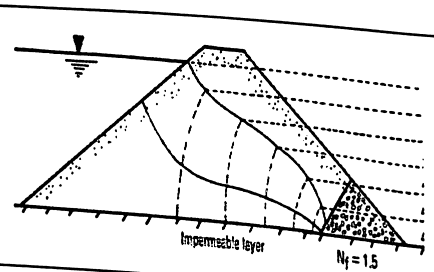
Analytical method consists of different methods such as Transformations and mapping method, Method of Fragments, Closed form solutions, Graphical flow net method etc. Numerical methods consist of use of Finite Element method and Finite Difference methods to solve Laplace equations.

3.3 ANALYTICAL APPROACH

Flow net method is one of the most widely used methods for seepage analysis. It can be adapted to many of the under seepage and through-seepage problems in dams and other hydraulic structures. Flow net consists of Flow lines and Equi-potential lines, which are perpendicular to each other. The flow lines represent paths along which water can flow through a cross section. The equipotential lines are lines of equal level or head. If boundary conditions and geometry of a flow region are known then a flow net provides a strong visual sense of what is happening in the flow region. The flow net is a singular solution to a specific seepage condition, i.e., there is only one family of curves that will solve the given geometry and boundary conditions. The flow lines and the equipotential lines are drawn by trial and error method to construct flow net. Fig. 3.3 shows flow net for a sheet pile wall in permeable foundation and unconfined flow through an earth dam.



Sheet pile wall in permeable foundation flow



A flow through an earthen dam

Fig 3.3 : Flow Net for Unconfined and Confined flow

3.3.1 Assumptions for Flow Net Construction

Basic properties of the seepage problem must be known or assumed in order to draw a flow net:

a) The geometry of the porous media must be known.
b) The boundary conditions must be determined
c) The assumptions required to develop Laplace's equation must hold
d) The porous media must be homogeneous and isotropic

The properties of a flow net are as follows:

i. Flow and equipotential lines are smooth curves.
ii. Flow lines and equipotential lines meet at right angles and make curvilinear squares.
iii. No two flow lines cross each other.
iv. No two flow or equipotential lines start from the same point.

In case of unconfined seepage through earth dams, phreatic line needs to be established first for construction of flow nets. The flow net is drawn by the method of parabola as shown in Fig 3.4. Curve a-e-f-b is the actual phreatic line. The parabola a′-e-f-b′-c′ drawn with its focus at c, coincides with the actual phreatic line, but with some deviations at the upstream and the downstream faces.

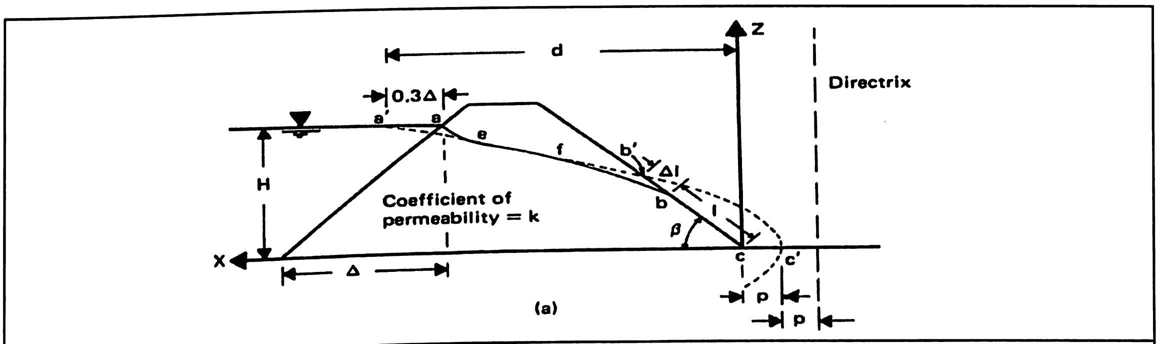
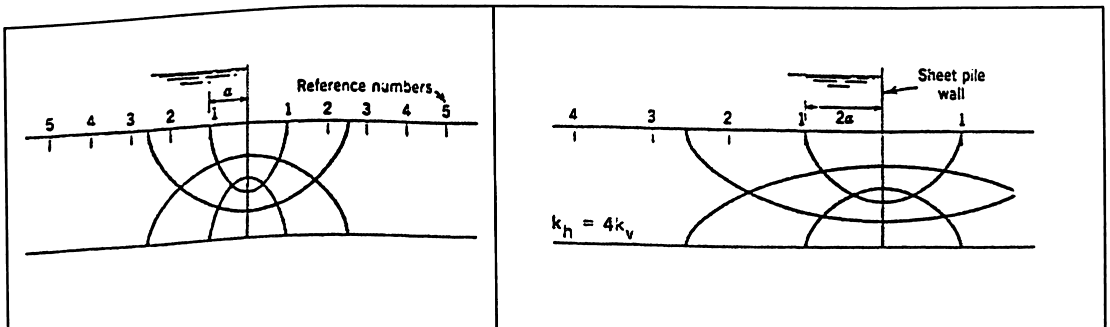


Fig 3.4 : Determination of Phreatic Line for Seepage through an Earthen Dam

3.3.2 Flow Net for Anisotropic Soil

It is observed that naturally occurring soils and man-placed soils have horizontal permeability (k_h) greater than vertical permeability (k_v). This affects the shape of a flow net since Laplace's equation is based on the assumption of an isotropic porous media. In order to compensate for anisotropy, the dimensions of the porous media are changed by the ratio of $\sqrt{(k_v / k_h)}$. The same ratio is applied to all other horizontal dimensions to produce a transformed section. The flow net is drawn on the transformed section (Fig 3.5). Then the section, including the flow net, is returned to the original (true section) which produces a non-square flow net. Computations are made using the non-square flow net. In the same manner, dimensions in the vertical direction could be changed by the factor $\sqrt{(k_h / k_v)}$, square or normal flow net drawn on the transformed section, then returned to true section. Pore pressure distribution and hydrostatic uplift may be computed from either section while hydraulic gradient and seepage forces must be determined from the true section.



$k_h = k_v$	$k_h = 4k_v$

Fig 3.5 : Flow Nets Constructed on Transformed Section and on True Section

3.3.3 Flow Net for Composite Sections

Seepage analysis involving different soils with different permeabilities e.g., stratified foundation materials and zoned dams is required follow certain rules for flow lines, equipotential lines, and lines of seepage crossing internal boundaries. Fig. 3.6 illustrates the deflection of flow lines and equipotential lines at interfaces. It should be noted that when flow goes from lower permeability soil to higher permeability soil, the distance between flow lines decreases (flow channel gets smaller) and the distance between equipotential drops increases. Fig. 3.7 shows flow net construction for seepage through soils of differing permeabilities. Flow lines and equipotential lines maintain continuity across the interface between the soils though direction will change abruptly. Additionally, the number of flow channels must remain constant throughout the flow net. (Cedergren, 1989)

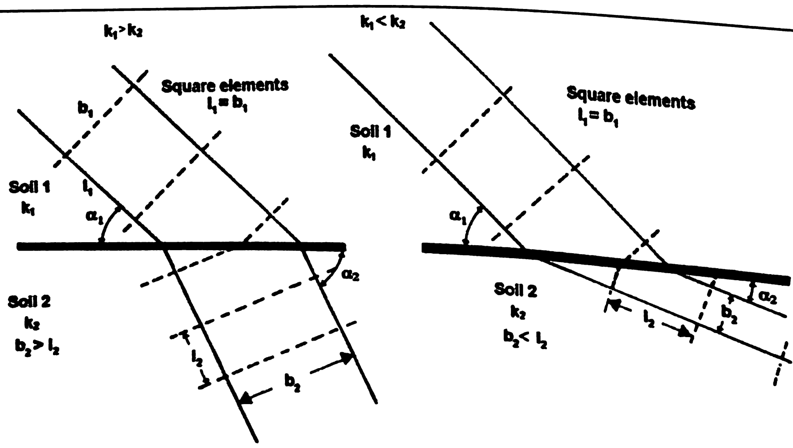


Fig 3.6 : Deflection of Flow Lines at the Interface of Different Permeability

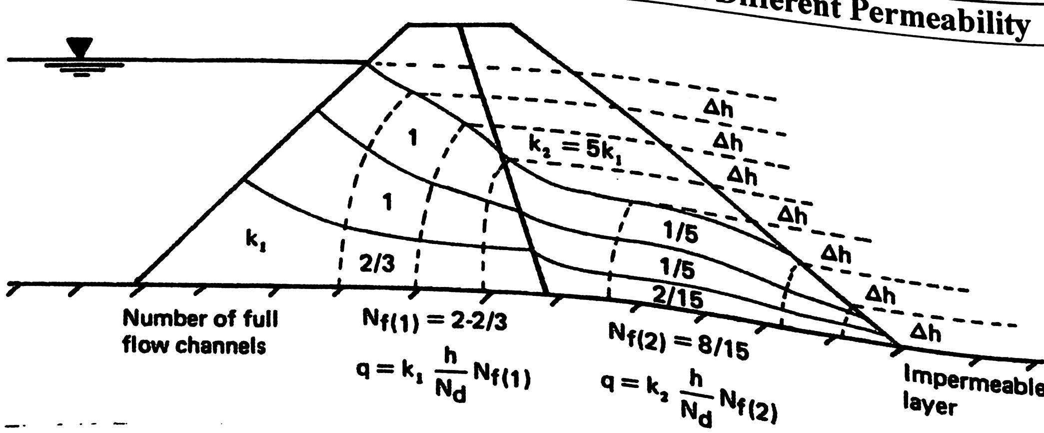


Fig 3.7 : Flow Net for Seepage through a Zoned Earth Dam

3.3.4 Determination of Seepage Parameters

A flow net is a picture of seepage conditions under given geometry and boundary conditions. It explains how pressures are distributed and where flow is being directed. The flow net gives important information about stability and flow quantity in two-dimensional idealization of the real situation. The seepage parameters determined from the flow net are:

a) Seepage quantities
b) Exit and Critical gradient
c) Heave
d) Seepage Force
e) Uplift Pressure

Each of the seepage parameter is explained briefly in following paragraphs.

a) Seepage Quantities

In a flow net, an equal volume of water per unit of time is passing through each of the complete flow channels. The total head (h) applied across the flow net is divided into equal portion of equipotential lines. In a flow net, N_f is the number of flow channels, (including any partial channel), and N_d is the number of equipotential drops, (including any partial drops). The ratio of (N_f / N_d) is called the shape factor, S, which is a characteristic of the given geometry, boundary conditions and permeability ratios (k_v/k_h). Using flow net, the quantity of flow per unit length (q) through the porous media can be determined as follows:

$$q = k \frac{h}{N_d} N_f \qquad \text{... Eq. (3.4)}$$

Flow net shown in Fig. 3.8 has $N_f = 2.3$, $N_d = 10$, h = 25 m and Permeability of soil , k=1.0 x 10^{-6} m/sec, which gives seepage quantity, q = 5.75 x 10^{-6} m^3 /sec. per unit length.

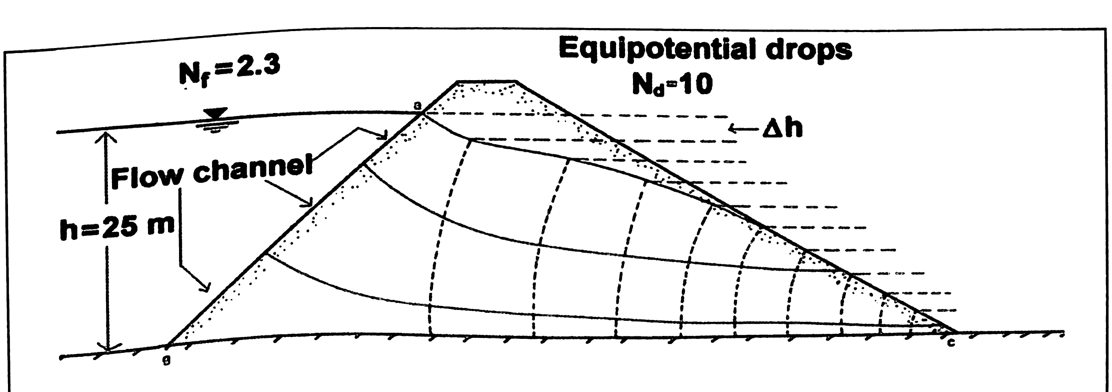


Fig 3.8 : Seepage Quantity Computations from Flow Net

b) Escape and Critical Gradients

The area in the hydraulic structure where seepage is exiting the porous media is characterized by the escape or exit gradient, which is the rate of loss of head per unit of length. For confined flow, the area of concern is usually along the uppermost flow line near the flow exit, e.g., at the downstream edge of a concrete or other impermeable structure. If the escape gradients for flow through embankments are too great, soil particles may be removed from downstream. This phenomenon, called flotation, can cause piping (the removal of soil particles by moving water) which can lead to undermining and loss of the structure.

The gradient at which flotation of particles begins is termed the critical gradient, i_{cr} . It is determined by the in-place unit weight of the soil and is the gradient at which upward drag forces on the soil particles equal the submerged weight of the soil particles. It is dependent on the specific gravity and density of the soil particles and can be defined in terms of specific gravity of solids, Gs , void ratio, e , and porosity, n :

$$i_{cr} = (G_S - 1)(1-n) \quad \text{OR} \quad i_{cr} = \frac{G_S - 1}{1+e} \qquad \text{... Eq (3.5)}$$

If typical values of Gs and e or n for sand are used in the above equations, i_{cr} will be approximately 1.0. Generally, factors of safety in the range of 4.0 -5.0 (Harr 1981) or 2.5 – 3.0 (Cedergren 1989) have been proposed.

c) Heave

A phenomenon called heave occurs when a mass of soil is lifted initially and followed by piping. This happens when the upward seepage force due to differential head equals the overlying buoyant weight of soil. Heave occurs under conditions of critical hydraulic gradient. For field conditions, the point at which minimum differential head offsets the overlying buoyant weight must be determined by calculations. Resistance to heave may be developed by placing very pervious material on the exit face, which will allow free passage of water but add weight to the exit face and thus add downward force. This very pervious material must meet filter criteria to prevent loss of the underlying soil through the weighting material.

d) Seepage Forces

Forces imposed on soil particles by the drag of water flowing between them are called seepage forces. They must be considered when analyzing the stability of slopes, embankments, and structures subject to pressures from earth masses. The magnitude of this force on a mass of soil is determined by i) the difference in piezometric head on each side of the soil mass, ii) the weight of water, and iii) the area perpendicular to flow.

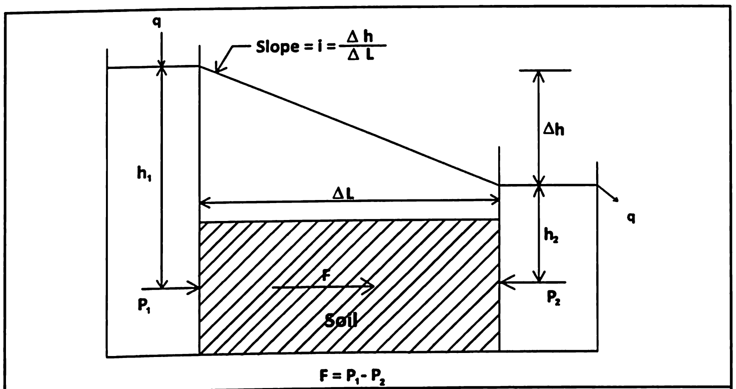


Fig 3.9 : Seepage Force

The seepage force acts in the direction along flow lines. The seepage force on plane A-A in Fig 3.9, is

$$F = \Delta h \, \gamma_w \, A \qquad \text{... Eq (3.6)}$$

e) Uplift Pressure

When seepage occurs beneath concrete or other impermeable structures or strata, the underside of this impermeable barrier is subject to a force which tends to lift the structure upward. The determination of this pressure or force is important in analyzing the stability of the structure. Summing of the uplift pressures over the bottom area of the spillway will give the total uplift force on the structure for a stability analysis.

An illustration of uplift pressure from flow net is given below (Fig 3.10). A proposed concrete barrage of 80 m wide, 1000 m long and 2.0 m thick is analyzed for uplift pressure. The crest of the barrage is 61.0 m and top of downstream floor is 55.0 m. Maximum water level is 70.0 m and tail water level is 55.0 m. The hydraulic head of 15.0 m corresponds to 100 % potential along CD and potential along AB on downstream is 0%. The potential lines for the seepage flow are given in Fig 3.10. The Potential along F-F is 60% corresponds to a Potential head of 9.0 m (i.e. 60% of 15.0 m). The elevation of energy head, h_{60}, is 64.0 m (i.e. = 55.0 m+9.0 m) at F-F. The uplift pressure head at the bottom of the barrage at elevation Z= 53.0 m (P_{53}) is computed from hydrostatic equation:

$$h_{60} = \frac{P_{53}}{\gamma_w} + z$$

$$P_{53} = (64 - 53)\ 1000\ Kg/m^2$$
$$P_{53} = 11000\ Kg/m^2\ =\ 1.1\ Kg/cm^2$$

Resisting force offered by 2.0 m thick concrete floor of density of 2400 Kg/m^3, is 4800 Kg/m^2. Considering factor of safety of 1.2, resisting force offered by concrete floor is 4000 Kg/m^2 (= 4800 / 1.2), which is inadequate to withstand uplift pressure of 11000 Kg/m^2. A concrete floor of 5.5m thick will be required to resist the above uplift pressure having Factor of safety of 1.2. (= 5.5 * 2400 / 11000). The design of floor thickness can be arrive at from the flow net.

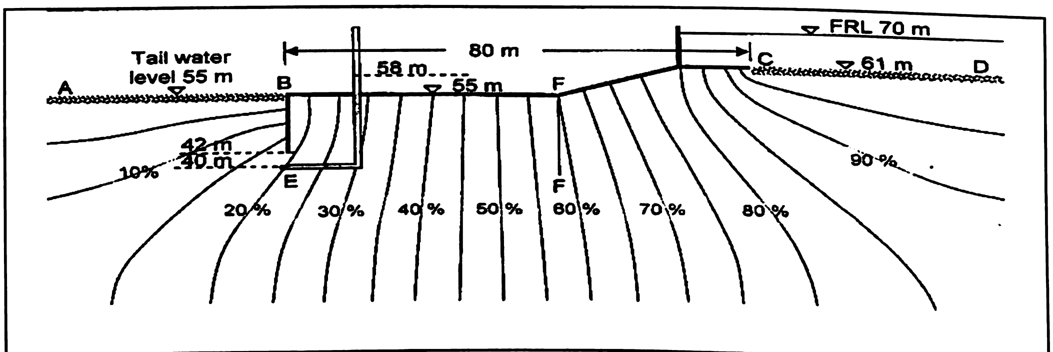


Fig 3.10 Equi-Potential Lines under a Barrage

3.4 NUMERICAL MODELS

Computer models are used to make acceptable approximations for the Laplace equation in complex flow conditions. The two primary methods of numerical solution are finite difference and finite element. Both can be used in one, two, or three-dimensional modelling. Several computer programs for these methods are available commercially.

3.4.1 Finite Difference Method

This method solves the Laplace equations by approximating them with a set of linear algebraic equations. The flow region is divided into a discrete rectangular grid with nodal points which are assigned values of head (known head values along fixed head boundaries or points, estimated heads for nodal points that do not have initially known head values). Using Darcy's law and the assumption that the head at a given node is the average of the surrounding nodes, a set of N linear algebraic equations with N unknown values of head are developed (N equals number of nodes).

Advantages:

Confined and transient flow problems can be solved by use of iterative techniques.

Disadvantages:

It is not suited to complex geometry, including sloping layers and pockets of materials of varying permeability. Irregular grids are difficult to input. Therefore, zones where seepage gradients or velocities are high cannot be accurately modeled.

3.4.2 Finite Element Method (FEM)

This method is also based on grid pattern (triangular or rectangular) which divides the flow region into discrete elements and provides N equations with N unknowns. Material properties, such as permeability, are specified for each element and boundary conditions (heads and flow rates) are set. A system of equations is solved to compute heads at nodes and flows in the elements.

Advantages:

a. Complex geometry including sloping layers of material can be easily accommodated.

b. By varying the size of elements, zones where seepage gradients or velocity are high can be accurately modeled.

c. Pockets of material in a layer can be modeled.

Disadvantages: This method is usually more costly.

3.4.3 Seepage Analysis by FEM

In an unconfined seepage through earth dam, the free (phreatic) surface is not known. The objective of the solution is to determine the elevation of the free surface. This surface is considered as boundary with a given head, since the pressure along the free surface is zero. On the other hand, this free surface is also a stream line, because flow is steady. That is free surface is a boundary with two boundary conditions, with an unknown location. This difficulty is overcome by iterative procedure. First, an initial estimate of the free surface is introduced, as a straight line or by Dupit's approximate formula. The free surface is then regarded as a stream line (i.e. impermeable boundary) with an unknown head distribution. The distribution of the piezometric head in the interior of the domain and on its boundary is calculated by using the finite element method. This will lead to values for the head at the points on the free surface. These heads can be compared with the elevation of these points. The iterations are continued till difference between computed head and elevation of the free surface is minimum. Fig.3.11 shows flow net by Finite Element through dam.

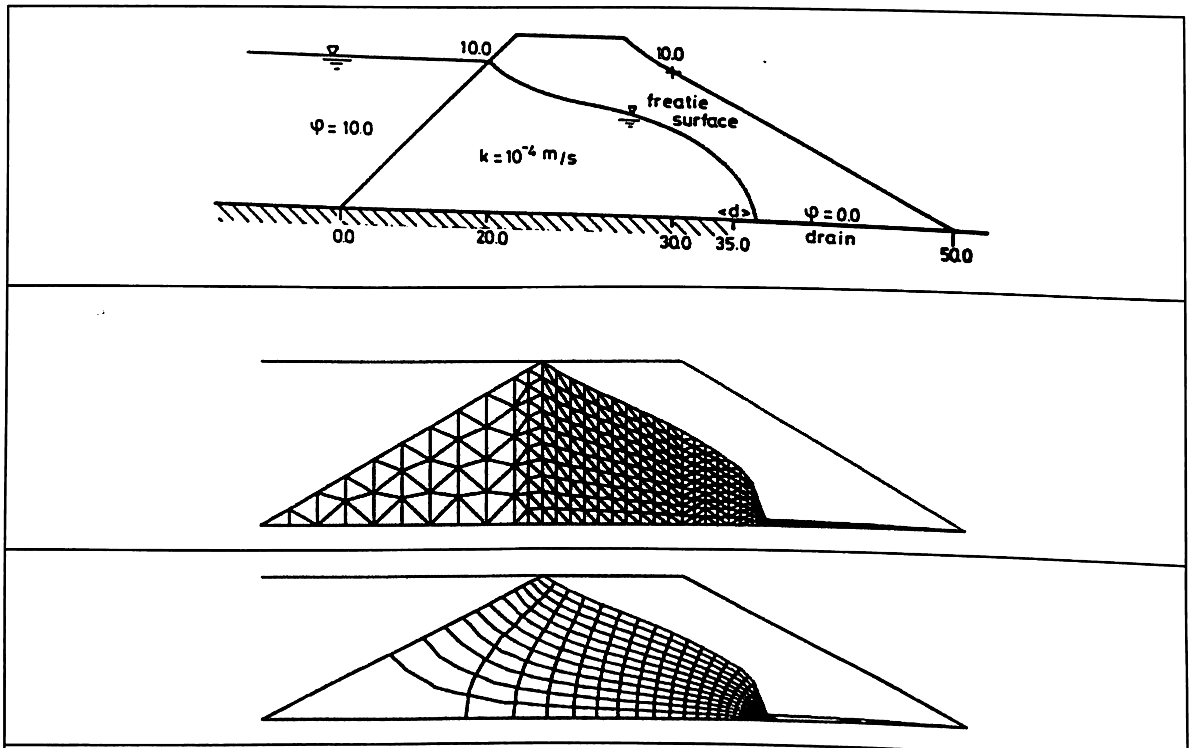


Fig 3.11 Flow Net for Seepage through Embankment using Finite Element Software

Seepage analysis plays an important role in deciding the appropriate seepage control measures to be adopted for a given earthen embankment dam and foundation conditions. Analytical and numerical methods prove to be very useful in working out various combinations of permeability to different zones and foundation to compute acceptable level seepage parameters (Zienkiewicz, 1993).

3.5 CASE STUDIES ON SEEPAGE INDUCED STRESSES

Seepage Induced Slope Distress of Dudhawa Earthen Dam, M.P.

Dudhawa dam was constructed to create one of the reservoirs in Mahanadi Irrigation Project, M.P. The dam comprises of a 2.9 km long and 24.7 m high earthen embankment across Mahanadi river. The earthen dam showed distress by leakages through foundation, sand boils, etc at its first impoundment in 1962. This resulted in under utilization of its full capacity. Remedial measures, such as relief wells and toe loading, were carried out as per the recommendation of the Dam safety Panel, but these measures were ineffective. In 1995 there was a serious concern about the stability of the dam because of high piezometric readings on downstream slope of the dam. Studies were undertaken to identify the cause of seepage through downstream body of dam and its effect on stability. Studies involved i) Field investigations ii) Laboratory tests to determine physical and engineering properties of soil, iii) conducting seepage and stability analysis of the dam and finally iv) recommendations (CWPRS Technical Report No. 3160).

Field investigation comprised of collection of disturbed and undisturbed soil samples along 3 sections of the dam from 9 auger boreholes of 5.0 m deep. In-situ density and moisture content of soil samples were determined. Laboratory tests such as Particle size gradation, Atterberg's Limits, Direct shear tests and permeability tests were conducted on soil samples. The test results indicated that the casing soil is classified as clayey sand (SC), while core is highly compressible clay (CH). Foundation soil consists of poorly graded sand (SC) in chainage 40-42 m, while it consists of highly compressible clay (CH) in chainage 83-84 m. The permeability values for Casing soil and sand in foundation are in the range 2.88 – 7.0 x 10^{-5} cm/sec, while for soil in core and in foundation is 1.0×10^{-8} cm/sec. The shear strength parameters, c and ø, for sand is in the range of 0.02 – 0.08 kg/cm^2 and 24.2 – 28.4 degrees respectively. In case of clay, these parameters range from 0.08 - 0.9 kg/cm^2 and 16.7-17.2 degrees.

First Seepage analysis was carried out using Finite element software (SOLVIA-TEMP), for the two sections: Ch 40-42 m and Ch 83-84 m, which were badly affected due to Leakage. The analysis used 8 noded, isoparametric elements. Boundary conditions were: Maximum water level (RL 426.7 m) on upstream, no tail water level (GL 404.3 m) on downstream and rock line was considered as impervious. Initially the analysis was carried out considering the filter zone as a freely draining zone, i.e. permeability value 1.0 x 10^{-3} cm/sec. However, the computed pressure heads on the downstream slope was at variance with the observed piezometers pressure head data. In order to study the effect of clogging in the filter due to fines, a parametric analysis were carried out considering the Filter zone as increasingly impervious, to account for clogging of filters. It was observed that the measured and computed total head values were in agreement, when the Filter zone permeability was considered as 1.0 x 10^{-5} cm/sec. The phreatic line emerged at the downstream slope of the earthen dam (Fig 3.12), as observed at site.

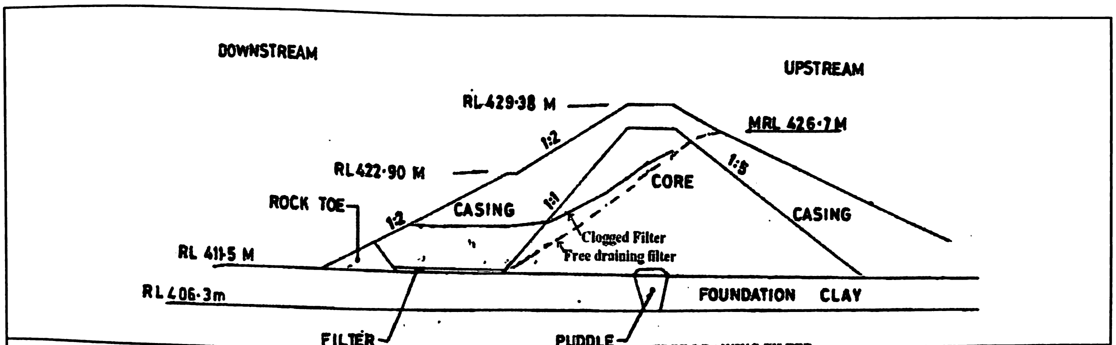


Fig.3.12: Phreatic Line for Free Draining and Chocked Filter (Ch 40-42 m)

Subsequently, a Slope stability analysis was carried out by Bishop's modified slip circle method, using the laboratory shear strength values of soil and the pressure head values computed from seepage analysis. The analysis indicated Factor of safety (FoS) of 1.26 and 1.12 for the sections at Ch 40-42 m and Ch 83-83 m respectively. Since the FoS were less than BIS requirement of 1.5, stability analysis were carried out with the rubble fill loading on the downstream slope. The analysis showed that a rubble loading of 1.5 m thick on d/s slope would give a required FoS of 1.5. The thickness of 1.5 m rubble fill was computed from the exit hydraulic gradient from seepage analysis.

Based on the analysis, recommendations were given to construct a rubble fill of 1.5m thick on the downstream slopes at Ch 40-42 m (11.0 m high) and at Ch 83-84 m (6.0 m high) and to extend it to a distance of 20 m from the toe, as shown in Fig. 3.13.

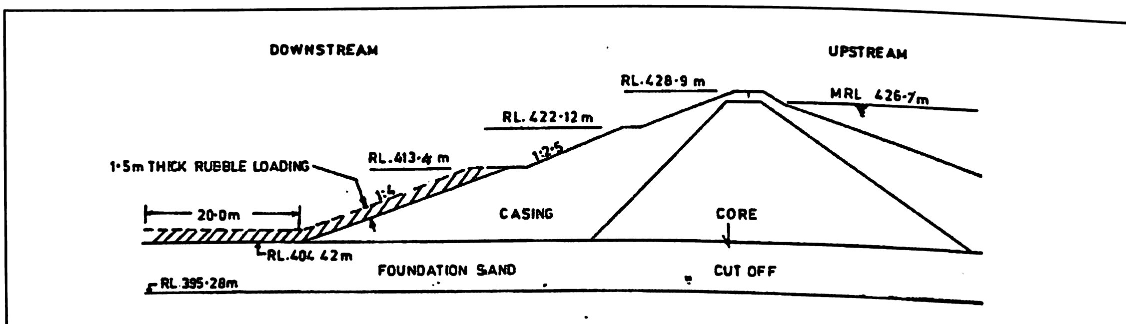


Fig 3.13 Remedial Measures Suggested for Ch 40-42 m

3.6 CONTROL MEASURES

Safety of dam is ensured by controlling seepage in dams as well as its foundations. There are two approaches for seepage control and they are: i) to reduce the quantity of seepage and ii) to provide safe outlet to seepage water. In the first case, the quantity of seepage can be reduced by providing highly impervious clay / bentonite barrier in the permeable zones. In the second case, seepage water is safely taken out by providing proper Filters and Drains, made of sand and gravels. In place of natural materials, now a days geosynthetic materials such as geotextiles and geomembranes, have come in vogue for use in seepage control measures. The application of various methods for control of seepage through earth dams and foundations are highlighted in this chapter (BIS 9429, 1999).

An earth dam generally consists of the following components to reduce the seepage: i.) Cut-off trench ii.) Impervious Core iii) Internal drainage system iv) Surface drainage. A rockfill dam consists of i) Main rockfill ii) internal core or upstream slope impervious membrane iii) Rubble cushion iv) cut-off wall. A typical earth dam with all seepage defensive measures is shown in Fig. 3.14.

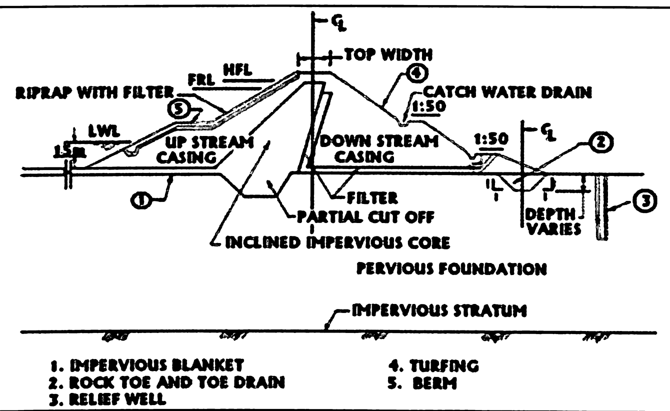


Fig 3.14 : A Typical Earth Dam With All Seepage Defensive Measures

In order to prevent the seepage water, defensive measures in the form of impervious barriers are incorporated in the design of earth / Rockfill dam design. The usual defensive measures adopted in earth dams are i) Cutoff trench , ii) Concrete diaphragm, iii) Grout curtain , iv) Sheet pile, v) Upstream Blanket vi) Downstream loading berm vii) Relief Wells, viii) Filter & Drains etc. The proper selection of particular seepage control measure is dependent on: a) Nature of foundation strata such as heterogeneity and its uncertain characteristics, b) The economic value of the water stored c) height of the dam d) damage potential due to dam break etc. These impervious barriers are described in following paragraphs.

3.6.1 Cutoff Trench

A triangular trench, dug in the permeable foundation below the dam and backfilled with highly impervious soil, is known as Cutoff trench. It is constructed to increase the path of seepage from upstream to downstream. Lengthening the path of seepage drastically decreases the potential head of the water, thereby preventing the downstream end from instability and piping. There are two types of cutoffs and they are: i) Partial cutoff and ii) Positive cutoff. Details of each are described in following paragraphs.

a) Partial Cutoff

In Partial Cutoff depth does not extend up to a full depth of pervious strata of foundation. In this case, a certain amount of under seepage will occur. It should penetrate, at least 95 % of the full depth, for appreciable reduction in seepage. They are effective when they extend down into an intermediate stratum of lower permeability. Its effectiveness decreases as the Ratio of Dam width to Penetration depth increases.

They are supplementary seepage control measures, since their efficacy is less both for reducing the rate of seepage or for reducing the pore water pressure (uplift) in the downstream. However, a partial cutoff in the form of grout curtain is effective in blocking internal erosion in the vulnerable zones like pockets of open gravel and boulders in proximity of fine grained soil.

b) Positive Cutoff

In Positive Cutoff, its depth extends up to a full depth of pervious strata of foundation. A pervious strata in foundation is normally underlain by an impervious stratum. The depth of an impervious stratum is established from boreholes and trial pits. Positive cutoff is preferred if depth of impervious stratum is not excessive. An open excavation is made during the construction of a cutoff trench. It allows the designer to see the actual natural conditions of foundation and to adjust the design accordingly. It requires little observation to ensure satisfactory performance.

In order to function efficiently, it must penetrate a short distance into impermeable stratum. The design demands that its Base width should be at least one-fourth of maximum difference between the FRL and TWL, but not less than 6 m. Its base should be wider, if the foundation soil is marginally impervious. In some cases, the gradation of backfill soil may be such that the foundation soil does not provide its protection against piping. In such cases, a filter layer between the backfill and the foundation soil is required on the downstream side of the cutoff trench (BIS 9429 (1999), BIS 8414 (1977)).

3.6.2 Case Studies on Seepage through Foundation

Foundation Seepage & Stability of Maskinala Earthen Dam, Karnataka

Maskinala dam is a composite dam having central concrete spillway flanked on either sides by earthen dams, constructed across a small seasonal stream located near village Maraldinni in Lingasugur Taluka of Raichur district, Karnataka. Total length of the dam is 890 m and maximum height of the earthen section is 19.86 m above foundation level. During excavation of foundation for right abutment of spillway, a thick sandy and gravelly strata was encountered in the foundation. Studies were carried out for delineation of this strata and its influence on seepage and stability of the earthen dam (CWPRS Technical Report No. 3585).

Subsoil investigations were carried out by drilling total 15 nos. of boreholes, out of which 7 were drilled from the top of earthen section along the C/L and remaining along upstream toe line and in reservoir area. Disturbed and undisturbed soil samples were collected. The foundation consist of strata of well graded sand (SW) of varying thickness (2.0 m to 8.0 m) followed by a strata of gravel(GW) of 2.0 m thickness, underlain by hard impervious rock. Borehole log revealed that impervious cut off had not intercepted completely the pervious gravelly strata. Foundation consist of well graded sand (SW) and its permeability value from laboratory tests is 1×10^{-5} m/s. Casing zone of dam is well graded sand (SW) which has permeability of 2.3×10^{-6} m/s. Zones such as Hearting, Cut-Off and upstream Clay blanket are high compressibility Clay with permeability values range from 1.0×10^{-8} to 3.0×10^{-8} m/s.

Steady state seepage analysis using Finite Element Method was carried out for computation of seepage quantities for Maximum water level (MWL) and to ascertain the effectiveness of cut off. Total head of 16.1 m and 0.0 m on upstream and downstream respectively was considered for the analysis. Following cases were examined:

(i) Dam section with existing cut off without upstream clay blanket

(ii) Dam section with existing cut off and impervious clay blanket on upstream extending for 50 m, 100 m and 200 m.

Computed seepage quantity for the above case (i) was 37.5 m^3/day/m length. Seepage quantities for case (ii) were found to be 27.3, 20.4 and 13.9 m^3/day/m for blanket length of 50 m, 100 m and 200 m respectively. The results show that use of the blanket is ineffective. The flow net with existing cut off and clay blanket is shown in Fig 3.15. Subsequently slope stability analysis by Bishop's slip circle method was carried out considering seepage forces. The slopes were found to be stable against slope failure with minimum Factor of safety of 1.57 and 2.10 for upstream and downstream slopes respectively.

In view of ineffectiveness of clay blanket in controlling seepage, it was recommended to provide an upstream positive cut off wall in the pervious strata and anchored deep into the impermeable rock. The cut off wall should be either a slurry trench or a diaphragm wall. Computed seepage quantity, with cut off wall in-place, was found to be 0.53 m^3/day/m, which is acceptable. Installation of piezometers and inclinometers was suggested for monitoring the performance of the earthen dam.

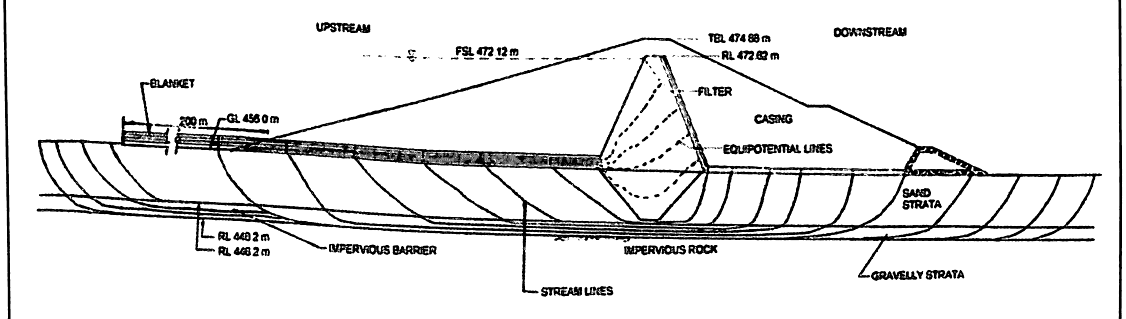


Fig.3.15 Flow net with Existing Cut Off and Clay Blanket

3.6.3 Concrete Diaphragm

The cutoff trench is an impracticable solution, when the depth of the pervious foundation strata is large (>50 m) and/or the foundation contains cobbles, boulders, cavernous limestone etc. In such situations, the concrete cutoff / diaphragm wall is an effective method for control of under-seepage. A single or a double diaphragm may be used for seepage control.

Though it is an attractive solution for seepage control, it has the risk of rupture and buckling. Hence, it has to be numerically analysed for its deformation behaviour to ascertain the extent of rupture due to relative movements of upstream and down-stream and also buckling due to down drag. The efficient seepage control demands constructing of two lines of diaphragms and also grouting the alluvium contained between them. This minimizes the uncertainty regarding i) the discontinuities created during the placement of concrete in a slurry trench and ii) defects formed in the joints. Grouting of the pervious soil within the two diaphragms could be carried out effectively because of the confinement.

3.6.4 Grout Curtain

Grout curtains are produced by injecting slurry of cement, clay, chemicals in the voids of the sediments within region assigned to the cutoff. An essential feature of all grouting procedure is successive injection, of progressively finer pockets of deposit. As much as grout cannot be made to penetrate the finer materials as long as more pervious pockets are available. The coarser materials are treated first, usually with the less expensive and thicker grouts, whereupon the finer portions are penetrated with less viscous fluids.

Effective Grout Curtain

The success of grouting is achieved on following factors (i) Hole spacing (ii) Suitability of hole inclination to intercept significant open jointing. (iii) Depth and hence the pressure to be applied. (iv) Appropriate grouting technology to suit the site conditions.

i) Hole Spacing

In order to form grout curtains, the boreholes are arranged in one or more rows parallel to each other. Borehole spacing depends on the effective radius of grouting. The typical borehole spacing in rock is 1 to 3 m. Typical row spacing is 1.5 to 2.5 m for the deep grout curtain and 2 to 4 m for short boreholes serving to consolidate and to seal the uppermost zone of the foundation. Split spacing, or closure, methods are proceeded with, through primary, secondary, tertiary sequences, with each sequence having the spacing of the previous one, until water tests in the grout holes, carried out before grouting.

ii) Suitability of Hole Inclination to Intercept Significant Open Jointing

Many times boreholes of neighboring rows are provided as offset to each other. Boreholes of the rows may be inclined across each other. This arrangement is favourable to seal sets of joints, which strike parallel to the valley and dip both sides 80°-90° (sub vertical). The orientation and inclination of the joints lead to irregular drilling arrangement. Different directions may be found in one individual project. For instance boreholes normal to the slopes of the abutment to cover joints developed by stress release, inclined boreholes in the valley to cover joints striking parallel to the valley, vertical boreholes to cover joints dipping sub horizontal etc.

iii) Depth and Pressure to be applied

The pressures at which the grout mix is injected is an important aspect and is to be based on geological formations, structural conditions of the rock mass and results of experimental grouting with uplift gauges installed. The 'rule of thumb' often adopted for grouting pressure is 0.25 Kg/cm^2 per meter depth, but this must be used with caution. Pressure should not be increased arbitrarily in an endeavor to force grout into cracks. Allowable pressure increase with each depth increases of the stage. The orientation of the bedding of fractures should also be considered in determining allowable pressures. Where uplift is the prime consideration, much lower pressures must be used in case the joints/bedding is horizontal. Each grout application is commenced at low pressures for the first five minutes and then over the next twenty-five minutes the pressure is gradually increased to the maximum.

iv) Appropriate Grouting Technology to Suit the Site Conditions

The filling up of existing voids in the substrata leads to stabilization or sealing as far as the grouted material becomes hard. The procedure is more effective the more complete void filling is achieved. The complete filling of the existing voids is made possible if the voids are connected to each other. Commonly, this applies to natural rock and soil. So, access to a system of voids is possible through widely spaced boreholes in the sense that the spacing exceeds by far the width of voids. The grouting material is kept flowing by the grout pressure until the final effective radius is reached. In view of disturbances in the grouting zone the effective radius must be limited. For the same reasons the grout pressure must always be limited. Inadequate pressures may lead to heaving of the surface and to excess stresses on embedded structures. During the procedure of grouting the material must neither start hardening nor block further penetration by sedimentation of solids.

3.6.5 Grouting Procedure

i) Grouting Materials and their Mixing

Pure cement grouts are unstable. The addition of suitable quantity of bentonite or clay to the cement grout transforms these unstable grouts into stable grouts. The various properties of grout materials used for injection should be assessed and recorded. According to Houlsby, A.C.(1982), normal Portland cement is used for foundation grouting. In foundation grouting there is no need for use of finer cement; penetration in finer cracks with normal cement can equally be good as with finer cement, when normal cement is mixed at high speed and injected with appropriate methods.

Grout should be mixed in machine operating at 1500 revolutions per minute or faster; the high-speed product has greater durability. Grout mixtures ranging from a very thin mixture of 20:1 to thick mixtures of 0.5:1 (by weight) have been used. Usually the range of mixtures falls from 5:1 to 0.8:1. It is only in exceptional circumstances that mixers leaner than 10:1 need be used. The choice of grout mixtures may be based on results of percolation tests conducted prior to grouting. According to Houlsby,A.C.(1982) "Water cement ratios for grout mixes may vary widely depending on the permeability of the foundation rock. Starting water-cement ratios usually range from 8:1 to 5:1 by volume. Most foundations have an optimum mix that can be injected which should be determined by trial in the field by gradually thickening the starting mix. An admixture such as sand or clay may be added, if large voids are encountered".

ii) Construction Aspects

Grouting operations may be performed from the surface of the excavated foundation, from the upstream fillet of dam, from the top of concrete/masonry/earth work placements for the dam, from galleries within the dam or any combination of these locations. Sometimes GI pipes left from the foundation level in masonry or concrete which is raised and the drilling and grouting is made through these pipes. However, it may be pointed out that if rock conditions permit, the grouting should be done preferably before concrete or masonry was laid, to avoid deformations of concrete or masonry due to upheaval. Whereas, for curtain grouting substantial load should be built up so that higher pressure could be used to ensure effective grouting.

3.6.6 Grouting Methods

The depth to which the holes are drilled will vary greatly with the characteristics of the foundation and the hydrostatic head. In a hard dense foundation, the depth may vary from 30 to 40 percent of head. In a poor foundation the holes will be deeper and may reach as deep as 70 percent of head. During the progress of the grouting, local conditions may determine the actual or final depth of grouting.

i) Stage Grouting

For stage grouting, the methods usually comprise of 'downstage without packer' and 'downstage with packer' and 'upstage'. Of these, the preferred method, if the high standard grouting is required, is the downstage without packer. However, this is not the cheapest method and may not always be warranted. The advantage of downstage grouting is that upper vulnerable parts receive multiple treatments. 'Upstage' or ascending stage method is relatively rapid and the cheap method but may be impracticable, if holes collapse readily or have walls so rough that the packer cannot be seated in them at required depths. It is also unsuited to difficult conditions, particularly where grout might travel upwards and enters the hole above the packer and/or might cause movement of surface rock. This is also known as 'packer grouting' method.

ii) Multiple Row Curtain

Multiple row curtains commonly have 3 to 5 rows to enable outer rows to be grouted first and then the inner ones. The innermost row is usually taken to greater depths.

3.6.7 Grout Curtain in Different Geotechnical Conditions

i) Grout Curtain in Pervious Soils

Grout cutoffs are generally effective when seepage occurs primarily through pockets, zones or layers of coarse materials (gravel, boulders and talus). Coarse sediments with in-situ permeability of 10^{-1} cm/s can generally be treated effectively with low cost grouts, namely, clay cement grout. The response of soils with permeability of the order of 10^{-2} cm/s and lower, grouting is uncertain. Silicate grouting has been effective in some cases for soils of initial permeability ranging from 10^{-2} to 10^{-3} cm/s. Close spacing of holes and expensive chemical grouts are required for grouting of soils of initial permeability of 10^{-3} cm/s and lower.

Most chemical grouts are extremely compressible; and some of them are vulnerable to leaching. Grout curtains with chemical grout that can withstand seepage pressure over a long period of time are relatively expensive. In the interest of safety and economy, it is generally advisable to adopt multiple row grouting. The outer rows are treated with cement – clay – bentonite grouts, so that in the inner rows expensive grout treatment starting with stable grout mixes and ending in chemical mixes can be adopted, which not only forms an effective grout curtain also resist erosion and leaching. The width of grout curtains should be adequate to contain stray pockets of fine sand, which is not groutable and to ensure that the gradient across the grout curtain is low enough to ensure that the grout injected is not eroded or leached by seepage. To provide the second line of defense blanket and relief wells are also provided with grout curtains. A typical grout curtain in pervious soils is shown in Fig.3.16.

ii) Grout Curtain in Rock

The installation of grout curtains in the abutments and foundations of the dams result in significant improvements only if these curtains are tied into more impervious rock at a reasonable depths. It will lengthen the seepage path and offer increased resistance to seepage, which sometimes may not essentially alter, the seepage quantity. The primary function of the grout curtains is to intercept and fill water passages such as, solution channels & fissures; they seldom provide an impervious barrier. Wherever the joint filler contains erodible material, a wide, and multiple line curtains should be employed in all cases regardless of the grout takes. Grouting will not be much effective if take of cement falls below 30 Kg/m. In these cases the grouted area will correspond to 2 to 8 times of the borehole size. So to achieve effective grouting the spacing of grout holes is to be reduced and 5 rows of boreholes are to be grouted. The purpose of a reasonable borehole arrangement is to achieve a coherent grouting zone. The establishment of a wide grouting zone also minimizes the risk of insufficient effectiveness.

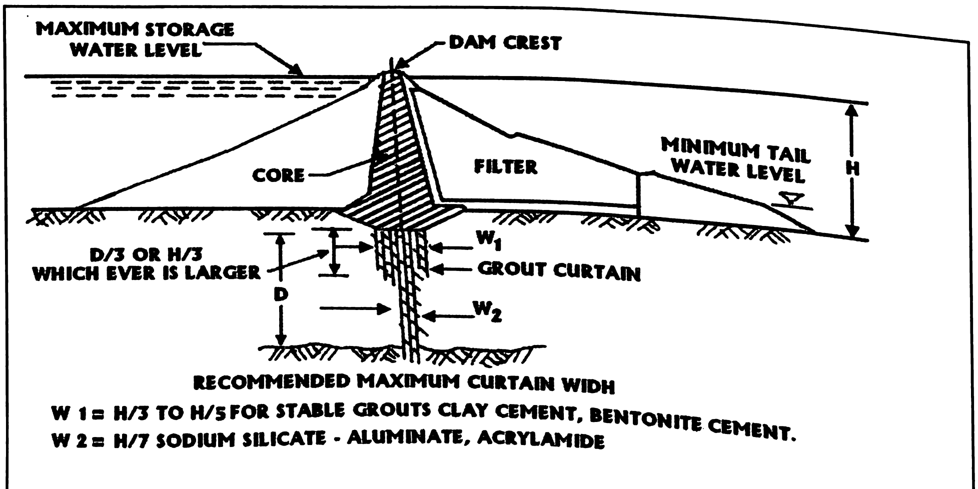


Fig.3.16 Grout Curtains in Pervious Soils

The standard positions for grout curtains are at the central line of core or slightly upstream of it for earthen dam and near the upstream heel for concrete/masonry-dams. The grout curtain is usually inclined upstream to get as clear of the drainage holes as feasible. This means that the grout holes frequently need be doubly inclined; with one of the directions of the inclination suiting this and other direction suitably intercepting optimum joints, bedding planes etc. The grout curtain is often presumed to be the location at which uplift pressure reduction commences. The Typical profile of grout curtain holes is shown in Fig. 3.17.

Tentative designs will usually specify a single line of holes drilled at 3 m centre to centre, although wider or closer spacing may be required depending on the rock condition. To permit application of high pressures without causing displacement in the rock or loss of grout through surface cracks, curtain grouting is carried out subsequent to consolidation grouting and after some of the concrete/masonry has been placed. Usually, it is done from galleries within the dam and from tunnels driven into the abutments especially for this purpose.

To facilitate drilling, 50-75 mm dia. pipes are embedded in the masonry from foundation to the floor of gallery. When the structure has reached an elevation that is sufficient to prevent upheaval, the grout holes are drilled through these pipes and into the foundation rocks. Although the tentative grouting plan may indicate holes to be drilled on 3 m centre to centre, the usual procedure will be first drill and grout holes approximately 12 m apart, or as far apart as necessary to prevent grout from on hole looking into another drilled but un grouted hole. Intermediate holes, located midway between the first holes, will then be drilled and grouted. Drilling and grouting of additional intermediate holes, splitting the spaces between completed holes, will continue until the desired spacing is reached or until the amount of grout accepted by the last group of intermediate holes indicates no further grouting is necessary.

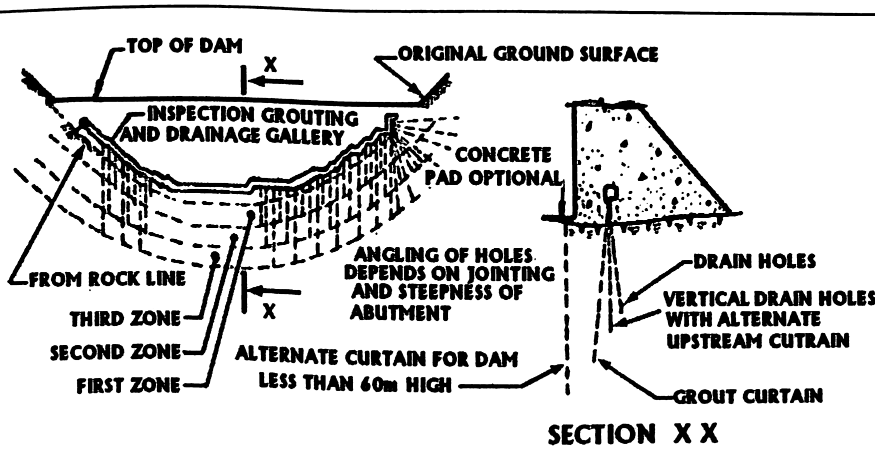


Fig. 3.17 Typical Profile of Grout Curtain Holes

3.6.8 Case Study on Seepage Control Measures

Seepage Control Measures to Prevent Water Pollution by Fly Ash, Korba, M.P.

National Thermal Power Corporation (NTPC) owns and operates Korba Super Thermal Power plant of 2100 MW installed capacity, near Champa in Bilaspur district, Madhya Pradesh. Approximately 3.5 millions tons of coal ash was produced every year which needs to be processed and disposed in the form of slurry through pipelines to the disposal lagoons. NTPC had demarcated about 6 sq.km of area near the Korba Plant for dumping the ash. Lagoons were to be created by constructing earthen / rockfill dykes along the periphery of the disposal area. CWPRS carried out studies for seepage and stability of the rockfill dyke (CWPRS Technical Report No. 3429).

The Foundation geotechnical investigation revealed the presence of highly weathered granite rock mass up to a depth varying from 1.5 m to 8.0 m from ground level. Fig.3.18 shows the section of Rockfill dyke along with the permeability values of the different zones of the dyke section.

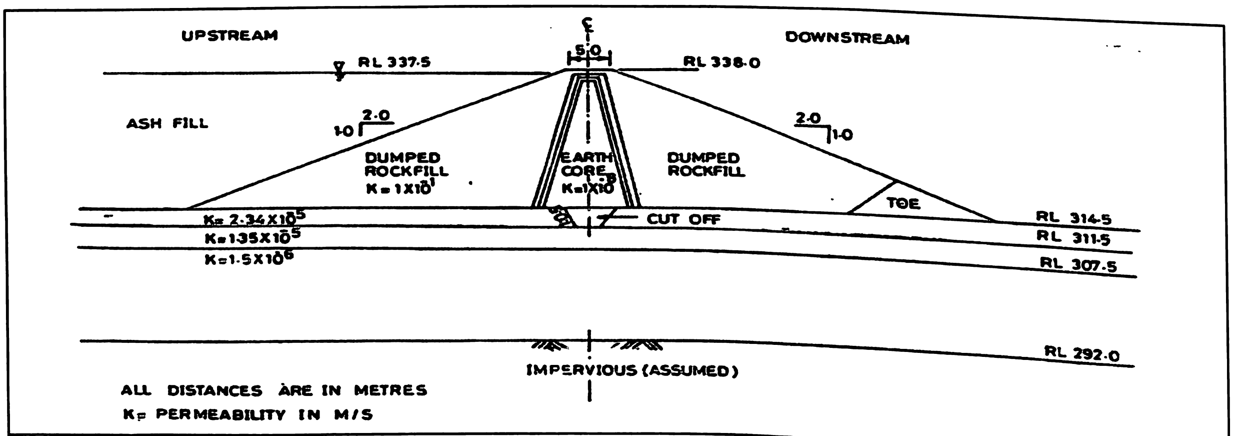


Fig.3.18: Rockfill Section with Foundation showing Permeability Values for Seepage Analysis

Estimation of seepage through the foundation and from the body of the rockfill dyke was carried out in view of possible pollution that would occur due to seepage in the adjacent river. Seepage computations were performed using finite element software SOLVIA-TEMP. It was assumed that standing water (to represent saturated fly ash) on the upstream would cause seepage. Permeability of the foundation was taken from the bore hole data. Permeability of rockfill and earthen core were taken from literature.

Seepage analysis estimated seepage discharge quantity of 8.76 cum/day /m length which was considered to be on higher side. In order to reduce seepage quantity through foundation, seepage control measures in the form of grout curtain was adopted. Grout curtain with permeability value 5

Lugeon gives seepage discharge quantity of 4.07 cum/day/m length. More impermeable grout curtain with permeability value 3 Lugeon gives seepage discharge quantity 3.12 cum/day/m length. However, seepage control measure in the form of grout with permeability value 3 Lugeon as well as impervious blanket gives seepage discharge quantity 1.30 cum/day /m length.

It was recommended to implement seepage control measures in the form of impermeable grout along with upstream clay blanket, only if quality of seepage water did not satisfy pollution control requirements. It was also suggested to construct suitable relief wells or observation wells at the toe of the dyke for collecting and checking the quality of the water.

3.7 SEEPAGE CONTROL MEASURES

3.7.1 Sheet Piles

Sheet piles are useful as barrier to arrest internal erosion is a sandy and silty stratum. However, they have been proved to be rather ineffective as a positive means of controlling seepage through pervious deposits. Even if sheet pile cutoffs are intact they are not watertight because of leakage across the interlocks. In addition, the locks may break because of defects in the steel or when a pile hits an obstacle. Once the lock is split, the width of the gap increases rapidly with increasing depth and may assume dimensions of a few metres. If steel sheet piles are driven to hard rock with a very uneven surface, a continuous row of triangular gaps may be present between their lower edges and the rock, or the piles may curl if they are driven too hard.

It appears difficult to justify the use of sheet piling as a means of controlling seepage, particularly when other less expensive means are available which provide the same, if not more positive results. Some methods may be used to improve the operating characteristics of sheet pile cutoffs. These include using vibrating pile driving hammers to reduce the probability of driving out of interlock and the use of bentonite mud to seal the interlocks. However, until such time that these techniques are perfected and become routine, sheet pile walls should be considered no more effective than partial cutoffs.

In barrages, the efficacy of sheet piles is primarily due to their ability to prevent excessive exit gradients in downstream zones vulnerable to scour. Sheet piles may also be effective in blocking the path of direct seepage at the contact of alluvium and rigid concrete floors of barrages. They also serve as an interception device against internal migration of soil particles.

3.7.2 Upstream Impervious Blanket

When the investigations do not provide definite indications of the depth and continuity of the impervious stratum or the depth of cutoff is excessive, consideration should be given to use of an impervious blanket. Blankets of adequate length in conjunction with relief wells or filter trenches have been successfully used. Adoption of a blanket-cum-relief well system imposes the obligation of maintaining continuous observations and exercising adequate control in installation of the drainage system.

If a positive cutoff is not required, or is too costly, an upstream impervious blanket combined with relief wells in the downstream section may be used (Fig.3.19). Filter trenches supplement relief wells in heterogeneous deposits and in zones of seepage concentrations. An upstream blanket may result in major project economies, particularly if the only alternative consists of deep grout curtains or concrete cutoff walls. Since a surface layer of relatively impervious soils often overlies alluvial deposits in river valleys, it is advantageous, if this natural impervious blanket can be incorporated into the overall scheme of seepage control.

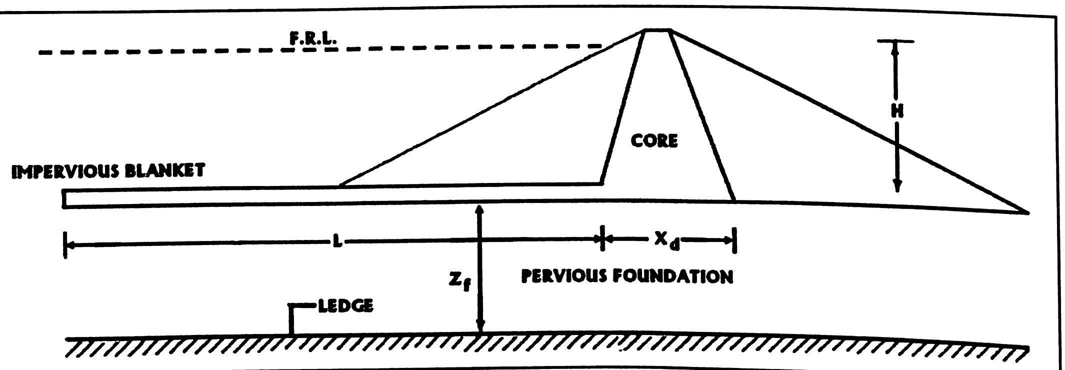


Fig.3.19 Upstream Impervious Blanket

Following basic requirements should be satisfied while selecting the length and thickness of the blanket.

a) Reduction of the quantity of under-seepage to the desired extent.

b) Limiting the exit gradients to the allowable limits for the substrata encountered.

The allowable seepage depends on economic considerations; therefore design decisions are governed by the estimation of seepage. This is in turn dependent on the degree of precision achieved in determination of permeability. It is advisable to check the permeability values measured in tests conducted in bore holes.

A blanket length of about 5 times the head, combined with relief wells and drainage trenches, can generally achieve effective control of exit gradients. A longer length of blanket is generally required for control of subsurface erosion and for reducing seepage to desirable limits. It should, however, be noted that there is a limit to the length of the blanket beyond which it may not be useful.

When past experience is inadequate or knowledge of geology indicates possible hazard of open zones in proximity with soils vulnerable to subsurface erosion, supplementary measures of seepage control shall be provided along with the blankets and relief wells. While selecting the length of the blanket the progressive reduction in efficacy of the increments to the blanket length especially when the blanket length is large relative to thickness should be considered.

3.7.3 Downstream Loading Berm

When a complete cutoff is not required or is too costly, and it is not feasible to construct an upstream impervious blanket, a downstream loading berm may be used to reduce uplift pressures in the pervious foundation underlying an impervious top stratum at the downstream toe of the dam. Other downstream under seepage control measures (relief wells or toe trench drains) are generally required with downstream seepage berms. Downstream seepage berms can be used to control under seepage efficiently where the downstream top stratum is relatively thin and uniform or where no top stratum is present, but they are not efficient where the top stratum is relatively thick and high uplift pressures develop. Downstream seepage berms may vary in type from impervious to completely free draining. The selection of the type of downstream seepage berm to use is based upon the availability of borrow materials and relative cost of each type.

3.7.4 Relief Wells

Relief wells are an important adjunct to most of the preceding basic schemes for seepage control. They are used not only in nearly all cases with upstream impervious blankets, but also along with other schemes. This is to provide an additional assurance that excess hydrostatic pressures do not develop in the downstream portion of the dam, which could lead to piping. They also reduce the quantity of uncontrolled seepage flowing downstream of the dam and, hence, they control to some extent the occurrence and/or discharge of springs. Relief wells should be extended deep enough into the foundation so that the effects of minor geological details on performance are minimized. It is necessary to note the importance of continuous observation and maintenance of relief wells, if they are essential to the overall system of seepage control. A typical relief well is given at Fig.3.20 (BIS 5050:1968)

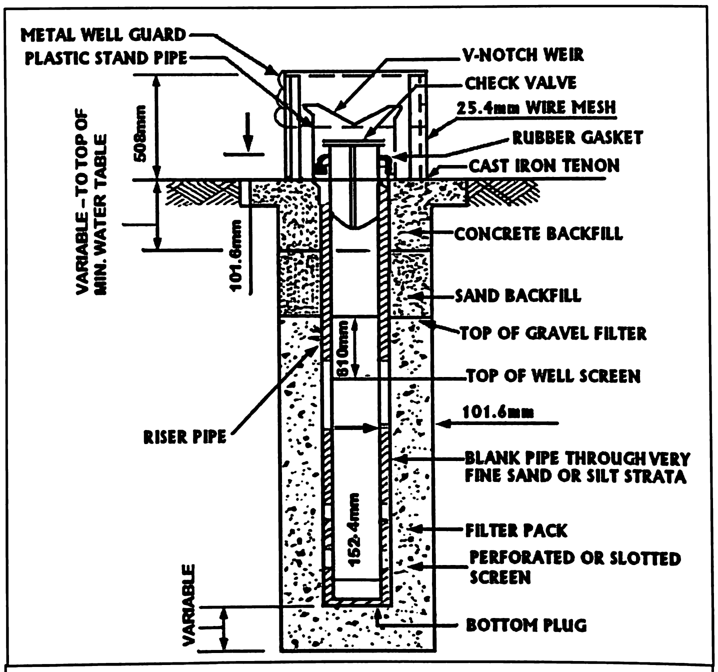


Fig 3.20 : Typical Relief Well

3.7.4.1 Limitation of Blankets and Relief Wells

Sometimes unfavorable site conditions make it difficult and expensive to place the blanket properly, to ensure its continuity and to protect it from erosion. The following considerations would influence the blanket layout and costs:

i) Presence of a deep pool in the river-requiring placement of blanket under water, dewatering of a large area after extensive coffer-damming operations.

ii) River diversion layout and schedule requiring construction of blanket in sections. This makes it difficult to ensure satisfactory junctions of various sections of the blanket especially in the zone of intersection of the blanket with the diversion cut.

iii) Unfavorable topography and geological features, such as abrupt steps in the hill sides, presence of talus and other pervious deposits of large extent on the abutment and flanks.

iv) Possibility of erosion of the blanket by high velocity flow near entry and exit zones of the diversion cut or tunnel.

The discharge from the relief wells may decrease with passage of time for one of several reasons: the reservoir may be silting up; the wells may be plugging with silt; or chemical deposits or products of corrosion may be obstructing the well screens. If the decrease in the discharge is due to silting of the reservoir, the water levels in the observation wells at full reservoir go down; in all other circumstances they go up. Sealing off any silt layers or lenses should prevent excessive discharge of silt during installation of the wells. Minor accumulations of silt should be flushed out periodically.

For this reason, and to permit replacement of deteriorated screens, the heads of the wells should be readily accessible.

3.7.5 Filters and Drains

The seepage control in hydraulic structures is also carried out using drainage methods. Filters and drains are provided in this structure to facilitate safe and quick seepage of water. Fig. 3.21 shows typical drains usually provided in earthen dams.

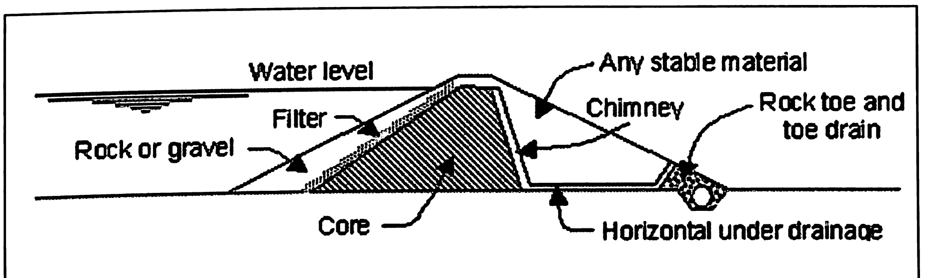


Fig 3.21 Typical Drain in an Earth Dam

Properly designed filters and drains are essential for safety of hydraulic structures to prevent damaging action of water in their foundation or other supporting soil / rock formations.

3.7.5.1 Filters

A filter should prevent excessive migration of soil particles, while at the same time allow liquid to flow freely through the filter layer. Filter is therefore summarized by two seemingly conflicting requirements,

1. It must retain soil implying that the size of filter pore space / opening should be smaller than a specified maximum value. In other words, its voids should not permit the migration of the particles from the protected zone (piping criteria).
2. The filter must be permeable enough to allow a relatively free flow through it, implying that the size of filter pore spaces and number of opening should be larger than a specified minimum value. In other words, it should be sufficiently more pervious than the protected zone to induce a sharp reduction in hydraulic gradient.

The function of filter is to retain the particles of a drained soil while allowing water to pass into a relatively free draining coarser zone. Various criteria for the filter are explained as below:

i) Piping Criteria

To prevent migration of erodible soil and rocks into or through filter, Terzaghi suggested following criterion for design of filter against piping,

$$\frac{D_{15}(of\ filter)}{D_{85}(of\ protected\ soil)} \leq 4$$

Where,

D_{15} = particle size, for which 15% by weight of particles are smaller.

D_{85} = particle size, for which 85% by weight of particles are smaller.

ii) Permeability Criteria

Filters are supposed to be more permeable than the base soil to be protected because they serve as a transition between drained soils and drains. To satisfy this requirement, filters and drains are designed by applying following criteria.

$$\frac{D_{15}(of\ filter)}{D_{15}(of\ protected\ soil)} \geq 5$$

Thus 15% size (D_{15}) of a filter material should be at least five times the 15% size of the protected soil but not less than 0.1 mm. This criterion ensures that filters will be about 20 to 25 times more permeable than protected soils.

iii) Internal Stability Criteria

The internal stability, also known as auto stability means the particles of the material must not migrate within the same material. Internally unstable soils cannot function as filters because they lose their fine particles due to seepage.

In general, soils having wider grading and higher uniformity coefficient ($C_u = D_{60}/D_{10}$) are internally unstable. US Bureau of Reclamation and US Corps of Engineers have recommended the criterion for filters as $D_5 > 0.074$ mm. In some cases, this may require washing of sand and sand-gravel mixtures. If the protected base soil is internally unstable due to high silt content, the piping criteria should be verified with the silt removed.

iv) Protection against Segregation

Segregation of filter materials during transportation, placing and spreading changes its grain size distribution in isolated zones and in contact with the protected soil. This changes the piping conditions at the contact and the auto stability of the filter itself. Segregation can be reduced to safe limits by using filter material of lower uniformity coefficient. Transporting and spreading the material at moisture content of 5% and being careful not to drop it from any significant height may further reduce segregation. It is advisable for the filter to have $C_u < 20$ in order to prevent segregation.

v) Non-Cohesive Filter

In order to prevent propagation of embankment cracks in the filter, non-cohesive soils are used as a filter material. The non-cohesiveness of filter material is tested by keeping a compacted sample in a tray and then carefully flooded. If the material slumps to its angle of repose with the rise of the water level, the material can be considered as non-cohesive.

vi) Graded Filters

If a considerable thickness of filter material is required to meet the discharge requirements, only a small portion of this total thickness is necessary to prevent erosion and clogging. In such cases, reduction in the thickness of filters, particularly the horizontal filters, consisting of fine filter material, is possible by using a graded filter consisting of internal layer of coarse material that has a higher permeability than the portion of fine filter. To provide a given discharge capacity, the required thickness of coarse layer in filter varies inversely with the square root of its coefficient of permeability. For design of these filters, Indian standard code of practice BIS 9429-1999, 'drainage system for earth and rockfill dams', can be used.

3.7.5.2 Drains

The main function of drains is to provide a controlled discharge of water from within the body of the dam. A drain and its outlet should provide channel flow conditions. Material used in drains should normally be 200 to 1000 times more permeable than the drained soil. Fine to coarse sized gravel (4.75 mm to 80 mm) is an ideal material and in most applications. In tailings dams, fine gravel (4.75 mm to 20 mm) is suitable.

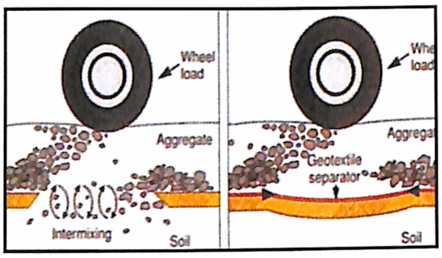
Installing perforated or slotted pipes can provide a large drainage capacity. Granular material surrounding slotted or perforated pipe should meet the following criteria.

3.8 GEOSYNTHETICS

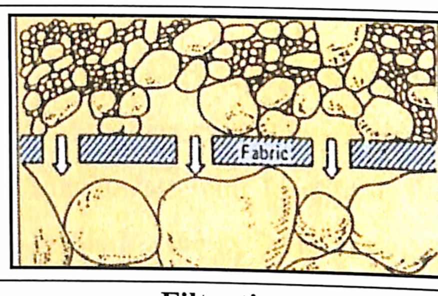
Geosynthetic sheets are man-made materials used in various geotechnical applications to enhance functioning of the structures. They are typically made from petrochemical-based polymers mainly Polypropylene, Polyethylene, Polyester, Polyvinyl chloride, Elastomers and Polyamides. They play a significant role in providing cost effective and viable solutions to seepage losses in hydraulic structures.

3.8.1 Functions of Geosynthetics

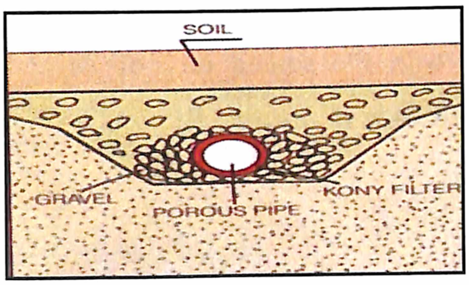
The primary function of a geosynthetic material when used in seepage control applications can be one of the following: (i) Separation, (ii) Filtration, (iii) Drainage, (iv) Fluid barrier or (v) Protection (Fig.3.22). In some cases the geosynthetic may also serve dual functions of reinforcement and seepage control. They are termed as geocomposite.



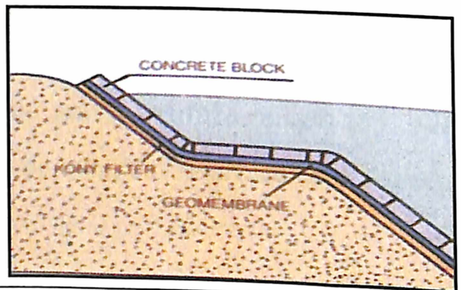
Separation



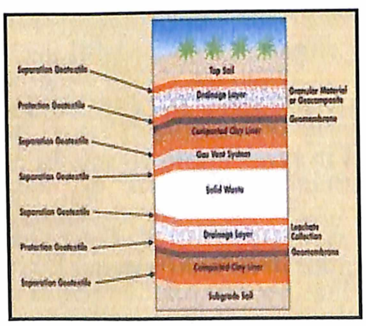
Filtration



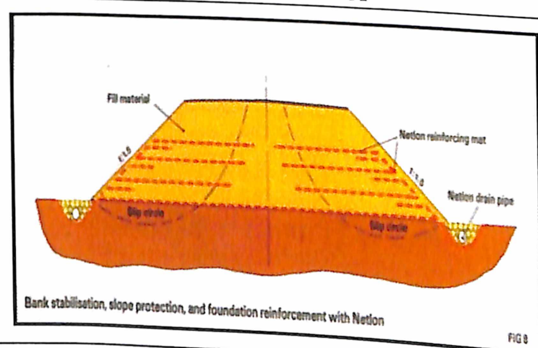
Drainage



Moisture Barrier



Protection



Reinforcement

Fig.3.22 - Various Functions of Geosynthetics

3.8.2 Types of Geosynthetics

A wide variety of products come under the generic term "Geosynthetics". These products are i) Geotextiles ii) Geogrids iii) Geomembranes iv) Geosynthetic clay liners v) Geocells vi) Geofoam vii) Geocomposites

i) Geotextiles - Geotextiles, as the name indicates, are flexible, textile-like fabrics. Geotextiles, when used in Civil Engineering structures, provide one of the following functions: separation, filtration, reinforcement or drainage. Geotextiles can be of different types, based on their manufacturing process. Non-woven geotextiles are made using synthetics filaments or fibers that are continuously extruded and spun, blown or placed on a moving belt. These items are then needle punched or heat bonded into a nonwoven mass. Woven fabrics are individual threads (monofilaments, multifilaments or fibrillated yarns) or slit films and tapes that are actually woven on a loom. Woven fabrics exhibit high tensile strength, high modulus, and low strains; while nonwoven fabrics have high permeability and high strain characteristics. Geotextiles are manufactured in a variety of geometric and polymeric compositions to meet a number of different applications.

ii) Geogrids - Geogrids are polymer grid-like sheets, consisting of integrally connected elements, with large apertures. These are used primarily for reinforcement of unstable soils in embankments, soil retaining walls, subgrade stabilization, embankment base reinforcement, etc. The apertures allow the soil to fill the space between the elements, thereby increasing soil interaction with the geogrid.

iii) Geomembranes - Geomembranes are impermeable polymeric sheets used as hydraulic barriers. Geomembranes are widely used to prevent contamination of valuable water resources and fluid migration to sub surface soil and ground water. Geomembranes made from Polymeric materials. They replace conventional liner materials such as concrete, clays, soils, etc.

iv) Geosynthetic clay liners - Geosynthetic clay liner (GCL) is a composite product comprising of a bentonite clay layer sandwiched between two layers of Nonwoven geotextile. The bentonite clay, on coming in contact with water, swells forming an impervious barrier to fluids. GCLs are extensively used for seepage control in hydraulic structures.

v) Geocells - Geocells are made up of HDPE flats which are welded together and which can be stretched out and folded. The cells on stretching are filled with stones or concrete forming a grid framework. Geocells are used to improve the bearing capacity of foundations on soft soils.

vi) Geofoam- Geofoam is a product consisting of many closed, but gas-filled, cells. Geofoam is used as a super lightweight fill, which makes it a viable option for landslide repair, and for embankments on soft, compressible deposits. Geofoam is also used for thermal insulation of pavements and foundations.

vii) Geocomposites - Geocomposites are hybrid systems of any of the above geosynthetic types, which can function as specifically, designed for use. Geocomposites include geonets, pavement edge drains, sheet (wall) drains, wick (strip) drains, used to expedite drainage. The core material could be High-density polyethylene (HDPE), polypropylene, polyvinyl chloride (PVC), high impact polystyrene or a combination of two polymers. Geonets, a type of geocomposite, are stiff polymer net-like sheets, which serve as horizontal drainage mats. Geonets are with in-plane openings used primarily as a drainage material in conjunction with soil mass. Different geosynthetic products are shown in Fig.3.23.

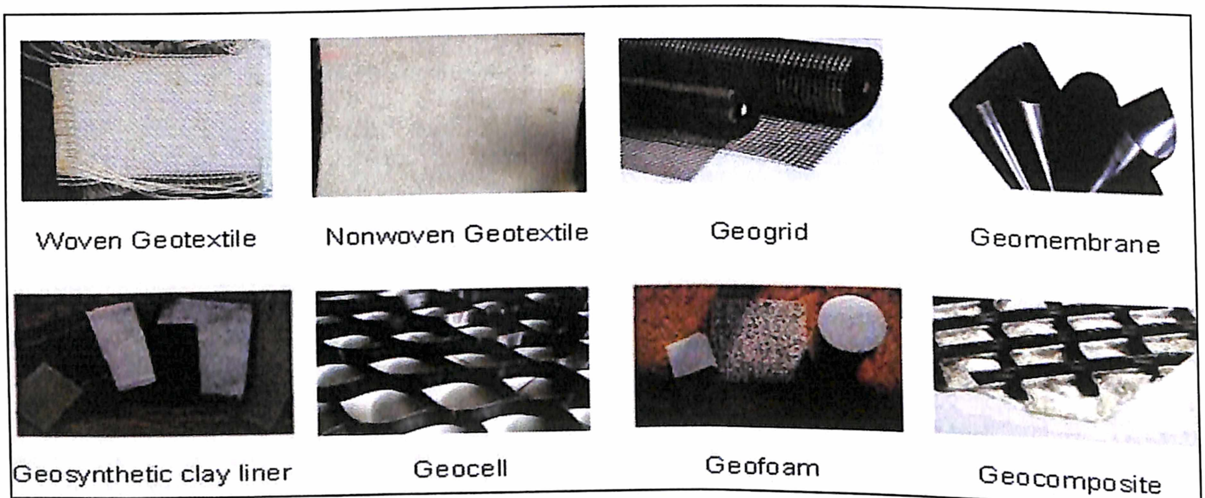


Fig. 3.23 Various Geosynthetic Products

3.8.3 Properties of Geosynthetics

Selection of geosynthetic material for a particular application necessarily depends on suitable properties of the geosynthetic used. If these properties are excessive, an uneconomical solution would be derived. On the other hand, if these properties are inadequate, the solution may cause a failure. As such, evaluation of properties of geosynthetics based on laboratory tests should be resorted to. Broadly, the geosynthetic properties can be classified into five different categories viz. (i) Physical, (ii) Mechanical, (iii) Hydraulic, (iv)Environmental and (v) Endurance. The properties corresponding to each of these categories for Geotextiles and Geomembranes are indicated in Table 1. (Koerner)

Table 1. Significant Properties of Geotextiles and Geomembranes

Category	Geotextiles	Geomembranes
Physical	Thickness Specific gravity Mass per unit area	Thickness Specific gravity Mass per unit area Water vapour transmission
Mechanical	Tensile strength & Elongation Compressibility Fatigue strength Burst strength Tear resistance (Trapezoidal tear / Tongue tear) Impact resistance Puncture resistance Interface friction with soil	Tensile strength & Elongation Modulus of Elasticity Tear resistance Impact resistance Puncture resistance Interface friction with soil Seam strength
Hydraulic	Percent open area Apparent opening size Permittivity(cross-plane permeability) Transmissivity(in-plane permeability) Soil retention test Clogging potential	--
Environmental	Resistance to chemicals Resistance to temperature Resistance to sunlight / UV / weather Resistance to bacteria Resistance to burial deterioration	Chemical compatibility Resistance to temperature Resistance to sunlight / UV / weather Resistance to bacteria Resistance to burial deterioration
Endurance	Creep Abrasion Clogging	Water absorption Aging

3.8.4 Applications of Geosynthetics in Embankment Dams as:

a) Filters and Drains

The geosynthetic industry, during the last four decades, has developed a wide range of materials that are useful in development of irrigation and hydraulic projects, especially for controlling seepage and erosion. Use of geosynthetics provides long-term solutions and benefits when used for seepage control and drainage applications. With the advancement in technology and manufacturing processes, it is now possible to design and manufacture a specific product to suit a particular application. The applications of geosynthetics, as a substitute to conventional filters / drains and for seepage control in earth and rock fill dams, is possible and can be executed as seepage control measures. (Zornberg & Weber, 2003)

A homogeneous earth or rockfill dam under steady seepage conditions will develop a zone of saturation, which will emerge on the downstream slope, resulting in a subsequent loss of stability. The use of high permeability soil is not acceptable since the reservoir losses will be to high. The solution to this problem is to create a zoned embankment with adequate drains, filters and cutoff trench.

The function of drains is to draw the zone of saturation away from the downstream face of the embankment. Geosynthetics have been used as a substitute or in combination with natural drains in dams. A number of situations exist for potential use of geotextiles in earth dams for drainage applications. The dashed lines in the Fig. 3.24 show various locations of geotextiles used in zoned earth embankments.

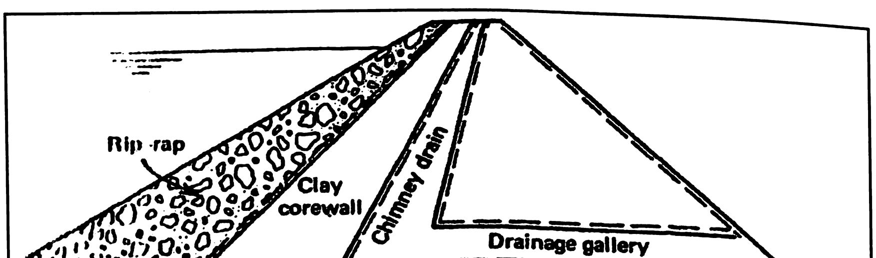


Fig.3.24 Locations of Geotextiles in Zoned Earth

For drainage applications Geocomposites can also be extensively used. The compressibility of geocomposites is low, as such they are suitable when drainage under high overburden pressure is required. In the case of embankment dams that exhibit seepage through their downstream slope, the construction of a drainage system in the downstream zone is required. A solution consists of using a geocomposite drain (GCD) placed on the entire downstream slope or only on the lower portion of it and covered with backfill. The GCD must be connected with the toe of the dam with outlet pipes or with a drainage blanket.

Migration of soil particles causing piping or erosion failure is controlled in earthen dams using filters. Geotextiles have been used worldwide as filters in various locations within embankment dams, both for new construction and rehabilitation purposes. The main use has been a replacement or a supplement for granular filters. The first application of a geotextile filter in an embankment dam was in 1970 at Valcros dam in France, 17 m high (Giroud and Gross, 1993). Polyester nonwoven geotextile filters were used both around the downstream gravel drain and also under the rip-rap, protecting the upper portion of the upstream slope.

b) Seepage Control

Seepage through dams is the most common cause of deterioration and structural damage. The hydraulic conductivity of geomembranes (10^{-15} m/s) being several orders of magnitude below that of low permeability clays (10^{-9} m/s), geomembranes have been used to rehabilitate embankment dams, particularly in order to minimize seepage through the upstream face. Geomembranes are placed on the upstream face of the dam, preventing water infiltration and preventing degradation of the dam material. Geomembranes are externally protected from atmospheric agents by the superposition of a cover layer, like concrete slabs, precast concrete elements, geosynthetic-reinforced gunite, and so on. Geomembranes have also been used in place of or in addition to clayey soil as an impervious barrier in the core of the dam.

In embankment dams where a geomembrane is used as barrier to fluid, a thick geotextile is placed on one or both sides of the geomembrane, to protect it from potential damage by adjacent granular layers. The layers of geotextile can be factory-bonded (Geocomposite) to the geomembrane or can be independently placed. The same technique could be applied for rehabilitation purposes. For example, at Goronyo secondary dam in Nigeria, 13 m high, constructed in 1982 and rehabilitated in 1987 with a geocomposite membrane liner (GCM) application, two different layers of geotextile were laid, both having a protection function. The lower geotextile, bonded in factory to the geomembrane, was glued to the original bituminous concrete facing, while the upper geotextile was independently placed between the PVC geomembrane and the cast-on-place concrete cover layer. Geomembranes can also be used in cut off trenches to prevent seepage of water through foundation (Fig.3.25).

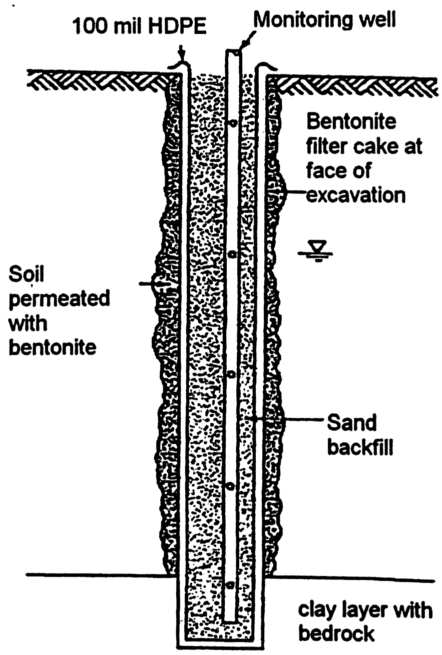


Fig 3.25 Application of Geosynthetics in Cut-Off Trench for Seepage Prevention

3.9 REFERENCES

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CHAPTER IV
MASONARY AND CONCRETE DAMS

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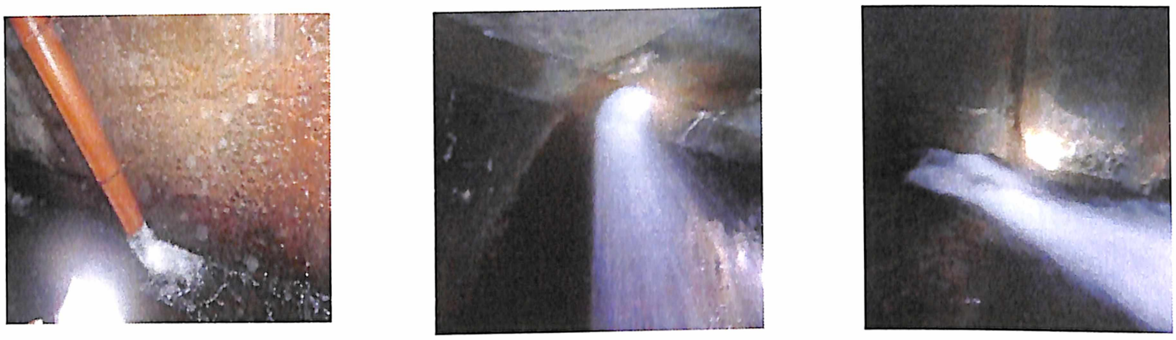
4.0 INTRODUCTION

The seepage occurs in these structures due to a number of causes such as constructional deficiencies, defects in structural members, effects of environmental changes on concrete / material used for construction, excessive loading on the structure etc.. Seepage not only results in loss of water, but if not attended in time, affects the structural integrity of the structure. The materials and the methodology for repairs of damages to these structures shall be suitably chosen as per site condition and quantum and causes of seepage. Seepage is tolerable during early ages of the structure but it increases with age, sometimes exceeding limiting value. However, in some of the recent masonry dams, seepage starts prematurely which makes the structure unsuitable for its intended use and makes it structurally weak. The incidences are rare in case of concrete dams as compared to masonry dams, where seepage is mainly because of cracking due to thermal effects, alkali aggregate reaction or constructional deficiencies.

4.1 EFFECT OF SEEPAGE ON GRAVITY DAM

The construction quality of masonry dam solely depends upon the skill of the mason doing the jointing work of stones. The ratio of mortar to stone depends upon and varies from mason to mason and sometimes may vary with location of work for an individual mason. Construction of masonry dams rests entirely on a group of manual laborers engaged in it. The procedure of construction therefore, is liable to involve numerous human errors affecting quality. The art of placing of mortar in joints and packing joints is most important factor governing quality of joints with respect to seepage. Since the quantity of stones (rubble) and the sand for making mortar is required in abundant, these materials have to be extracted from a number of quarries and as such quality of these ingredients varies to large extent. Seals of rubber or copper are sometimes provided at joints to serve as water stops. Breakage of these seals is more likely during construction giving way to passage of water. These and many other factors discussed herein make a masonry dam more susceptible to seepage. Fig. 4.1 shows seepages in galleries of some masonry dams.



Figs. 4.1 Seepage in galleries of some Masonry Dam

Seepage affects the functionality of dams and its intended use. Seepage deteriorates structural strength affecting its stability and integrity. Some of the old masonry dams in India, namely the Krishnarajasagar dam and Khadakwasla dam were constructed using the codes present at that time and were not provided with any type of seepage draining arrangements like porous drains or drainage gallery. However, these dams are still performing well, giving satisfactory service for decades. Seepage builds up pore pressure which is harmful for the structure. Present standards of construction of dams have therefore made provisions of vertical and horizontal porous drains and drainage gallery for collection and draining of seepage.

In case of concrete dams, it takes a very long period for the pore pressures to build up to the level of design pressure. In concrete dams, the seepage occurs at the bottom of the dam as well as through any plane because of differential pressure gradient from upstream to downstream and is known as Uplift pressure. Further, depending on the composition and grade of concrete used for the construction, dams contain a certain percentage of pores. The seeping water through dam body in due course, may fill in these pores and exerts pressure known as pore pressure. Seepage has both a physical and a chemical influence on the concrete and plays a noticeable role on the state of stresses and the stability of the dam. Both uplift and pore pressures are destabilizing forces and hence, these two major parameters are required to be measured and monitored. To quote the example of Hirakud dam, even after a period of 25 years after its construction, no sign of seepage was observed in a hole drilled from the top of the dam about 8 m from the upstream face of dam.

Due to insufficient data on pore pressure in masonry dams in India, it is difficult to conclude about permissible rate of seepage in masonry dams. If suitable provision of porous drains as per design standards is made in masonry dams, possibility of attaining steady state seepage above minimum draw down level appear to be remote. But over a period of few years, the possibility of saturation in the vicinity of about 3 m thickness of dam below minimum draw down level cannot be ignored. If the porous drains function effectively, the likelihood of saturation of d/s portion can be completely denied. In view of above, the locations of pore pressure meters in dams shall be meticulously selected. Provision of pore pressure meters in three vertical planes confining to the cross-section of the dam below minimum draw down level in a block is a most suitable arrangement. In case of concrete dams, these meters shall preferably be provided in construction joints.

The occurrence and persistence of seepage may result in (i) Structural stability & (ii) leaching.

4.1.1 Pore-pressures on Structural Stability

Even though drains are provided in the dam for seepage discharge, because of various reasons these drains are not functioning effectively. A study made on few dams on the effect of the drains on stability of the dam has revealed that,

- The effect of variation in uplift relief is more pronounced for upstream stress than for downstream stress, shear friction factor or sliding friction factor.
- The relation of upstream stress and relief of uplift is almost linear and this relationship does not vary much for different elevations of the same dam.

4.1.2 Leaching

Leaching is a process wherein the seepage water enters pores of mortar and takes away soluble particles and increases porosity. Leaching is dominant if water is acidic and if the degree of solubility of constituents of cement is more. It also depends on the rate of flow of water. Pure water too has greater capacity of lime dissolution which invariably is the case in fresh water reservoirs. The phenomenon is more obvious in masonry dams than in concrete dams. Leaching action is revealed by lime deposits on exposed surfaces of dams, predominantly on downstream surface and in the galleries. The visual examination of the exposed areas more precisely in the galleries of dam, give an idea of damages due to leaching. Leaching effect in the galleries of some of the dams is shown in Fig.4.2.

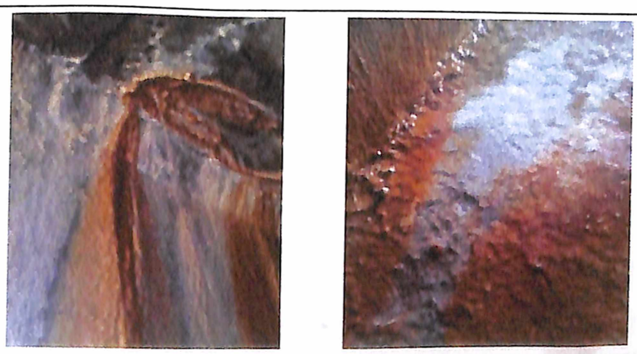


Fig. 4.2 Leaching in Gallery

A limit to the leaching phenomenon is yet to be established. Some experiments have found out that leaching effect is measurable up to approximately 30 mm from crack and beyond 30 cm the effect is negligible. Leaching through concrete dams forms loose and porous layers of concrete. Since numbers of joints in masonry are more, effect of leaching in masonry dams is difficult to evaluate. Leaching of lime will be high in initial stage and gets reduced with time in a properly constructed dam.

4.2 DETECTION OF SEEPAGE

Methods for seepage investigation include study of the related site geology, water balance studies and study of dam instrumentation data. Integration of different methods enables a better understanding of the problem and offer cost effective solutions. Other non-conventional methods like tracer techniques borehole logging, geophysical methods, hydro-geological methods and remote sensing techniques are often used in conjunction with other techniques give a better understanding of the sub-surface properties. The chapter 2 describes the details of the methods and potential in assessing seepage through hydraulic structures.

4.2.1 Theoretical Approach

Theoretically, a dam constructed using cement mortar is supposed to be impermeable. The stones used in the construction can also be considered impervious for practical considerations. However, the combination of these two used in construction of masonry dam do not practically result in an impermeable homogeneous medium. A number of factors affect seepage through dam like quality of construction and reservoir water level. The mode of filling and emptying the reservoir also affects seepage. Both these factors vary with dam locations. Hence a general code for predicting permissible limit of seepage cannot be formulated. However, these limits can be established for each individual dam assuming the quality of construction as good and the dam is assumed to be constructed as per standard design method. Central Water Commission, New Delhi has evolved a method for estimating permissible seepage in masonry dam where drains are provided (Narayana G S et al 1983)

The methods assume:

- No water escapes to downstream portion past the drainage pipes.
- Masonry is permeable having high coefficient of permeability in horizontal direction.

Though mortar and stones individually are considered to have low permeability, their composition is not so because of presence of joints and pores which is unavoidable in masonry construction. It is also assumed that a linear relation is assumed to exist between pressure variation at u/s face and drain and that cent percent seepage is collected by drains which in practice are provided at 3 m interval. It has been shown by the electrical analogy experiments that seepage heads are reduced by more than 80% at drainage pipe line and the rest of the cross section experiences very small hydraulic gradients. Seepage in body of dam is a measure of hydraulic head. When reservoir is at its full level, the seepage is maximum. With these assumptions the upper limit of seepage is estimated as follows. The cross sections and upstream elevations of a typical dam are shown in figure 4.3 and 4.4.

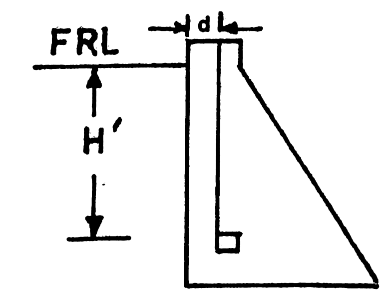


Figure 4.3: Cross-section of dam

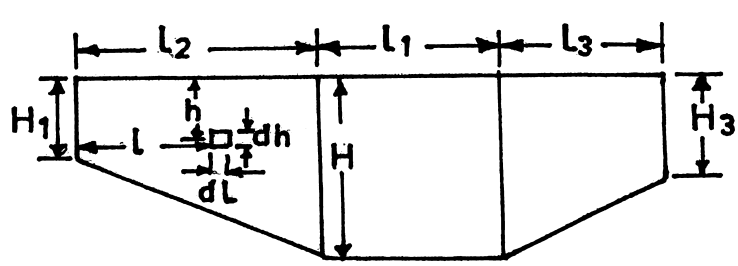


Figure 4.4: Upstream elevation of dam

The notations used are k = permeability of masonry and

h/d = i = hydraulic gradient of flow (linear pressure variation is assumed).

By Darcy's law Q = k i A

Taking an elemental rectangle *dl* ×*dh* at height 'h' below reservoir level and distance '*l*' away from left abutment

$$dQ = \text{k}i dldh = k\,\frac{h}{d} dldh$$

Therefore,

$$Q = \int_{\partial}^{l_2} \int_{\partial}^{H^1} k\frac{h}{d} dldh$$

Where H_l is the maximum reservoir head at any distance *l*, hence, H′ can be expressed as

$$H' = H_1 + \frac{(H-H_1)l}{l_2}$$

Substituting in the above formula and on integration we get,

$$Q_A = \frac{Kl_2}{6d}(H^2 + H_1^2 + HH_1)$$

For rectangles $H = H_1$. Hence Q for rectangle is given by

$$Q_B = \frac{Kl_1H^2}{2d}$$

and $$Q_C = \frac{Kl_3}{6d}(H^2 + H_3^2 + HH_3)$$

Since the total seepage Q = Q_A + Q_B + Q_C i.e.

$$Q = \frac{Kl_2}{6d}(H^2 + H_1^2 + HH_1) + \frac{Kl_1H^2}{2d} + \frac{Kl_3}{6d}(H^2 + H_3^2 + HH_3)$$

With the help of above equation it is possible to estimate maximum upper limit of seepage at different reservoir elevations. However, it shall be noted that, predictability of maximum limit of seepage will depend on permeability value of masonry/ concrete representing whole dam. By obtaining seepage value in well constructed dam, an exercise can be done with the use of above formula for prediction of maximum permissible limits in other dams.

The seepage data for right side of Hirakud concrete dam was obtained and analyzed, which indicated that during early stages of operation of dam, the seepage through the body of dam was not much affected due to variations in reservoir level. Seepage reduced with fall in level but did increase with rise remarkably. The trend followed by seepage with respect to reservoir level is universal. The seepage through foundation steadily decreased with time. This may be due to clogging of seepage paths in the foundation or due to ineffectiveness of drains. However in 1967, the seepage through body of dam indicated variations with respect to reservoir level. The trend of foundation seepage remained unchanged. The maximum seepage through body increased with time. Investigations were taken up for such an increase and cause identified.

The analysis of data of seepage for Hemavathy composite dam in Karanataka indicated seepage on right bank was found higher than that of left bank. This was attributed to spacing of porous drain 6 m interval on left where as at 3 m interval on right side. There was a variation of seepage followed the reservoir head during early stage of operation of dam.

4.3 SEEPAGE CONTROL MEASURES IN MASONRY DAMS

Since seepage in masonry dams is more pronounced than in concrete dams, remedial measures to arrest the seepage in masonry dams is considered in detail in following paragraphs.

4.3.1 Grouting

Grouting aims at filling of the cavities/ fissures with selected material to impart impermeability and strength. It is necessary that the material should block the water passages from upstream to downstream to avoid blocking on the downstream side producing uplift pressure. In case of very long dams without expansion joints, the treatment may be carried out in alternate blocks of suitable length. Commonly used grout materials are cement grout and the epoxy grouts.

These grouts are sometimes modified using fillers such as sand, carborandum powder, plaster of paris or suitable compatible additive depending on width and depth of cracks, volume of cavities etc. Tests on the grout materials and trials are necessary to identify suitable grout materials, their mix proportion, grout pressures, grout hole spacing, and precautions during handling etc. Choices of grout materials and grout mixes are dependent on shape, size and continuity of the cavities/ fissures, seepage water velocities and the strength requirement.

Grouting has proved to be an effective cure for a number of the dams like Shirawta dam, Watwhan Dam, Hemavathy dam, Bhandardara dam, Radhanagari dam, Pagara dam, Sakhya Sagar dam etc. An example of effective strengthening of dam by grouting is epoxy grouting of cracks in the prestigious Hirakud dam, Orissa. Grouting of the cracks resulted in reducing seepage to a substantial level. Grouting technique is elaborated in following paragraphs by illustrating an example of grouting of Talkalale dam (Mallikarjuna P R, et al, 1996)

Talakalale dam is masonry gravity dam with a length of 354 m and a maximum height of 62.5 m from deepest foundation. The bottom portion of the dam (about 12.5 m) is built with stone masonry making use of red cement mortar (cement + sand + burnt brick powder) and next 50 m is built with lime surkhi mortar. The masonry has size stone facing both upstream and downstream and random rubble masonry in the hearting of the dam. Vertical porous pipes were inserted at 6 m interval to drain off seepage water connecting to drainage gallery. The dam construction was started in October 1959 and completed in June 1963. No sluices were provided in the body of the dam. To reduce seepage/leakage found on downstream face, cement grouting were carried out during 1964-65 and in 1965-67 effecting seepage got reduced from 3.91 to 0.23 cusecs but started increasing from 1970 onwards. New drainage holes were drilled to the whole height up to drainage gallery and grouting undertaken from the top of dam escaped freely from downstream face along with seepage water. Due to washing away of binding material, leaching and nonfunctioning of drains holes, the density of masonry reduced from 2330 to 2160 kg/m^3. As the seepage could not be effectively reduced, the work of systematic underwater pointing of all the joints on upstream face was taken up by raking up both horizontal and vertical joints up to about 7.5 cm deep and sealing with cement, sand and accelerator in the proportion of 1:0.5:0.3. This paste was forced into the joints. Seepage got reduced to 3.45 cusecs by pointing over an area of about 3860 m^2 out of total area of 9200 m^2. Thus systemic underwater pointing on the total upstream face of the masonry was proved effective and successful, the seepage thus reduced to 0.25 cusecs.

4.3.2 Steel Jacketing

This is one of the most preventative measures for arresting entry of water through upstream face of dam. It can be quickly adopted particularly when emergent remedial measures are required.

4.3.3 Cable Anchoring

In this method, vertical anchors are provided near to the upstream face of the dams. The locations of anchors are finalized depending on extent of damages and their locations in dam. Addition of this vertical force reduces overturning moment and imparts greater frictional resistance against sliding. The cables are fabricated from high tensile wires or strands. The design involves providing adequate factor of safety against overturning and sliding considering the additional pre-stressing force. Salient features of this method are, speed of construction, non requirement of any draw down of reservoir level, limited operations and economically attractive. The technique is particularly best suited for carrying out emergency treatment. The process is explained in short as follows:

- Drilling of cable holes by percussive 'down-the- hole' drills.
- Waterproofing of cable holes by neat cement-water grout at maximum pressure of 3.5 kg/cm^2.
- Re-drilling of holes by percussive drills and washing of holes and plugging the top.
- Testing each hole for water tightness.
- Preparation of cables - High tensile steel wire for cables, testing and preparation of cables.

4.3.4 Pointing

Pointing is the process of sealing of joints between stones in masonry construction. Usually cement mortar in the ratio 1:3 to 1:4 prepared to have suitable workability is used as pointing material. The voids in between the stones are first filled with stone chips by placing mortar as binding material. The surface is then jointed with mortar extending on both sides of joint. The cement mortar pointing gets deteriorated with time due to various causes, the main reason being leaching action. Leaching carries fines present in mortar and disintegrates it making unsuitable to hold water. If the pointing of the masonry on the upstream face is damaged or peeled off then such locations are to be identified and repaired with modified repair mortars like epoxy mortar, polymer modified mortar etc.

Anjunem Irrigation project is located on Costi river near Anjunem village of Sattari Taluka in North Goa District. The project envisages a gravity masonry dam; pick up weir and two canals, one on right bank and the other on left bank. The masonry dam is 176.0 m in length and 42.8 m in height from the deepest foundation level. The dam have 11 blocks including a spillway comprising four bays each of 7.62 m width and 5 piers each of 3.0 m width. It was suspected that the seepage through the foundation gallery and the discharge through the seepage collection drains in gallery have been increasing. Upstream face of Anjunem dam and seepage in drainage gallery is shown vide Figs.4.5 and 4.6 respectively. Sweating was also observed on the d/s surface of the dam. The seepage quantity recorded was of the order of 3608 liter/minute including about 525 liter/minute from below the foundation gallery. Since the magnitude of seepage was found to be on a higher side than the permissible one, it was decided to examine causes of seepage and to arrest the seepage by repairs and strengthening of the dam.

Fig. 4.5 Anjunem Dam u/s face

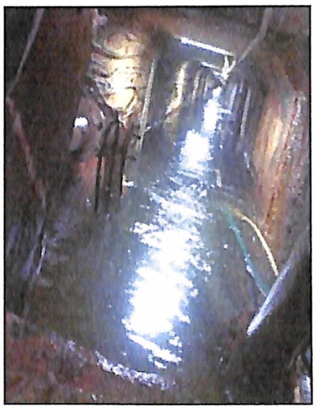


Fig. 4.6 Seepage in drainage gallery

The studies for identification of suitable repair materials and to build up suitable repair methodology to repair and strengthen the dam were entrusted to Central Water & Power Research Station (CWPRS Technical Report No.5019). Accordingly site visit was undertaken and on the basis of laboratory test results and in view of the exposure of repaired surface to ultraviolet radiations (sun rays), the Cementitious based polymeric compound was selected for use. The compound is a mixture of cement, polymer, ironite ceramic material and water. The repair methodology was suggested and the repair material was found to have a minimum compressive strength and bond strength as 300 kg/cm^2 and 9 kg/cm^2 respectively. The abrasion resistance (loss per unit surface area) was found to be 0.30 gm/cm^2 as against value of 0.4 gm/cm^2 for cement concrete surface. Pointing work in progress at Anjunem dam, is shown vide Fig. 4.7.


Fig.4.7: Pointing work in progress at Anjunem dam-Goa

View of the repaired area and area under repair in progress is shown vide Fig 4.8.

Fig.4.8: Pointing work in progress at Anjunem dam-Goa

In order to assess efficacy of repairs and bonding between masonry and repair material, cores were extracted from repaired area on u/s face of the dam (Fig. 4.9 , Fig 4.10).

Fig. 4.9 Core drilling in progress

Fig. 4.10 View of extracted cores

4.3.5 Application of Geo-membrane towards controlling seepage of Kadamparai Dam

In this method, upstream face of the dam is jacketed with geo-membrane by clamping it to surface. This stops the entry of water through the upstream face of dam thus arresting seepage. The methodology of geo-membrane jacketing has been used in Kadamparai dam, Tamil Nadu (Sadagopan A. A. et al, 2005) is discussed in following paragraphs. The Kadamparai dam (Fig. 4.11) is a composite structure consisting of a central stone masonry gravity dam, with earthen embankments. It was observed that the seepage through the drainage gallery of Kadamparai dam was increasing gradually well above the allowable limit (Fig. 4.12).

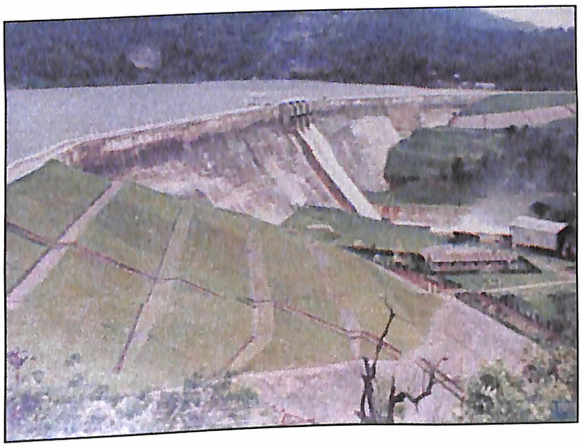


Fig. 4.11 View of Kadamparai Dam

Fig. 4.12 Leakages through gallery

Various means were tried to minimize leakage by vertical drilling from the crest and grouting, racking and packing including chemical treatment in underwater conditions. View showing raking of joints is shown vide Fig. 4.13. However, these methods were unsuccessful and seepage kept on increasing and was recorded about 1400 lpm at full supply level. Accordingly decision was taken for installing a waterproofing geo-membrane on upstream face.

Fig.4.13 Views showing raking of joints in u/s face of Kadamparai dam.

Fig.4.7: Pointing work in progress at Anjunem dam-Goa

View of the repaired area and area under repair in progress is shown vide Fig 4.8.

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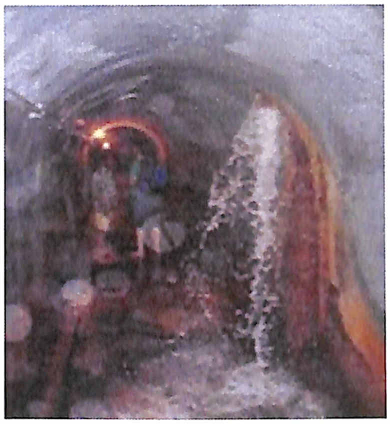

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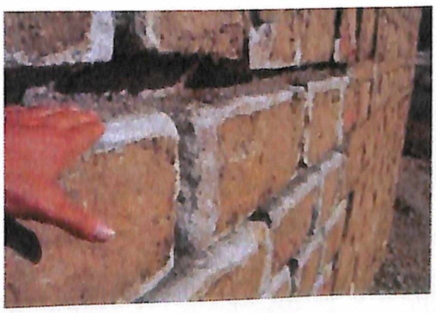

Fig.4.13 Views showing raking of joints in u/s face of Kadamparai dam.

Methodology adopted for jacketing

The concept entails the installation of the drained waterproofing geo-membrane from the crest down to the foundation (Fig. 4.14), installation of the geo-membrane system from traveling platforms and suspending from the crest (Fig. 4.15) and installation of geotextile on the upstream face to protect the geo-membrane against puncturing (Fig. 4.16).

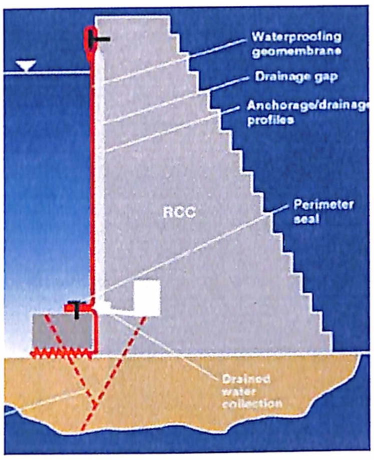

Fig. 4.14 Concept of installation of geo-membrane

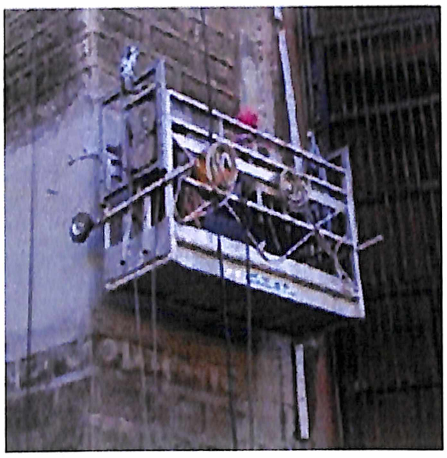

Fig. 4.15 Installation of geo-membrane from crest

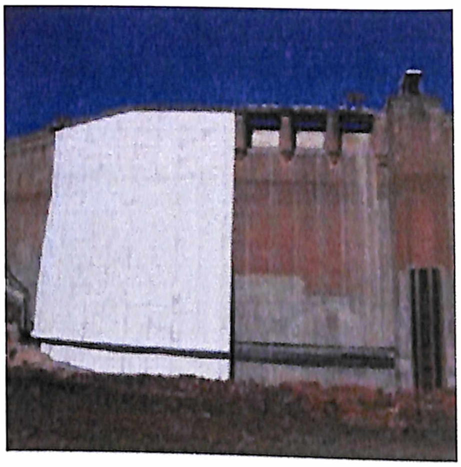

Fig. 4.16 Installation of geotextile to protect geo-membrane

Laying of PVC geo-composite (2.5 mm PVC + 500 g/m^2 geotextile) over the antipuncturing geotextile is carried out on the u/s face of dam body. The rolls of geocomposite were overlapped and welded together (Fig.4.17). The perimeter watertight seal was created by placing chemical anchors, bedding mortar, geocomposite, gasket and stainless steel batten strip and finally specified torque was applied to the bolts of the anchors (Fig. 4.18).

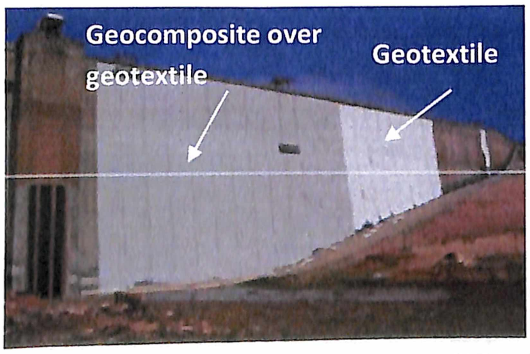

Fig.4.17 Placement geocomposite over initial geotextile layer

Fig. 4.18 View showing anchoring of geocomposite on face of dam

The whole installation covering more than 17,000 m^2 area of u/s face of Kadamparai dam including installation of monitoring system, was completed in 3 months under supervision of Tamil Nadu Electricity Board. A view of the upstream portion of the the dam after geo-membrane installation is shown in Fig.4.19. At present Kadamparai dam has attained its full supply level and rate of leakage has been reduced from 1400 lpm to around 100 lpm. Thus the repair works using geo membrane jacketing worked well in reducing the seepage to a large extent.

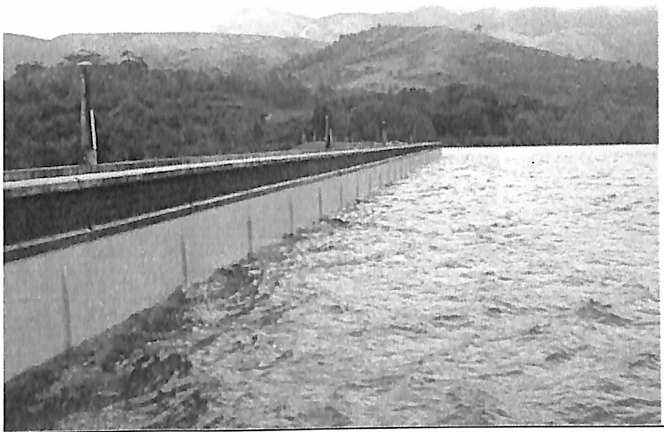

Fig.4.19 View of u/s face of Kadamparai dam after geo-membrane installation

4.3.6 Guniting

The stones and the cement mortar though are supposed to be individually impermeable, their composition stone masonry is not so because of joints in between. The composition does not form a homogeneous structure. In spite of pointing of the joints in between the stone by using cement mortar, seepage persists since operation of dams and increases with time. Guniting is therefore done on upstream surface of dams, invariably in masonry dams. This is a simple process where flowable concrete is pressed on surface to be treated. A steel wire mesh is fixed on surface to be gunited by anchors as a provision for tensile stresses. It is quick and easy process. However it is found that the life of guniting is less than desired because the metallic mesh inside get rusted and expands developing cracks, finally detaching from surface. Hence the guniting process has to be done periodically (every 10 years). Use of bond coats on surface or additive enhances life of guniting.

4.3.7 Case study on Identification of suitable repair material

Repairing damages to Manikdoh Dam, Maharashtra

The 53 m high and 927 m long Manikdoh dam across river Kukadi is a composite dam constructed in UCR Masonry during the period 1976-83. The dam is located near village Manikdoh

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in Pune district of Maharashtra. The dam comprises of an overflow section of length 92 m and inspection and foundation gallery of size 2.3 m x 1.5 m. The upstream face of the dam was gunitted soon after the construction (Photo 4.20). Leakages were observed in dam due to delamination of guniting, weakening of pointing, formation of cavities in masonry portion and other structural defects (Fig. 4.21).

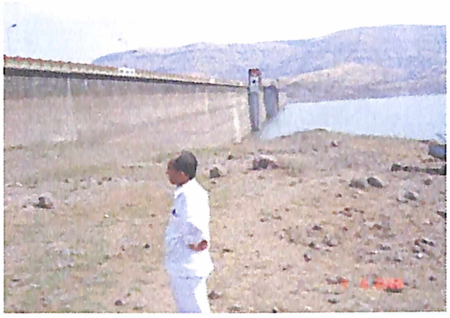

Fig.4.20 View of u/s face showing gunited surface of Manikdoh Dam.

Fig. 4.21 Deep Cavities in U/s face and leakages in gallery

The project authorities referred the problem to CWPRS and requested to suggest suitable repair materials for treatment of damaged portions. Accordingly studies were carried out in concrete laboratory on epoxy compounds and cementitious materials from different firms to identify suitable material for repairs. Necessary studies to determine strength properties of epoxy mortar, epoxy concrete & cementitious compound were conducted CWPRS Technical Report No. 4498 (2007). On the basis of these studies epoxy mortar of proportion 1:6.5 (epoxy: quartz sand by weight), epoxy concrete of proportion 1:1.5:4 (epoxy: quartz sand: aggregates by weight) and cementitious system Beck Bond Micrete were recommended for repair work. The recommended epoxy mortar prepared using epoxy resin 505C and epoxy hardener EH411 with quartz sand as filler indicated compressive strength 400 Kg/cm^2, tensile strength 69 Kg/cm^2 , 7 days bond strength with stone in direct tension under dry condition more than 25 Kg/cm^2 and modulus of elasticity 0.91 x 10^5 Kg/cm^2. The selected epoxy concrete with epoxy resin 505C and epoxy hardener EH411 indicated compressive strength 572 Kg/cm^2, 7 days Bond strength with concrete in direct tension under dry condition more than 24 Kg/cm^2 and modulus of elasticity 1.40 x 10^5 Kg/cm^2. Similarly recommended cementitious material Beck Bond Micrete indicated 7 days compressive strength 353.3 Kg/cm^2, briquette tensile strength 32.1 Kg/cm^2 and modulus of elasticity 2.79 x 10^5 Kg/cm^2.

4.4 SEEPAGE CONTROL MEASURES IN CONCRETE DAMS

Concrete dams are considered to be more homogeneous as compared to masonry dams. The causes of seepage in concrete dams differ from that of the masonry dams. Cracking due to alkali aggregate reaction, thermal gradients and structural loading are the main causes leading to seepage in concrete dams. The cracks generated due to these processes are developing continuously with time because of their nature. Hence suitable remedial measures are required to be taken during the construction itself to avoid cracking due to these effects. Use of sound aggregates, suitable placement temperature of concrete, post cooling of concrete, exercising good quality control measure and adequate design are some of the remedial measures to avoid cracking. The commonly adopted seepage control measure in concrete dams is grouting the cracks. Remedial measures such as strengthening of structure are resorted in case of cracks developed due to excessive structural loading. A number of methods such as buttressing, concrete backing, anchoring etc are available for strengthening the structures.

The width of cracks in concrete dam is small. Concrete being of higher strength than masonry, the material used to grout the cracks shall have good bond strength and low viscosity. Epoxy compounds are well suite for grouting cracks in concrete dams. These compounds have exceptionally good strength and bond properties. These compounds can also be modified using fillers to have the required viscosity as per width and depth of crack.

A number of concrete dams such as Hirakud dam, Karjan dam, Rihand dam etc. which were damaged due to thermal cracking, Alkali Aggregate Reaction (AAR) cracking and rehabilitated by grouting the cracks. Seepage can be effectively controlled by;

- Lengthening the path of the Seepage
- Strengthening of either foundation or upstream/downstream face and/or abutments
- Providing Drainage Galleries
- Periodical Review
- Reservoir Silt

4.4.1 Lengthening of Seepage Path

By extending the seepage path, the larger part of the water potential will be spent on overcoming the frictional resistance of the flow through the earth medium on the way from the upstream face towards the downstream face of the dam. Aprons are provided to achieve the same. Upstream and downstream aprons have the effect of increasing the seepage path under the dam. The effectiveness of upstream aprons in reducing uplift is compromised if cracks and joints in the apron permit leakage.

4.4.2 Strengthening of Foundation, U/s, Abutments & D/s

Proper treatments are given to foundation rock, foundation to reduce weak zones by reinforcing with stronger material after removing weaker material. This can be achieved by grouting. The depth of the grout curtain holes depends upon the nature of the rock in foundation and in general, it may range from 30 to 40 percent of the head of the water on good foundation and to 70 percent of head on poor foundations. Vertical and horizontal water-impermeable elements are constructed by sheet piling, as cement – grouting curtain, or as a diaphragm wall, most often a concrete or asphalt diaphragm wall. The horizontal element are constructed in the kind of covering, i.e. blanket, in front of the dam, made of poorly permeable earth material, or else asphalt, concrete, or reinforced concrete.

Drainage Galleries

For reducing the pressure in the bottom of hydraulic structures, as well as in the body of earth dams, drainages are an efficient means. They are structures in the form of an opening or a gallery in the dam, i.e. pit or well, shaft and in the foundation. Drainage facilitates taking away the water towards the downstream face of the dam and influences the velocity of the seepage and the magnitude of the uplift pressure.

Periodical Review

In general, maintenance should include, but not be limited to periodic testing to locate clogged and inoperative drains, re-drilling or cleaning of drains which have become clogged, installation of additional drains to achieve design concept and periodic monitoring and calibration of pressure gauges. Uplift reduction due to drainage assumes that the drainage system vents the high pressure area under the dam to tail water pressure.

Reservoir Silt

Reservoir silt can reduce uplift under a dam by acting as an upstream apron. Because of potential liquefaction of the silt during a seismic event, uplift reduction due to silt may be lost in seismic situations. If liquefaction occurs, pore pressure in the silt will increase. This condition of elevated pore pressure may persist for some time after the seismic event. For this reason, uplift reduction due to silt may not be relied upon when considering post earthquake stability.

4.5 SIGNIFICANCE OF SEEPAGE CONTROL MEASURES

- Absolute imperviousness cannot be realized in practice but possible to roughly assess the probable upper limit of seepage for a particular dam for the full reservoir level condition.

- Guniting appears to have an edge over pointing as work is quicker and management of quality control should be better but guniting should not be considered as a preventive measure and can be considered as a remedial measure since it acts as an additional line of defense.
- For seepage due to voids created by bleeding of grout, re-grouting will help.
- Grouting when carried out close to drainage holes could result in chocking which necessitates reactivation of the drainage holes. Grouting also enlarges the internal drainage system by blocking the porous drains. Additional drainage holes have to be taken wherever the existing holes are not functioning.
- Pointing of upstream face followed by grouting will help in reducing seepage.
- If the guniting has to be applied at the construction stage, the reinforcement may be properly anchored so as to provide resistance against minor tensions developing at u/s face of the dam.
- It shall be assured that the concrete membrane provided on the upstream face does not get separated from the body of the masonry dam.
- Seepage water mostly carries lime resulting in the formation of encrustations which may block the drain pipes rendering them ineffective in relieving pore pressure.
- Guniting up to MDDL is practical but below MDDL is impossible unless low level outlets are available for depleting the reservoir. In certain cases, dumping earth against u/s face up to MDDL can be carried out and above MDDL guniting can be carried out.

4.6 REFERENCES

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CHAPTER V
CANALS AND RESERVOIRS

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5.0 INTRODUCTION

Availability of fresh water resources, suitable for human consumption and irrigation, is limited. Hence, water must be collected, stored in ponds / reservoirs and transported via canals wherever it is required. Appropriate planning and design of irrigation projects thus play a key role in achieving their high equitable performance.

An irrigation system consists of a reservoir, created by construction of a dam and a network of canals, starting with head-works, main canal, branch canals and major and minor distributaries. Seepage losses in and around reservoirs, lakes, ponds, etc. are mainly due to unfavourable geological conditions. Seepage from reservoir or foundation is a serious concern not only because of water loss but is also a threat to safety of the hydraulic structure. It is therefore necessary to detect, measure and control seepage through reservoir and foundation by adopting suitable remedial measures like foundation grouting, cutoff trenches, impervious blankets and other seepage barriers. Canal seepage losses are generally controlled and minimized by providing different types of linings. This chapter highlights the causes and consequences of seepage through canals and reservoirs along with various methods for detection and measurement of seepage and remedial measures adopted for reducing / stopping seepage.

5.1 SEEPAGE LOSSES AND SEEPAGE MECHANISM IN CANALS

Canals are a major means of transporting and delivering water for irrigation. In India canal irrigation is about 39% of the total irrigated area. Loss of irrigation water in a canal system, due to seepage and evaporation, occurs during its conveyance through canal, sub-canal, distributaries and finally during application in the field. Seepage losses constitute a significant percentage of the usable water. By the time the water reaches the field, it has been estimated that the seepage losses are of the order of 45% of the water supplied at the head of the canal (Sharma and Chawla,1975). The estimated loss of water by seepage from unlined canals is listed in Table 5.1. It is evaluated that if the seepage loss is prevented, about 6,000,000 ha of area could be additionally irrigated in India.

Table 5.1 Estimated Seepage losses in unlined canals

Type of soil	Seepage losses (m^3/day/m^2 of wetted perimeter)		
Clay and impermeable loamy clay canals	0.08	to	0.11
Ordinary clayey loams	0.15	to	0.23
Sandy clays	0.22	to	0.31
Clayey Sands	0.30	to	0.46
Sandy soils	0.45	to	0.53
Sand and gravel mixtures	0.60	to	0.76
Ordinary gravelly soils	0.76	to	0.91
Gravelly soils, very permeable	0.90	to	1.83

Seepage from canals can be mainly horizontal or vertical, or a combination of the two. The dominant mechanism at a site affects the rate of seepage, nature of impact and approach to remediation. The mechanism of seepage from canals are shown in Fig. 5.1 and illustrated below.

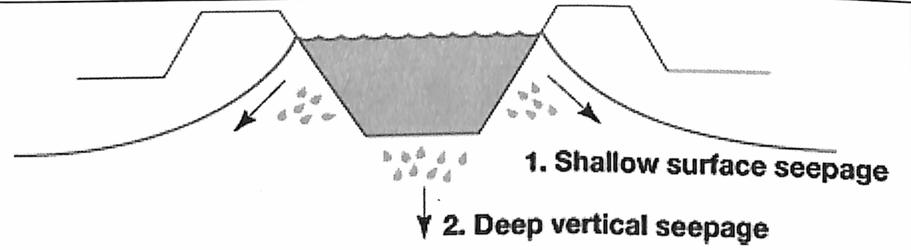


Fig. 5.1 Channel Seepage Mechanism

5.1.1 Shallow Surface Seepage

Lateral seepage through horizontal pathways is a major cause for canal remediation as it can lead to perched water tables, soil water logging, degradation, and bank instability due to saturation. Even if water loss is not necessarily high, there can be a major impact on channel operation and maintenance and the local environment.

Estimation of seepage rates is not always possible, and may be of significance mainly because of its impact on soil conditions, especially water logging and salinity. Locations can be detected by surface mapping and remote sensing. Remediation using cut-off walls, trenches and bank lining may be all that is needed and this is likely to be less expensive than lining of the entire wetted perimeter of the channel.

5.1.2 Vertical Seepage

Vertical seepage processes are complicated to map, however, there are local and regional groundwater impacts from seepage, including increased recharge and rising water table. The assessment of impact of seepage on water table needs to be taken into consideration before extensive remediation is undertaken. For example, in some irrigation areas, existing high water table from regional irrigation cause land degradation. Remediation might have no effect on adjacent land systems if high water table is due to external factors that are not altered by canal works. Under conditions of deep (vertical) drainage, canal works can increase seepage. Examples include deepening of canals and exposure of potentially high-seepage pathways, or de-silting of the existing canal and reopening blocked seepage pathways.

5.2 DETECTION AND MEASUREMENT OF SEEPAGE

The assessment of canal seepage is done by mathematical methods i.e, empirical, analytical and analog and by direct (traditional) techniques viz. seepage meter method (accuracy ±20%), stream-gauge methods (accuracy ±5% (Hotchkiss, R.H, 2001)), ponding tests, inflow–outflow method, etc or by using Acoustic Doppler Current Profilers (ADCP) (Kinzli. K. D, et. al, 2010). Among non-conventional methods, geophysical techniques like electrical methods (electrical resistivity and self potential method (Watt and Khan, 2007), electromagnetic and ground penetrating radar combined with airborne remote sensing technique, borehole geophysical methods (natural gamma, resistivity and neutron logging, etc.) and tracer techniques provides rapid and inexpensive spatially-dense information over large volumes regarding potential seepage areas of the canal system (Ronald Kaufmann, 2009), when canal is in full service. But the application of more than one of these techniques is required for reducing the ambiguity in interpretation of anomalous features and quantification, as well. Nowadays, the non-conventional tracer method is being increasingly used as an advanced and cost effective technique to detect seepage zones, estimate seepage losses, delineate seepage path and movement of water (Flury. M et. al, 2003, Moser. H, 1995) because of its stable and sensitive nature, easy solubility in cold water and easy detectability at low concentrations, having water like movement and without degradation during the time frame of interest (Markus Flury, et. al, 2003). The details of these techniques are discussed in the relevant chapters.

5.3 REMEDIAL MEASURES FOR CANALS

5.3.1 Canal lining

Seepage losses from canals could be effectively minimized by providing an impervious medium, known as 'Lining', between the porous soil and the water flowing in the system. Lining of canals can significantly reduce seepage losses during conveyance. Lined channels have a smaller surface area for a given discharge than unlined channels. Typically a lined channel will have 40% of the unlined surface area for a given discharge. Therefore, even at the same loss rate per unit area there will be a saving of water. The additional cost due to canal lining is rather a wise investment as the benefits of water saved in terms of additional irrigation, food production and reduced canal section compensate the cost of lining.

Other possible benefits of lining a canal include:

- Water conservation
- No seepage of water into adjacent area, prevents water logging
- Reduce canal dimensions and
- Reduce maintenance

5.3.2 Choice of Lining

Before the decision is made to line a canal, the costs and benefits of lining have to be evaluated. If lining is required, a high quality of construction is essential. Further, without adequate supervision, poor construction of channels will cause reduction in its life and lead to higher maintenance costs.

The choice of type of lining depends primarily on the following factors;

- **Economy:** The cost of the lining must be justified by the benefits achieved by its implementation.
- **Structural stability:** The lining must withstand the static and dynamic forces exerted on it under variable conditions.
- **Impermeability:** The type of lining chosen must reduce the seepage losses considerably.
- **Durability:** The lining must be able to withstand the destructive effect of chemical action of salts, abrasion due to the sediment transported by the water, Ultraviolet rays due to sun, moisture changes, destructive effect of weeds, rodents, vandalism, etc.
- **Reparability:** All types of lining get damaged with time so the type adopted must permit easy repair and replacement operations.

- **Fast implementation:** As the implementations of the lining require total or partial interruption of the canal, types of linings which can be installed within short times have obvious advantages.

5.3.3 Types of Canal Lining

The most common types of lining irrigation canals can be classified as:

- Hard surface linings
- Earth linings
- Synthetic membrane linings

Hard Surface Linings

Hard surface linings require a sub-grade well compacted in such a way that no settlement will occur, besides the fact that the surface of the sub-grade be set at the exact elevations and slopes corresponding to canal profile.

The main problem regarding seepage prevention with hard surface lining is that after cracking, the seepage can be large; also it increases considerably with time due to deterioration. According to calculations made with a numerical model, the seepage from a canal whose rigid canal lining has been cracked to 1% of the lined area, will reach almost 70% of the amount of seepage in the same canal but unlined (Report on Canal Lining in Haidergarh and Juanpur branch canals, Uttar Pradesh). The expected seepage losses from hard surface lined canals are assumed as 0.05 $m^3/day/m^2$ of wetted perimeter (BIS: 4745-1968).

Types of Hard surface lining

There are seven different types of hard surface lining applied for canal lining.

i) Boulder Lining / Stone Pitching

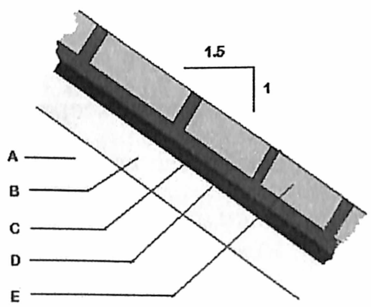
This type of canal lining is achieved by proper placement and packing of stones, either after laying a filter layer over the soil surface or without a filter, depending upon the site requirement. To reduce the resistance to flow, 20 mm to 25 mm thick cement plaster is provided as a finishing surface. Stones are generally placed on leveled sub-grade and hand packed. This type of lining is suitable where stones of required specification are available in abundance locally.

ii) Brick / Burnt clay tile / Precast Concrete Tile Lining

Brick lining is very common type of canal lining in India, the probable reasons being non-requirement of skilled mason for its construction, ample availability of material and cost effectiveness. Seepage losses from single-brick layer linings in good conditions can be as low as 0.05 $m^3/day/m^2$ of wetted perimeter. But in this type of lining also, as with all hard surface linings, seepage will increase with time and even reach a much higher rate approaching the rate corresponding to the canal in unlined condition.

Brick tiles can be plastered to increase its efficiency and durability. Sometimes a layer of tiles is laid over a layer of brick masonry. The top layer is laid in 1:3 cement mortar over 15 mm thick layer of plaster in 1:3 cement plaster.

In order to further reduce the amount of seepage, 0.5 mm to 1.0 mm thick impermeable synthetic geo-membrane, sandwiched between two nonwoven geotextile fabrics can be laid over the soil and under the hard surface lining. But due to the relatively low friction angles between the synthetic geo-membrane and soil, and between the geo-membrane and mortar layer under the bricks, a 1.5:1 side slope cannot ensure stability of lining against sliding. Due to space restrictions it is not possible to decrease the inclination of the side slope to 2:1 or flatter. As such, 6 to 8 mm thick bitumen (asphalt) layer is sprayed in situ under the lining (Fig 5.2).



LEGEND:

A - Sub-grade (compacted, sterilized and smoothened)

B - 100 mm thick layer of selected local sand and silt

C - 6 to 8 mm thick impermeable asphaltic hot-mixed sprayed membrane

D - 1 cm thick 1:4 (cement to sand) mortar

E - 7 cm thick brick

Fig. 5.2 Brick Asphalt Lining

The asphalt should be applied at a temperature between 175°C and 210°C. In order to prevent the penetration of vegetation into the impermeable asphaltic layer, the soil under the lining should be cleaned and free from any organic matter including weeds, seeds etc.

iii) Concrete (Reinforced or Unreinforced)

Concrete, cast in place or pre-cast, with or without reinforcement is a most common type of lining in irrigation canals. Water losses from seepage in concrete lined canals can be as low as 0.04 $m^3/day/m^2$ of wetted perimeter. Concrete lining can be placed in many ways, including hand placing by plastering or by using forms and pouring alternate panels or by using prefabricated concrete elements.

Lining thickness of about 5 cm to 12 cm is generally adopted for larger canals and stable side slopes and are considered to be between 1.5H:1V to 1.25H:1V. Reinforcement to the extent of 0.1% to 0.4% of the area in longitudinal direction and 0.1% to 0.2% of the area in the transverse direction reduces width of the shrinkage cracks, thereby reducing seepage. The concrete must not be very fluid to avoid it, creeping downward from the sides. On steep side slopes, formwork is necessary to hold the concrete in place until it sets. Small openings or expansion joints spaced at intervals of 1.5 m to 3 m are needed for the expansion and contraction of non-reinforced concrete. These joints are filled with flexible, asphaltic material to prevent water leakage. For small canals, prefabricated concrete elements can also be used. In areas where the ground water table is likely to rise above the invert level of the lining and cause undue uplift pressure, drains are laid below the lining to release water and relieve pressure.

Shotcrete or gunite lining is a cement and sand mixture (consisting of about 1 part of cement to 2 to 4 parts of sand), with water, applied pneumatically under pressure using a special nozzle to the canal surface which is to be lined. The lining can be reinforced by means of a wire mesh, if necessary. The thickness of this type of lining varies from 2.5 cm to 8 cm according to the design discharge of the canal. If total imperviousness is required, a geo-membrane can be installed before the shotcrete mixture is applied. Shotcrete lining is convenient for lining small sections, for repair of old linings and for placing linings around curves or structures like piers, abutments, etc.

The subgrade material over which the concrete lining is placed should satisfy certain standards viz. it should not be prone to differential settlements and it should be of non swelling type. The minimum specifications suggested for backing material are as follows:

Gravel content	< 10 %
Sand content	30 - 40 %
Silt content	45 - 50 %
Clay content	20 - 25 %
Compaction	96 % of standard proctor density, in layers not exceeding 20 cm
Moisture content	Optimum Moisture Content (OMC) ± 1.5 %
Shear Parameters	Cohesion (c) 0.2 kg/cm^2 for friction angle 25° to 0.5 kg/cm^2 for 2 to 15°

There are few demerits of concrete as a lining material like relatively high cost, longer period of time required for installation and lack of capability to adjust itself to differential settlement of the underlying soil. Fig 5.3 shows the concrete lined canal.

Fig. 5.3 View of Concrete Lined Canal

iv) Asphaltic-Concrete Mixture

In order to ensure a high degree of impermeability of the hard surface linings, it is recommended to spray over the soil surface, before installing the lining, an asphalt mixture in such a way as to form an impermeable layer. This type of lining consists of a mixture of 6.5% to 8.5% (by weight) of asphalt and cement and selected well graded aggregates passing through 19 mm sieve, mixed hot applied either by hand or by means of adequate equipment to form 5 cm to 10 cm thick layer depending on the design discharge of the canal.

Susceptibility to penetration by growth of vegetation and non suitability in areas prone to high temperature conditions are the major drawbacks of lining with exposed asphaltic material.

v) Soil Cement and Mortar Lining

A 15 cm to 20 cm thick layer of soil-cement, which is a mixture of 2% to 8% cement by weight and selected soil containing not more than 10% to 35% passing the 0.075 sieve, has been successfully utilized for canal lining. Although the cost of this type of lining is much lower than concrete lining, it easily cracks and seepage may start recurring. In order to attain a satisfactory degree of imperviousness, a synthetic geomembrane or a sprayed layer of bitumen must be installed under the soil-cement lining.

vi) Concrete filled HDPE (High Density Polyethylene) Cellular Confinement Systems

Cellular confinement systems are geocells made of HDPE (High Density Poly-Ethylene) filled with concrete (Fig 5.4). The thickness of this type of lining can be 5 cm, 7.5 cm, 10 cm, 15 cm or 20 cm depending upon the canal discharge, velocity of flow and degree of turbulence. This type of lining is more elastic than other hard surface linings and can adjust itself to moderated differential settlements of the underlying soil. In order to ensure a good degree of impermeability in this type of lining it is recommended to install a geo-membrane or asphaltic layer under the cellular confinement system.

vii) Concrete filled Synthetic Mattresses

This type of lining consists of a polypropylene, polyester or nylon mattress filled with concrete under pressure. The thickness of the resulting layer is between 10 and 20 cm, according to the hydraulic characteristics of the flow in the canal. Similar to cellular confinement systems, to achieve the desired degree of impermeability, it is recommended to spray an asphaltic layer over the soil surface, before installing the concrete filled mattress.

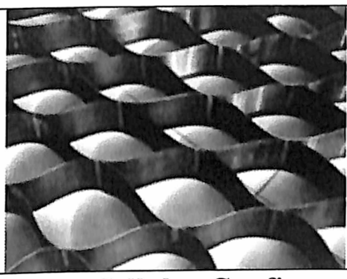


Fig 5.4 HDPE Cellular Confinement System

Earth Lining

One of the oldest methods for reducing seepage losses is to remove the porous earth and replace it with soil of low hydraulic conductivity viz. clay material. The clay is moistened and placed in layers on the bed and sides of the canal. Each layer should be well compacted. Care must be taken to avoid cracks due to moisture changes. Clay layer can be covered with gravel for protection against weathering, erosion and mechanical damage. In places where erosion can occur due to the hydraulic characteristics of the flow, a protective lining must be provided over the earth lining.

The different types of earth linings used in canals are classified as follows:

- **Stabilized Earth Linings:** The sub-grade is stabilized using either clay for granular sub-grade or by adding chemicals that compact the soil.
- **Loose Earth Blankets:** Fine grained soil is laid on the sub-grade and evenly spread. However, this type of lining is prone to erosion, and requires flatter side slopes of canal.
- **Compacted Earth Linings:** In this lining the graded soil containing about 15 percent clay is spread over the sub-grade and compacted.
- **Buried Bentonite Membranes:** Bentonite is a special type of clay soil, found naturally, which swells considerably when wetted. Buried bentonite linings for canals are constructed by spreading soil-bentonite mixtures over the sub-grade and covering it with gravel or compacted earth.

- **Soil-Cement Lining:** Cement and sandy soil are mixed and then compacted at optimum moisture content or cement and soil is machine mixed with water and then laid.

Synthetic Membrane Lining

Synthetic impermeable membrane linings (Geo-membranes) can be laid either with an exposed membrane or with a buried membrane. Exposed membrane linings are susceptible to deterioration due to sun exposure, weed puncture, livestock traffic, maintenance equipment and vandalism. As such, membranes buried under soil layer are a preferred choice for canal lining. According to United States Bureau of Reclamation (USBR), characteristics of the soil to be adopted for covering the geo-membrane, should permit attaining high in situ density corresponding to a compaction density of at least 95% from Standard Proctor test. The surface of soil under the geo-membrane must be free from puncturing objects like stones, sticks, roots, etc. It may even be necessary to replace the original soil with a layer of clean soil of 20 cm to 25 cm thick under and over the geo-membrane. Instead, or additionally, a non woven geotextile could be provided. Growth of unwanted weeds and other vegetation that can damage the membrane should be prevented. Fig. 5.5 shows application of synthetic membrane lining.

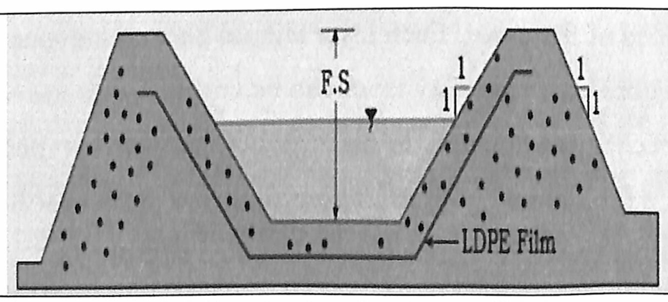


Fig. 5.5 Synthetic Membrane Lining

According to US Environmental Protection Agency guidelines, for typical conditions usually met in canal lining works, the minimum requirements of the geo-membranes required to be fulfilled are given in Table 5.2.

Geo-membranes can be made of Polyvinyl Chloride (PVC), Low Density Polyethylene (LDPE), High Density Polyethylene (HDPE), Flexible Polypropylene (FPP) etc. Since they will be always buried, their Ultraviolet resistance is not critical (unless exposed to the sun for weeks during their installation).

i) **LDPE Membrane**

Cement concrete cover over LDPE membrane has been used in canals with banks as steep as 1.3:1 on Ravi Canal (J&K) and 1:1 in Malaprabha canal (Karnataka). The thickness of LDPE film depends on whether or not it is the primary water barrier constituent of the lining. In case it is used as a primary water barrier its thickness varies from 150 micron-250 micron and a rigid cover is provided to protect it against damage and heaving. The rigid cover may also be provided as the main lining and the film is provided as a secondary lining material. (generally 100 micron film is provided for this purpose).

Table 5.2 Minimum Geomembrane Requirements for Canal Lining

Geomembrane Property	Test Method (ASTM)		Minimum Value
Thickness	D 1593	Standard specification for Nonrigid Vinyl Chloride Plastic Film and Sheeting	0.75 mm
Tensile (25 mm strip)	D 882	Standard test method for Tensile Properties of Thin Plastic Sheeting	9.0 kN/m
Tear	D 1004	Standard Test method for Tear Resistance (Graves Tear) of Plastic Film and Sheeting	46 N
Puncture	D 4833	Standard Test Method for Index Puncture Resistance of Geotextiles, Geomembranes and Related Products	140 N
Impact	D 3998	Test Method for Pendulum-Impact Resistance of Extrusion Plastometer Extrudates	12 Joules

ii) HDPE Membrane

In order to keep the cost of the lining as low as possible (while not compromising on safety), an HDPE type of membrane is recommended. According to the US bureau of Reclamation, the membranes to be adopted should be at least 0.50 mm thick to permit their seaming by means of welding, and fulfill minimum mechanical and chemical requirements especially those regarding tear and puncture resistance and seam strength.

The seaming of the geo-membranes is made by means of welding with special equipment and performed under supervision. Usually, an overlap of about 15 cm is required. Another consideration must be given to the scour that is expected to occur in the vicinity of structures. This scour can reach a depth far exceeding the depth of laying of geo-membrane in the canal. Therefore, near structures (piers, abutments, cutoffs etc), the geo-membrane must be laid well under the scouring depth.

The advantages of synthetic film lining are:

- It proves to be a superior moisture barrier than any other construction material. Permeability tests conducted on this type of lining showed that a 400 gauge PE film subjected to a hydraulic head of 3.65 meter has stayed water tight for about 15 years.
- By use of membranes, the total time involved in lining work can be substantially reduced, thus facilitating wider coverage during short closure of canals.
- The other advantages include ease of installation and transportation being light weight, impermeable to liquids and gases, flexibility, resistance to microbiological attacks, etc.

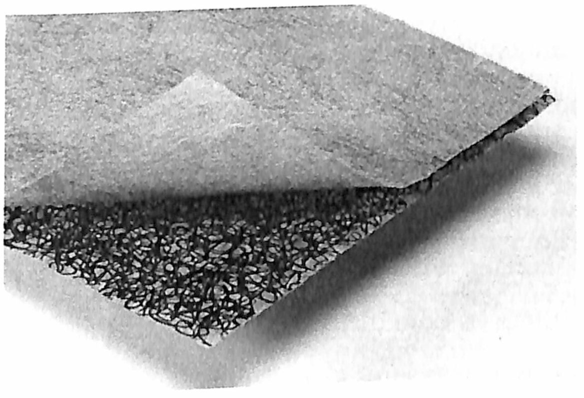
Notwithstanding the above, synthetic film lining has the following disadvantages:

The synthetic film is susceptible to puncturing by uneven soil surface, impact of construction traffic and equipments, weed growth etc. A layer of sand would help as a leveling course. Risk of vandalism with regard to the film is high due to lack of awareness amongst farmers regarding the material and value of water. Also, burrowing by rodents proves to be a major cause of menace. The easy and practicable solution to rodent menace is laying of sand layer. Want of specialized equipments and skilled labour for installation and seaming of the material is another consideration, which to some extent has been overcome due to increasing use of geosynthetics in India and thereby more number of manufacturers and suppliers coming up for the products.

iii) Geo-composites

Lining Geo-composites are found in combinations of membrane-grid-textile, membrane-grid or grid-textile sandwiches (Fig 5.6) which can be used depending on the requirement of the project. In case of the canal lining systems, Geo-composites having HDPE membrane-grid-textile combination are most suitable. The grid eliminates the requirement of graded filter for drainage resulting in considerable reduction of time and costs and increasing canal capacity of discharge. The textile layer provided in these composites prevents the fines from interfering with the drainage path. Geo-composites are used in canal lining system with water proof membrane in contact with the cover (required to weigh down the geo composite on slope) and geotextile is laid against the well graded slope. Geo-composites are available in light weight roll form which can be easily laid on slopes as well as on prepared canal beds. They have strength ranging from 6.5 kN/m to 23 kN/m which can adequately resist soil pressure on slopes and distribute concentrated forces preventing local shear failure conditions from developing.

Geosynthetic Clay Liners (GCL's) are also widely used these days for lining purposes. GCL's are composites that combine geotextile outer layers with a core of low-permeability sodium bentonite clay. Bentonite is a natural sealant, which hydrates on contact with water. GCL's are used as a replacement for compacted clay liners. GCL's can provide approximately the same hydraulic protection as one metre of compacted clay.



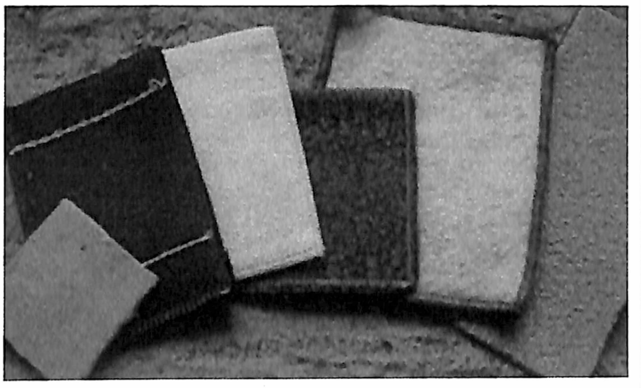


Fig. 5.6 Geo-composites and Geosynthetic Clay Liners

5.3.4 Case Study on Canal Lining

Design of slopes of drain to collect subsurface water, Muktsar, Punjab

Punjab Construction & Drainage circle had constructed 6 km long main drain, for draining water, collected from subsurface drainages from the surrounding water logged areas, so as to maintain the water table below root zone for agriculture requirements. The drain was constructed in 4.0 m cutting with bottom width of 2.6 m and side slopes of 1:1. However, since its operation, sloughing and failure of side slopes of the drain had become regular feature, resulting in local obstruction to flow and flattening of bed slope of the drain.

Site investigation was carried out to assess the general condition of the surrounding area of the drain. The average field bulk density and moisture content were found to be 1.95 gm/cc of 27% respectively. Dynamic cone penetration tests (DCPT) were carried out to assess compactness of top strata. The average DCPT values, from ground level up to bed level, were found to range from 1 to 7 (corresponding SPT value being 0 – 4) and more than 10 for strata below the bed level of the drain. Disturbed and undisturbed soil samples were collected along the length of drain by drilling auger holes / trial pits.

The soils from the slope and foundation were classified as Clayey silt (CL-ML) as per BIS. The shear strength parameters from Direct shear tests were c=0.07 kg/cm^2 and ϕ=6^0. The maximum dry density (MDD) and optimum moisture content (OMC) from Standard proctor test were 1.73 gm/cc and 12% respectively. The corresponding shear strength parameters were found to be c=0.19 kg/sq.cm and ϕ=18^0. Slope stability analysis of design slope of the drain and water table being 2.5 m below bed level indicated the factor of safety of 0.9 as against 1.3.

The slopes of the drain are required to collect the seepage water effectively from the surrounding water logged area and at the same time maintain stable slopes for proper functioning as a drain. Fulfilling these objectives, two alternate proposals of the cross-sections of the drain, were recommended. In the first proposal, the existing loose clayey silt material was to be excavated from 5 m X 18 m section and the soil to be re-compacted with maximum dry density (1700 kg/cu.m.) and optimum moisture content (12%) to form a section having with side slopes of 1:1.5. A layer horizontal filter below the bank level and also vertical filters on both the banks of the canal are to be constructed along the length of the drain. Slope protection measures consisting of a non-woven geosynthetic layer was suggested (Fig 5.7).

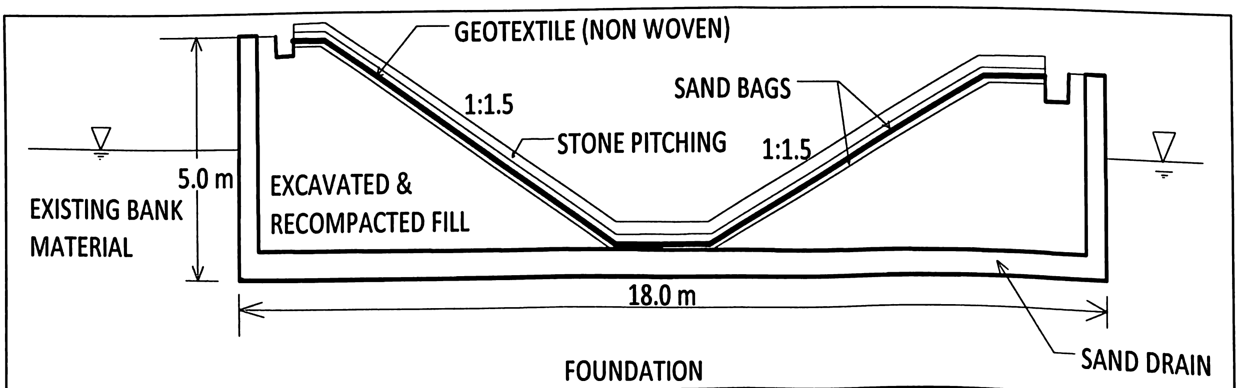


Fig. 5.7 Recommended Solution - First Proposal

In the second proposal, the existing bank is to be redressed to slope of 1:1.5 (V: H) and a double layer filter is to be provided. Bottom filter would comprise of 0.55 m thick layer of sand gravel mixture laid over dressed slope. The upper filter layer would consist of a geotextile filter with a layer of half filled sand bags of 15 cm thickness on either side of it and a protective layer of pitching with open joints on the top of cover layer (Fig 5.8). (CWPRS Technical Report no. 3654)

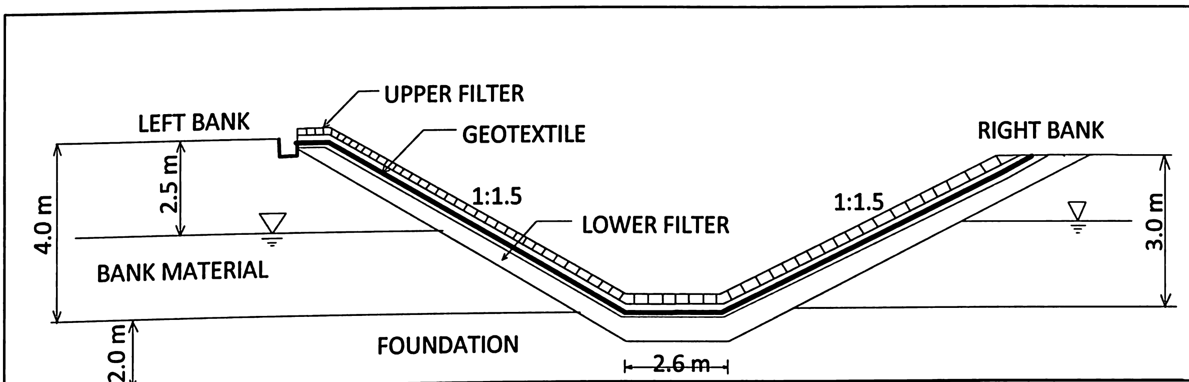


Fig. 5.8 Recommended Solution - Second Proposal

5.4 SEEPAGE LOSSES IN RESERVOIRS

A reservoir is a manmade lake that is created when a dam is built on a river. River water backs up behind the dam creating a reservoir. Seepage losses in reservoirs diminish the volume of water in reservoirs. The loss of water through reservoir seepage should be considered during planning and designing a project.

5.4.1 Seepage Measurement

Seepage identification and measurement is often undertaken in situations where there is a known or suspected seepage problem requiring investigation. After visual inspection which is only a preliminary assessment, the site specific parameters that influence seepage have to be gathered and mapped. Data that has to be collected includes groundwater depth and quality, pond dimensions and capacity, evidence of seepage etc. Available data should be carefully evaluated to develop a conceptual understanding of seepage mechanisms and to identify factors that might allow successful measurement. This helps in selection of a particular technique that may be most appropriate.

If mapping of zones of different seepage or potential seepage is required, geophysical surveys, subsurface methods such as soil and geological profile classification, and both groundwater and surface observations have to be used. Estimates of the rate for different mapped zones can be undertaken using the techniques listed above, once the mapping is complete. Variable Zones of seepages can be rapidly and cost effectively identified using a mapping process based on geophysical surveys or remote sensing.

Water balance analysis involves a system-wide analysis of flow data, taking account of all diversions and inflows (including rainfall and evaporation). In some respects this is similar to inflow-outflow testing but differs in that it does not focus on one particular section of channel and is not likely to involve specific testing. The advantage of regional water balance analysis method is that it enables an estimate of magnitude of seepage loss. This may be broken down into channel or sub-channel sections depending on the location of gauges.

Geophysics is often the preferred technique as it is applicable in a wider range of situations. Remote sensing also has the potential to rapidly and cost effectively cover large areas. In addition, indirect methods include Soil and geological profile classification, Groundwater assessment (Including water-level monitoring, mathematical modelling and hydro chemical (tracer) investigations. Each method has its benefits and limitations. However, a combination of direct and indirect (spatial analysis) methods can provide greater benefits than either method on its own as it can provide information on both the magnitude and spatial variability of seepage.

5.4.2 Seepage Control Measures

Understanding the local geology and finding out sub surface conditions is crucial in building a strong, water tight reservoir. A thin clay layer overlying gravel or an underlying limestone can create large passages for water to escape through the rock. To understand the local geology, trial pits should be excavated across the reservoir site. Detailed geological and geophysical investigations can also be taken up depending upon what is discovered in the trial pit excavations. Sites with extremely unfavourable conditions should be avoided as far as possible.

To control seepage from reservoir through the dam foundation, it is necessary to either to check or drain away the seepage water safely, so that stability and functioning of the dam is not affected. Measures like foundation grouting, providing cutoff trenches and upstream impervious blanket can be adopted to control reservoir seepage.

Foundation Grouting

The principal objective of foundation grouting is to establish an effective barrier to seepage, thereby reducing hydrostatic uplift pressure under the structure (Figs.5.9a and b). Secondary objectives are to consolidate the rock under the structures thus creating a more unified and monolithic foundation, reducing deformability of jointed rocks and control of potential piping. Grouting is often used as a means of controlling seepage by filling the open cracks / joints in a rock foundation so that water cannot leak out. Grouting is carried out by pumping grout (a mixture of cement and water) under pressure into holes drilled into the rock foundation in order to seal voids, cracks, seams, fissures or other cavities in soils or rock strata.

Grouting operations and techniques are not only influenced by the subsurface conditions but also by purpose and objectives of the grouting program. Grouting has been effective as a remedial treatment to correct foundation deficiencies or to repair damages. The types of grouting treatment applied to hydraulic structures are i) Curtain grouting, ii) Area grouting, iii) Contact grouting, iv) Backfilling boreholes and v) Specialized grouting.

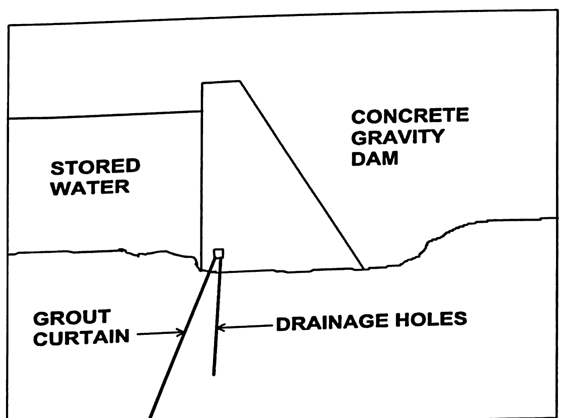


Fig 5.9 (a) Dam Section showing Grout Curtain

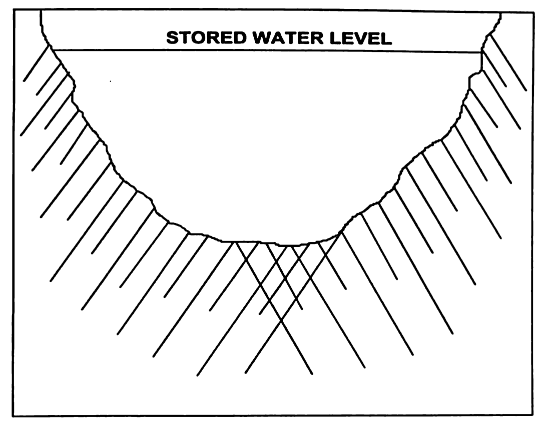


Fig. 5.9 (b) Typical Layout of a Grout Curtain

Cutoff Trenches or Walls

A cutoff trench is a trench excavated in the foundation below the dam and backfilled with an impervious material, such as clay or concrete, to form a watertight barrier. Cut off trenches or walls are designed to lengthen the seepage path, dissipate reservoir head to reduce exit gradients at safe levels and reduce seepage of water from reservoir through the pervious dam foundation.

Upstream Impervious Blanket

An upstream impervious blanket immediately upstream of a dam is used to seal the reservoir bottom and sides thereby reduce seepage quantities and pressures beneath a dam.

Use of Geosynthetics

Geosynthetics prove to be an effective protection against seepage losses when used as liners in reservoirs. Their low cost, wide spread availability and relative ease of installation make geo-membranes and GCLs more popular for lining architectural ponds, recreational ponds, agricultural lagoons and potable water reservoirs.

It can be summarized that the most satisfactory and optimum solution to seepage problem in canals and reservoirs is site specific depending upon varied factors viz. the site conditions, economy, availability of materials etc. However, with innovation, research and improved technology; new advancements are coming up for solution to seepage problem in canals and reservoirs. Adopting these methodologies can surely lead to a successful way in preventing loss of precious water and benefit economy.

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CHAPTER VI
SUMMARY AND CONCLUSIONS

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A huge amount of financial investment is made in planning, designing, construction, operation and maintenance of hydraulic structures for storage of water to meet the needs of water supply, irrigation and hydropower for socio-economic development. However, seepage through these structures is a cause of great concern since it is a potential threat to safety and stability of the structure. There are reported incidences of catastrophic dam failures in India and other parts of world due to excessive or uncontrolled seepage. In India, about 75% of dam failures are due to seepage related problems.

Investigations conducted on dam failures, occurred in various countries have confirmed that a majority of these failures could have been avoided by proper design, construction and regulation of seepage. The main causes of seepage in hydraulic structures are ageing of structure, presence of fracture, fault or shear zones in the pervious foundation, construction deficiencies, uneven settlement of structure, faulty design, etc. In a way, it can be said that occurrence of seepage is unavoidable owing to numerous reasons discussed in the preceding chapters. As such, adoption of remedial measures to control and regulate seepage to acceptable limits is the only feasible alternative.

Investigations for seepage control or regulation start right from studies which involve understanding of site specific geological characteristics. These comprise of geological, geophysical and hydrological investigations. It is mentioned that inadequate understanding of site specific geological parameters related to foundation rock mass behavior has led to many dam failures in the past. Non-conventional techniques involving tracer techniques, borehole logging and remote sensing techniques play a key role in carrying out such investigations. Tracer techniques help in identifying the origin of seepage, its path and interconnection between two water bodies and also help to determine seepage velocity. Borehole logging investigations can provide in-situ assessment of engineering properties of the subsurface, potential seepage pathways, lithological variations and solution activity. Remote sensing techniques can be used to identify potential seepage sites in conjunction with other spatial data. Geophysical methods, in general, address investigating and monitoring seepage and internal erosion in hydraulic structures such as mapping of geologic features and monitoring of seepage. Rock Mechanic studies determining in-situ permeability of dam foundation by Packer tests further helps to ascertain quantum of seepage which further acts as a guide towards effective seepage control measures.

Mathematical modelling by analytical and numerical methods provides a great tool for assessment of seepage through earth dams and foundation. With the advent of high speed computers and advanced software's, these tools are becoming immensely popular. The effects of seepage through dam body and parameters such as seepage discharge, uplift, pore pressures, etc. are required to be continuously monitored by proper instrumentation. Periodical recording of instrument data, its analysis and interpretation is also equally significant.

Once the occurrence of seepage, its source and amount are detected, decision is to be made to adopt suitable remedial measures for its control. There are two approaches for seepage control, of which first is to reduce the quantity of seepage and second is to provide safe outlet to seepage water by proper drainage. The quantity of seepage can be reduced by interception of pervious zones as well as by providing barriers in various forms, such as grout curtains, cutoff trenches, and diaphragms. It can also be achieved by lengthening the path of seepage by providing impervious blankets. The type of remedial measure adopted depends on the location and quantity of seepage and on the type of the structure. Remedial measures generally adopted for concrete and masonry dams are foundation grouting, body grouting, pointing, cable anchoring, concrete / mortar jacketing, geo-membrane lining, steel jacketing, guniting, etc. Grouting aims at filling of cavities / fissures with selected material to impart water tightness and strength. In embankment dams, increasing the path of seepage drastically decreases the potential head of water, thereby preventing the downstream end from instability and piping. Cut off trenches are constructed to reduce reservoir seepage. Seepage control is also carried out using filters and drains to facilitate safe and quick seepage of water, however, optimum design of these structures should be ensured.

For canals, seepage control measures include lining with different materials viz. earth, concrete, bricks, tiles, etc. The conventional lining methods have undergone transformation during the years with emerging new technologies and innovative materials. Application of geosynthetic materials for lining is one such innovation which ensures speedy and effective remedial measure against seepage. Off late, application of geosynthetics for seepage control is gaining wide popularity due to their inherent advantages as lining materials for hydraulic structures.

Detection of seepage, its timely and efficient control is a skilled task. This should be based on sound background of study and analysis as regards to the occurrence of seepage, its location and quantity. It is necessary to identify the defects in hydraulic structures which are susceptible for seepage by applying advanced and integrated methods for assessment of seepage. The cost of these investigations may not be more than 10% of total cost of repair of the structure. Control measures should be adopted to mitigate seepage so as to avoid future consequences. Multidisciplinary techniques such as geological and geotechnical methods, dam instrumentation, geophysical methods, tracer techniques, nuclear logging and mathematical modeling for monitoring, detecting, analyzing seepage should be made use of effectively to arrive at an optimum solution for seepage problems. Practical utility of each of these methods has also been highlighted in related case studies. It is evident that effective application of these techniques not only helps to identify the source and cause of seepage but also to adopt suitable remedial measures for mitigation of seepage through hydraulic structures.

Owing to their vast experience in conducting seepage studies involving multidisciplinary approaches comprising of field studies, laboratory investigations, data interpretation and analysis and advising on the remedial measures; authors have, over the decades, developed an expertise in providing cost effective and viable solutions for seepage related problems. Although a number of methods are available, identification of causes of seepage and implementing suitable remedial measures is a challenging job for construction engineers based on site specific scenario. A number of related case studies describing their objectives and methodologies adopted have been given with an aim that these would serve as a guide to construction engineers to resolve problems faced at site.

In India, many dams are ageing and have various structural deficiencies and short comings in operation and monitoring facilities. To reduce the risk of failures, regular inspections are necessary to identify the defects and whenever severe deficiencies are observed, comprehensive remedial measures are required to be undertaken. To summarize, it can be stated that timely adoption of appropriate seepage monitoring, detection and analysis measures using conventional and non-conventional techniques and appropriate repair methodologies for rehabilitation of structures will surely lead to safe functioning of the hydraulic structure throughout its design life.

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