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जल संसाधन, नदी विकास और गंगा संरक्षण मंत्रालय

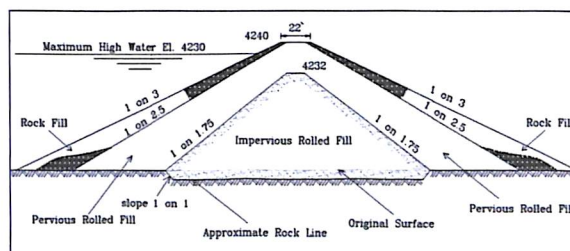
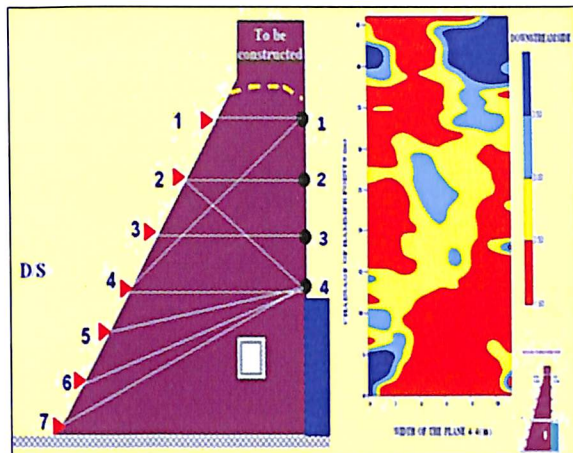


Technical Memorandum
on

DAM SAFETY
&
REHABILITATION

Edited by

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CWPRS
R8394



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DAM SAFETY AND REHABILITATION

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Director : S. Govindan

PREFACE

Dams are constructed by making huge investment for development of water resources for socio-economical development. In spite of taking proper care in planning, design and execution stages, there are incidences of distress in dams. As such, it becomes essential to diagnose dam distress and causes to ensure dam safety by using several possible techniques to arrive at the most suitable controlling measures to rehabilitate the dam. There are reported incidences of catastrophic dam failures all over the globe. To reduce the risk of failure regular monitoring and analyzing defects / distresses by applying advanced methods helps in deciding appropriate and economical remedial measures can be decided pertaining to the type of structures.

Central Water & Power Research Station, Pune, is a premier hydraulic research institute offering wide range of R&D services in problems related to dam safety and rehabilitation. For the past few decades, CWPRS has developed expertise in providing cost effective and viable solutions for dam safety and rehabilitation 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 safety and rehabilitation of gravity and embankment dams. Several case studies are also provided for the purpose of illustration.

This document is organized into five chapters, with Chapter I giving a general introduction about the various aspects of dam safety and rehabilitation, the basics of distress and its causes, consequences of distress in concrete, masonry and embankment dams. Chapter II deals with monitoring and assessment of dam safety which includes visual inspection, dam instrumentation, seismic tomography, borehole logging, tracer techniques, laboratory determination of engineering properties, alkali aggregate reaction and stability analysis by FEM.

Chapter III and IV deals with the safety methods for rehabilitation of gravity dams and embankment dams along with case studies respectively. Chapter V summarizes the important aspects for dam safety and rehabilitation like first detecting the dam seepage using different methods and then employing suitable remedial measures to mitigate the seepage, highlighting importance of safety aspects of dams.

The document is expected to be of great help to practicing engineers, researchers, scientists, consultants and managers of Water Resources projects to mitigate problems related to dam safety and rehabilitation.

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CHAPTER – 1

INTRODUCTION

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1.0 PREAMBLE

Dams are constructed to store large amount of water for irrigation, power generation, flood control, etc. A huge amount of financial investment is made in planning, designing, construction, operation and maintenance of dams for storage of water to meet the needs of water supply, irrigation and hydropower for socio economic development. Safety of dams is very important for safeguarding the national investments and the benefits derived. An unsafe dam also constitutes a hazard to human life and property in the downstream reaches. As such proper analysis, diagnosis of dam distresses and their corresponding causes are essential to ensure dam safety and to adopt appropriate remedial measures to rehabilitate the dam.

Most of the dams in India were constructed during the period when knowledge about dam construction and technology was limited. Now guidelines have been developed based on state of art technology, in terms of design, construction and maintenance of dams; need arises to rehabilitate dams following a check of their compliance with current standards and also due to distresses resulting from ageing, foundation failure, seepage, heavy floods, earthquakes, etc. In addition, for augmentation of storage capacity, it is necessary to increase height of the dam which necessitates rehabilitation of the structure.

Undertaking dam safety and rehabilitation measures is a skilled task. This should be based on sound background of study and analysis as regards to the occurrence of distress, its location and amount. It is necessary to identify the defects in dams which are susceptible for threatening the safety of the structure by applying advanced and integrated methods. 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 rehabilitate dams 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 distresses in dams should be made use of

effectively to arrive at an optimum solution for dam rehabilitation 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 the problem but also to evolve suitable remedial measures for attaining stability of dams.

1.1 CAUSES OF DISTRESS IN DAMS

The possible causes of deterioration of the structure should be understood before taking up the repair / restoration work failing which either the repairs will become ineffective resulting in wastage of money or sometimes lead to further damage necessitating additional expenditures. Usually deficiency in design and deficiency in construction are the two prime factors responsible for distress. The probable main causes of distresses are described with respect to type of dams i.e. 1) Concrete & masonry dams & 2) Embankment dams.

1.1.1 Concrete Dams

Concrete is a composite material that consists of an inorganic hydraulic binding medium (cement) embedded with fine aggregate (typically sand) and coarse aggregate (typically gravel), water and admixtures. The rate of strength development and the final strength depend on many factors including mix design, water/cement ratio, placement procedure, and aggregate and curing conditions. Cracking, spalling and disintegration are the three basic symptoms of distress in concrete structure. Concrete starts out as a fluid, low strength material and undergoes physical and chemical changes as it gains strength and hardens. Due to low strength in the pre-hardened plastic state, concrete is susceptible to cracking for small stress. Even in its full hardness, the tensile strength of concrete is only about 10% of its compressive strength, making it less resistant to tensile stress. Cracks in concrete may be classified into those that develop before hardening (plastic) and those after hardening.

Cracks may be classified by direction, width, and depth. Surface cracks consist of a roughly hexagonal pattern of short cracks indicating that the concrete surface had more restraint than the concrete interior or substrate.

Before attempting crack repair, the cause of the crack or the source of the stress must be identified. Epoxy resin injection is a common way of repairing structural cracks. Epoxy

has excellent cohesive strength and provides a repair with strength comparable to or greater than that of the concrete.

1.1.2 Masonry Dams

The most common deterioration of masonry dam is in the form of seepage ranging from minor to excessive. The seepage through masonry structures is mainly attributed to improper cement mortar ratio, type of cement, poor quality of stones, stiffness in the joints, in-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 seepage due to thermal cracking in masonry dam can be ignored which is more evident in concrete dams and is detrimental to structure. The art of placing of mortar in joints and packing joints is most important factor governing quality of joints with respect to seepage.

The main problems posed by seepage are: Loss of Storage, Blocking of drains, Structural Safety, Aesthetics etc. The probable main causes of distress in Concrete & Masonry Dams in different stages are given below:-

Design stage

- Inadequacy of site investigations
- Lack of due attention to data of hydrological, geological, geophysical and geotechnical investigations. These parameters have a direct impact on future operation and life of structure.
- Poor mix design
- Poor provision of shrinkage and movement
- Insufficient reinforcement resulting into overstresses and surface cracking
- Inadequate water seal provision at joints resulting into excess leakage of water through joints
- Undersized section

Construction stage

- Poorly graded aggregates in mixes
- Use of weak aggregates
- Excess water and corresponding segregation resulting in porous concrete and leakage

- Excessive use of cement producing high heat of hydration
- Improper concrete mixes, placement and curing
- Poor surface (joint) preparation resulting into weak planes and leakage
- Substandard shuttering resulting in offsets, bulging of surfaces, which play an important role in cavitations damage
- Improper shaping of foundation excavation like leaving some sharp steps resulting in stress concentration
- Improper shaping of transition and flow surfaces leading to cavitational damage.

Volume changes

- Change in moisture content result in cracking due to drying shrinkage
- Thermal stresses which are of tensile nature, developed due to temperature differential between interior and the outside surface of the structure causing cracking
- Temperature change in concrete due to solar radiation

Reaction of concrete constituents

- Reaction between alkalis of cement and silica from aggregates results in excessive expansion causing premature deterioration of concrete in form of cracks. This reaction is known as alkali aggregate reactivity. Hence, care should be taken at the time of geological investigations of rocks since they may contain reactive minerals like opal, chalcedony, zeolite etc.
- Solubility of cement hydration products in water makes hardened cement in concrete to deteriorate by dissolution and ultimately removal of the compounds responsible for strength of crystalline contacts in it. Since calcium hydroxide is relatively soluble ingredient of hardened Portland cement, its deterioration is called '**lime leaching**'. A concrete surface covered with leached material indicates that the destruction is going on deep inside the structure.
- The general requirement of water for concrete mix and curing is that it should be fresh, clear, free from excessive sodium alkalis, organic matters or other deleterious substances. But sometimes it may contain impurities like acids, sulphates etc. due to mixing of industrial wastes, manures etc.

Other causes

- Uneven foundation settlement

- Unforeseen action like heavy floods, earthquake of large magnitude etc.
- Excessive uplifts
- Corrosion of reinforcement
- Abrasive, impact, cavitation action of water associated with partially completed structures
- Improper action of spillways causing scour very near the structure which could result in undermining of the foundation.
- Stress concentrations around openings
- Ageing of materials with time
- Inadequate care and maintenance after construction

Energy Dissipation Devices

In case of dams, the common type of dissipation devices are hydraulic jump type stilling basins, roller buckets, flip buckets, slotted buckets, apron etc. The reasons for their distressful behavior are:-

- Unanticipated tail water conditions
- Non-adherence to stipulated gate operation schedule
- Return and cross currents
- Inadequate dissipation action
- Overlooking of design norms
- Invert level of bucket below the river bed level
- Hydraulic jacking
- Abrasive action
- Cavitations
- Hydrodynamic uplift

1.1.3 Embankment Dams

Embankment dams are defined as those constructed primarily of natural materials of earth, namely soil and rock. The principal vulnerability of an embankment dam is that it may damage or even fail if insufficient height or spillway capacity allows overtopping and erosion of the dam, or if uncontrolled seepage results in internal erosion of the embankment and its foundation. Embankment dams should be designed and constructed specifically for the conditions at a particular site. Failure to properly address one or more of these factors could have implications with regard to safety of a dam.

Earth and rockfill dams are subjected to seepage through embankment, foundation and abutments. Seepage, if excessive can result in failure of a dam, if, it begins to transport soil particles from body of the dam or from its foundation. The transport of soil materials is responsible for internal erosion and the process is responsible for the failure of dams due to piping. In general, the failure caused by internal erosion is influenced by the fact that the dams have no positive preventive measures to preclude internal erosion through the embankments, at the foundation contacts with the embankments, or within the foundation. If there is no particle movement through seepage water, the structure may be considered as safe. However, the excessive seepage attributes to functional failure of dam. The uplift pressures, instability of downstream slope, piping through the embankment and / or foundation, and erosion of material migration is associated with seepage.

1.2 SYMPTOMS OF DISTRESS

The various symptoms of deterioration/distresses in dams are described below:-

1.2.1 Concrete & Masonry

In concrete hydraulic structures, the three basic visual symptoms of distress in concrete are cracking, spalling and disintegration whereas in masonry structures leakage through the joints is the main visual symptoms of distress. Cracking is the unplanned discontinuity resulting from restrained movement. Cracks may be vertical, horizontal, diagonal, longitudinal, transverse hairline, random etc.

Moisture absorption- Masonry stones may absorb water from the surrounding mortar or from stored water resulting in weakening of the structure and producing dampness

Temperature effects- During the construction phase of masonry dams, the reservoirs were completely emptied by the end of each season resulting in exposure of mass of the dam to full effect of sunrays and maximum expansion. Thus in dams without the expansion joints considerable movements were observed. After the first rainfall, there is a rapid fall in the temperature affecting the surface layer of masonry. This alternate wetting and drying and temperature gradients cause damage to the masonry and assist seepage of water into the masonry.

Leaching- Leaching is a process wherein the seepage water enters pores of mortar and takes away soluble particles thus increases porosity. Leaching is dominant if water is acidic and if the degree of solubility of constituents of cement is more. The phenomenon is more obvious in masonry dams than in concrete dams. 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. Leaching is seen in the galleries in the form of tough white deposits at the joints because of removal of lime from water of the masonry.

Excessive uplift pressures - Blocking of drainage holes and gradual increase of water from reservoir into masonry leads to uplift and fissuring which favors further seepage of water.

Construction defects - During construction, floods to pass over during the rains, this could result in weakening of a few masonry layers, which led to some seepage across the junctions of old and new masonry for want of proper cleaning and bonding.

Earthquakes / floods - Acceleration from an earthquake or flood exerts adverse stresses.

Contraction joints - Joints become source of seepage in masonry either due to rupture of water seals or due to development of cracks at the interface between masonry and concrete.

1.2.2 Embankment Dams

Embankment dams are subject to several different types of distresses. These are explained in detail below.

Seepage (Controlled or Uncontrolled)

All embankment dams pass water through the embankment and foundation materials. Passage of water through the embankment and foundation materials is called seepage. Uncontrolled seepage through embankment dam and its foundation reduces embankment stability by increasing the actuating forces and decreasing the resisting forces. Seepage becomes a problem when embankment or foundation materials are moved by water flow or when excessive water pressure builds up in the dam or its foundation. Uncontrolled seepage

can be a problem and is a major cause of embankment dam failure. Problems due to seepage can be divided into following three categories:

Instability - Seepage causes stability problems when high water pressure and saturation in the embankment and foundation soils cause the earth materials to lose strength. If embankment seepage comes close to or emerges on the lower downstream slope very often the seepage will cause sloughing, shallow slides or even deep-seated slides.

Piping - Piping occurs when reservoir water moving through the pores of the dam or foundation soil exerts a tractive force on the soil particles through which it is flowing, sufficient to remove them at the seepage exit point. This erosion progresses in an upstream direction forming a "pipe" through the dam or foundation. The pipe continually enlarges as erosion removes soil adjacent to the pipe. Usually the overlying embankment eventually collapses causing a breach of the dam.

Internal Erosion - Internal erosion may appear to be the same as piping because in either case soil particles are moved by the erosive force of flowing water. A failure resulting from internal erosion may look very similar to a failure caused by piping. However, the mechanisms of piping and internal erosion failures are very different. Internal erosion occurs when seepage flows along established pathways, such as:

- Along cracks or other defects in the soil or bedrock in the cross-section,
- Along boundaries between soil and bedrock, or
- Between soil and appurtenant structures.

Cracking

Another serious type of distress in earth dam is cracking. Cracks are linear separations that appear in the crest or slopes of the dam separating previously intact embankment material. Cracking of the impervious core results into failure of an earth dam by erosion, piping, breaching, etc. Cracking occurs due to foundation settlement and/or differential movements within the embankment. Differential moments may occur due to unsuitable or poorly compacted fill materials, different compressibility and stress-strain characteristics of the various fill materials; and variation in thickness of fill over irregularly shaped or steeply

inclined abutments. Cracking in an embankment dam falls into the following three major categories:

i) Transverse Cracking – Transverse cracking appears in a direction roughly perpendicular to the axis of the dam. If these cracks extend into the core below the reservoir level, they are especially dangerous because they could create a path for concentrated seepage through the core causing very rapid erosion and eventually breach of the dam. Transverse cracks usually appear on the dam crest near abutments. The presence of transverse cracking indicates differential settlement within the embankment or underlying foundation. This type of cracking frequently develops when:

- Compressible material overlies abutments consisting of steep or irregular rock.
- Areas of compressible material are in the foundation.
- Sections of the embankment have been excavated and replaced, such as for conduits

ii) Longitudinal Cracking - Longitudinal cracking occurs in a direction roughly parallel to the axis of the dam. Longitudinal cracking is an indication of:

- Uneven settlement between adjacent embankment zones of differing compressibility
- The beginning scarp of an unstable slope. In this case, the crack may appear arc-shaped.

Longitudinal cracks allow water to enter the embankment. When water enters the embankment the strength of the embankment material adjacent to the crack may be lowered. The lower strength of the embankment material can lead to or accelerate slope stability failure.

iii) Desiccation Cracking - Desiccation cracking is caused by drying out and shrinking of certain types of embankment soils. Desiccation cracks usually develop in a random, honeycomb pattern. Typically, desiccation cracking occurs in the crest and on downstream slope. The worst desiccation cracking develops when a combination of following factors is present :

- A hot, dry climate accompanied by long periods in which the reservoir remains empty
- An embankment that is composed of highly plastic soil, such as clay

Usually, desiccation cracking is not harmful unless it becomes severe. The major threat of severe desiccation cracking is that it can contribute to formation of gullies. Surface

runoff erosion concentrating in the desiccation cracks or gullies can result in eventual damage to the dam. Also, heavy rains can fill up these cracks and cause portions of embankment to become unstable and to slip along crack surfaces where the water has lowered the strength of embankment material. Deep cracks that extend through the core can cause a breach of the dam when the reservoir rises and cracks fail to swell rapidly enough to reseal the area.

Instability

Instability of the embankment is very serious. The primary visual indicators of instability are: Slides, Bulges, Seepage and Cracks. Other clues include cracking and misalignment of features (e.g., walls, guardrails, pavement stripes, appurtenant structures, or reservoir upstream contacts). Instability in an earthen dam can be of any of the following types :

i) **Slides** - Slide phenomena can also be termed as displacements, slumps, slips and sloughs. Slides can be grouped into two major categories: Shallow Slides and Deep-Seated Slides. Slides in an embankment dam may lead to:

- Obstruction of water conveyance structures and drains
- Larger, deep-seated slides.
- Surface erosion or maintenance problems.

a) **Shallow Slides** - Shallow slides in the upstream slope are often the result of an overly steep slope aggravated by rapid lowering of the reservoir. They pose no immediate threat to the integrity of the dam. Shallow slides in the downstream slope also indicate an overly steep slope. In addition, these slides may also indicate low strength or a loss of strength in the embankment material. Low strength or a loss of strength can be the result of poorly compacted material, and can also be caused by saturation of the slope due to seepage, infiltration of surface runoff, or a clogged drain.

b) **Deep-Seated Slides** - Deep-seated slides are serious threats to the safety of the dam. Occurrence of deep seated slide failure in an embankment dam can be confirmed by any of the following visual damages :

- **Well defined Scarp** - A scarp is a relatively flat area with a steep back slope.
- **Toe bulge** - A toe bulge is the lower portion of a deep seated failure and is produced by rotational or horizontal movement of embankment material.

- **Arc-Shaped Cracks** - Arc-shaped cracks in the slope are indications that a slide is beginning. This type of crack may develop into a large scarp in the slope at the top of the slide.

ii) Lateral Spreading - Excessive settlement may result in lateral spreading and bulges, which may be accompanied by parallel longitudinal tension cracks on the slope. Bulges are most evident at the toe of the dam. A toe bulge due to lateral spreading may mean that there has been some loss of freeboard.

iii) Sinkholes - Sinkholes are formed when the removal of subsurface embankment or foundation material causes overlying material to collapse into the resulting void. The presence of a sinkhole may indicate that material is being or has been transported out of the dam or foundation through the process of internal erosion or piping. The decomposition of buried wood or other vegetative matter, and animal burrows can also cause sinkholes. Sinkholes are often associated with a karst foundation. Karst refers to a region characterized by distinctive features such as caverns, "lost" rivers, large springs, barren uplands, and thin soils. Such topography usually exists in areas of limestone bedrock which has formed these features due to solutioning and weathering of the bedrock. Sinkholes usually have steep, bucket-like sides.

iv) Depressions - A depression is a form of settlement in the embankment or foundation that is less serious than a sinkhole. Localized depressions have gently sloping, bowl-like sides. Depressions are caused by:

- a) **Erosion** - Wave action against the upstream slope that removes embankment fines or bedding from beneath riprap may form a depression as the riprap settles into the vacated space.
- b) **Localized settlement** in the embankment due to poor compaction or foundation due to compressible materials.
- c) **Loss of sub-surface material** through the decay of vegetative matter, or through internal erosion or piping. Some areas that appear to be depressions may be the result of improper final grading following construction. Sometimes a way to distinguish between depressions and sinkholes is to look at their profiles. A localized depression may be the initial manifestation of a sinkhole to follow.

1.3 DIAGNOSIS OF DISTRESS

Diagnosis of causes of distress is the process of eliminating possibilities until some conclusion is drawn. For diagnosis, the interrelationship of symptoms and potential causes must be known.

1.3.1 Concrete and Masonry dams

Knowing the symptoms in Concrete & Masonry dams, the check for the related causes can be made as given below which serves as a guide.

Sl.No.	Symptom	Potential Causes
1	Cracking	Construction operation, Shrinkage stresses, Temperature stresses, Chemical reaction, Poor design details, Overstresses due to external load, temperature and shrinkage, Moisture absorption
2	Spalling	Corrosion of reinforcement, Under design, Defective design details, Weathering
3	Disintegration	Unsound material, Weathering condition, Abrasive action, Change in constituents
4	Swelling (Growth)	Chemical reaction, Absorption of moisture by concrete, Rise of temperature in concrete mass
5	Popouts	Internal pressure
6	Pitting	Cavitation, Corrosion, Localised disintegration
7	Distortion	Buckling (failure by lateral or torsional instability), Curling (distortion of original linear plane to curved shapes), Warping (deviation from original shape due to temperature/ moisture differential), Faulting (vertical settlement of a member adjacent to joint/cracks), Settling (lowering of elevation due to imposed loads/shrinkage/displacement of support), Tilting (movement of a structure)
8	Erosion	Abrasion, Cavitation
9	Seepage	Corrosion of reinforcement, Exudation (discharge of viscous gel like material from pores or opening), Efflorescence (deposition of salts on the surfaces), Incrustation (hard crust or coating formed on surface)

1.3.2 Embankment Dams

Diagnosis of distresses in embankment dams can be done by regular inspections. Direct observation is one of the most effective methods of detecting deteriorations in

foundation and dam body. Embankment dam should also be monitored and examined for cracks, leakages, saturated areas or wet spots, springs, sinkholes, evidence of piping, erosion, excessive growth of vegetation, frost, heave, crest alignment, bulging or depression of slopes and berms, animal burrows, and deterioration of rip-rap or other slope protection materials.

Periodic and systematic measurements in respect of seepage, phreatic level, uplift, pore pressure, water turbidity, horizontal displacement, vertical displacement and rainfall may also be used in detecting the deterioration in foundation and dam body. Test results on soil samples from dam and foundation and chemical and physical analysis of seepage can be used to investigate the deterioration in respect of shear strength, seepage and internal erosion of the foundation and dam body.

1.4 OVERVIEW OF DAM SAFETY

There are reported incidents of catastrophic dam failures in India and other parts of world due to several causes. To reduce the risk of failures, regular health inspections are necessary to identify the defects by applying advanced and integrated methods. Investigations for dam safety and rehabilitation 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. The cost of investigations may not be significant when compared to the total cost of repair of the structure. Control measures should be adopted to mitigate problems so as to avoid future consequences. As such, timely adoption of monitoring, detection and analysis measures using conventional and non-conventional techniques and appropriate repair methodologies for rehabilitation of dams should be undertaken. This will ensure safe functioning of dams throughout its design life.

The present technical memorandum is aimed at providing comprehensive information on various practical and theoretical aspects of Dam safety and rehabilitation. Conventional and nonconventional methods of investigations are discussed along with case studies. The methods of analysis and remedial measures are also highlighted along with appropriate case histories. Nonconventional techniques involving tracer techniques, borehole logging and

remote sensing techniques play a key role in carrying out investigations for dam safety. Borehole logging investigations can provide in-situ assessment of engineering properties, potential seepage pathways, lithological variations and solution activity. Geophysical methods, in general, address investigating and monitoring structural parameters, mapping of geologic features and monitoring of seepage.

Mathematical modelling by analytical and numerical methods (FEM and FDM) provides a great tool for assessment of stability of dams and foundation. With the advent of high speed computers and advanced softwares, these tools are being immensely popular. Various parameters such as settlement, uplift, displacements, 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.



CHAPTER II

MONITORING AND ASSESSMENT OF DAM SAFETY

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2.0 INTRODUCTION

The dam safety procedure is followed by a general plan of physical inspection of dam, monitoring dam with instrumentation and surveillance, external and internal examination of dams, measurement of displacement settlement, seepage, uplift pressure water level etc. It is essential to take periodic review of these observations. Based on these results, action plan for rehabilitation of dam is decided. As such monitoring assessment of distress in the dams is very crucial stage in dam safety. The compilation of data interpretation of result on occurrence of deterioration and distresses in dams will provided for recommendation of appropriate action for rehabilitation. Apart from these, monitoring and assessment of distress will able to provide emergency action plan for avoiding consequences. Monitoring and assessment of dam safety is adopted by employing following method:

- i) Physical inspection of dam to observe damages.
- ii) Dam instrumentation and surveillance
- iii) Seismic tomography for assessing weak zones
- iv) In- situ determination of engineering properties by nuclear and sonic logging
- v) Dam seepage investigations by tracer techniques.
- vi) Laboratory determination of engineering properties
- vii) Alkali aggregate reaction for identification reactive mineral
- viii) Stability analysis of dam by FEM

2.1 PHYSICAL INSPECTION OF DAM

Physical inspection of dam is generally made periodically to notice the external damages of dam. These damages in the form of cracks, seepage tilting of dam, opening of joints, settlement at several location of operation and foundation gallery, upstream and

downstream of faces of dam. The extent these phenomenon noticed will provide the preliminary idea of overall health of dam.

Visual inspection should be made in two phase since the scope and completeness of each investigation depend upon the availability and suitability of engineering data, the validity of original design assumptions and the physical condition of the dam.

First Phase

First Phase is an inspection to assess the general condition of the dam and determine the need for any additional engineering investigations and analyses. It would consist of a visual examination of the dam and a review of available engineering data, including operating records. Damages in the form of cracks, seepage, tilting of dam, opening of joints settlement etc., are generally observed at several locations such as operation and foundation galleries, upstream and downstream faces of the dam. The physical inspection gives preliminary idea about the overall health of the dam. The type of further investigations to be carried out is decided based on findings of physical inspection of the dam. It is not intended that costly explorations or analyses would be performed during a first Phase inspection (CWC Publication No.21/87).

Second Phase

Second Phase investigations are performed where the results of the first Phase inspection indicate the need for additional investigations and studies. Second Phase would include, as required, all additional visual examinations, measurements, foundation exploration & testing, materials testing, hydraulic & hydrologic analyses and structural stability analyses deemed essential to evaluate the safety of the dam. The first Phase investigation will develop an assessment of the general condition with respect to safety of the project based upon available data and a visual inspection, determines any need for emergency measures and conclude if additional studies, investigation and analyses are necessary and warranted. A review will be made of pertinent, existing and available engineering data relative to the design, construction and operation of the dam and appurtenant structures, including electrical and mechanical operating equipment and measurement from inspection and performance instruments and devices and a detailed systematic visual inspection will be performed of those features relating to the stability and operational adequacy of the project. Based upon the findings of the review of engineering data and the visual inspection, an evaluation will be made of the general condition of the dam, including where possible the assessment of the hydraulic and hydrologic capabilities and the structural stability.

To the extent feasible the requisite engineering data such as Construction drawings indicating plans, elevation and sections of the dam, appurtenant structures including the details of the discharge facilities such as outlet works, spillways and operating equipment, Description of Drainage basin – Drainage area and basin runoff characteristics, Design flood-design assumptions and analysis, storage of flood control zone, Spillway capacity and flood routing criteria, geology and foundation, Construction History-including diversion scheme, construction sequence, construction problems, alterations, repairs, Operation and regulation plan under normal conditions and during floods and other emergency conditions. Flood Warning Systems, Operation record-experiences during past major floods, Stability and stress analysis of the dam, spillway and appurtenant, structures and features include the assumed properties of materials and all pertinent applied loads. Instrumentations and records of performance observations relating to the design, construction and operation of the dam and appurtenant structures, should be collected from existing records and reviewed to aid in evaluating the adequacy of hydraulic and hydrologic capabilities and stability of the dam. Where the necessary engineering data are not available, inadequate or invalid, a listing should be made of these specific additional data deemed necessary by the engineer in charge of the investigation and included in the first Phase report. The field inspection of the dam, appurtenant structures, reservoir area and downstream channel in the vicinity of the dam should be conducted in a systematic manner to minimize the possibility of any significant feature being overlooked. A detailed checklist should be developed and followed for each dam inspection to document the examination of each significant structural and hydraulic feature, including electrical and mechanical equipment for operation of the control facilities that affect the safety of the dam. The checklist can be altered to suit the characteristics of the particular project or structure. The “Special Items” category listed at the end of each major heading as for listing those items which are specific to the structure but have not been included in the general checklist items.

Particular attention should be given to detecting evidence of leakage, erosion, seepage, excessive wetness or slushiness in the areas downstream of dam, presence of sand boils, change in water table conditions downstream, slope instability, undue settlement, displacement, tilting, cracking, deterioration and improper functioning of drains and relief wells. If the dam is instrumented, look for any evidence of excessive pore pressure conditions. It may be examined whether there had been any encroachment on the free board allowance made in the design. The adequacy and quality of maintenance and operating

procedures and operation of control facilities should be examined as they pertain to the safety of the dam. Photographs and drawings should be used freely to record conditions in order to minimize descriptions.

INSPECTION SCHEDULE

Frequency of inspections and monitoring data collection depends on the rate of system changes. Routine inspections of dam are on a weekly or monthly basis. Frequency of inspection should be change depending on the season. In wet or snow melt season dam inspection should be daily to weekly. During dry season dams inspection should be monthly. By site personnel, Engineer inspections are on a semi-annual to annual basis. Designer should inspect the dams after every earthquakes and floods. A dam safety review panel inspect the dam after every 5 to 10 years.

2.2 SURVELLANCE THROUGH INSTRUMENTATION

An effective dam safety-monitoring program is essential for dam owners to manage the risks associated with the operations and maintenance of a dam. Use of instrumentation can improve the dam owner's ability to monitor the on-going safe performance of the dam by providing more comprehensive and timely information (USSD). An instrumentation and measurement system are installed for this purpose. Instrumentation and measurement systems on dams provide data for evaluating the dam's performance compared to design and an early warning of changes that could affect the integrity of the dam. In addition, it increases knowledge of dam behavior and assist in investigations and diagnosis of abnormal dam performance (ASCE, 2000).

In our country, mainly dams are owned by states and other agencies; the safety of these dams is the principal concern of the state agencies and involves various aspects of investigations during different stages of dam i.e planning, design, construction, operation and maintenance (CWC, 1986). Proper instrumentation and its subsequent analysis would provide vital information to detect any unwarranted developments in dam behavior to the applied loads or any undetected foundation surprises. It is felt that, there are many historical cases of dam failures where early warning signs of failure might have been detected if a good dam safety-monitoring program had been in place. As Indian dams are getting older and older and need proper revaluation of their safety with present standards instrumentation analysis whether already installed are provided during rehabilitation process would help the dam owners. Constantly listening to the dam through instruments and visual observations, correlating the observed behavior with construction activities and revising the design or

correcting the expectations accordingly is the key to a successful Monitoring Program during major rehabilitation work (Elena Sossenkina et al.).

2.2.1 Need for an Instrumentation and Measurement System

The need for instrumentation and monitoring can be grouped as (ICOLD bulletin 60-1988):

- Measurement during Construction and First Filling
- Measurement during Operation
- Measurement for Technical and Scientific Purposes

2.2.2 Parameters Measured

As per IS: 7436-1976, *Guide for types of measurements for structures in rivervalley projects and criteria for choice of measuring instruments, Part I (Concrete and Masonry Dams) & Part II (Earth and Rock fill)* mentions the parameters to be measured in detail separately. In this section the parameters are discussed in common along with their engineering principles in short. The parameters measured in different type of dams are tabulated in Table 2.1.

Table – 2.1 Typical Measurements Required for different type of Dams

Type of Instrument	Embankment		Concrete Gravity		Integral Powerhouse	
	Proposed	Existing	Proposed	Existing	Proposed	Existing
Visual Observation	√	√	√	√	√	√
Reservoir Level	√	√	√	√	√	√
Tail water Level	√	√	√	√	√	√
Drain flow, seepage and leakage	√	√	√	√	√	√
Pore/Uplift Pressure	√	√	√	√	√	√
Surface settlement	√					
Surface alignment	√			√	√	√
Internal Movement	√	√	√	√	√	√
Join/Crack Displacement	√	√	√	√	√	√
Foundation Movement	√	√	√	√	√	√
Temperature Movement	√	√	√	√	√	√
Seismic Loads	√	√	√	√	√	√

The different parameters for type of dam is discussed as under;

Water Level and Pressure

Water pressure is a general term that includes pressure within a reservoir or other body of water, porepressure, and uplift pressure. Water pressure within soils and within concrete is commonly referred to as pore pressure. Water pressure acting upward on the base of concrete dams is commonly known as uplift pressure.

Water level and water pressure are directly related by the depth below the water surface or phreatic surface. Relatively high excess pore water pressures may develop in impervious zones and compressible foundation strata during construction of embankment dams as the height of the dam increases. The inability of the dam or foundation to maintain effective strength during construction may lead to deformation or, in extreme cases, slope or bearing capacity failures.

Seepage and Leakage

An interstitial movement of water through a dam, the foundation, or the abutments is called Seepage whereas flow of water through holes or cracks is termed as leakage. The primary factors influencing the amount of seepage and leakage are the same as that influencing pressure distribution. The amount of seepage or leakage is directly proportional to permeability and pressure. It is possible to have large flow with high pressure, large flow with low pressure, low flow with high pressure, or low flow with low pressure. Most of the factors that influence the amount of seepage or leakage do not change during the life of a project.

Movement

All structures move as the result of applied loads. Embankments settle and spread over time as the result of consolidation and secondary settlement of the dam and foundation from self-weight. Embankments also deform due to external loads produced by reservoir water, rapid drawdown, earthquakes, undermining, swelling clays, and piping.

Concrete dams deform due to internal loads such as pore pressure, cooling, and alkali aggregate reaction of concrete; and external loads caused by air and reservoir temperature,

solar radiation, reservoir levels, uplift pressure, wind, earthquakes, undermining, ice, overflowing water, swelling clay, and foundation settlement.

Three types of movement are observed in dams, they are:

- i) Surface movement
- ii) Internal movement
- iii) Joint movement

Stress

Direct measurement of stress developed inside the mass of concrete or masonry helps in watching the structural behaviour of dams and their foundations. Deflection of the diaphragm is measured by a Carlson-type transducer and is converted to stress. The instruments only measures compressive stress (IS: 7436(part II)-1976).

Strain

Earth pressures within fill and against concrete structures are commonly measured with earth pressure cells. These are also known as total pressure cells. They consist of two flexible diaphragms sealed around the periphery, with a fluid in the annular space between the diaphragms. Pressure is determined by measuring the increase in fluid pressure behind the diaphragm with pneumatic or vibrating-wire sensors. Earth pressure cells should have similar stiffness as the surrounding soil to avoid inaccurate measurements of in-situ stress caused by arching.

Temperature

Temperature measurements of a dam, foundation, or instruments, are often required to reduce data from instruments, increase precision, or to interpret results. Temperature is also commonly measured in concrete dams under construction to evaluate mix design, placement rates, and block and lift sizes; to time grouting of block joints; and to evaluate thermal loads.

2.2.3 Types of Instrumentation

Instruments are classified by the type of transducers used for observing and transmitting data. There are many technologies evolved in the development of instrumentation history. Major among them are generally classified as i) Resistance or Carlson type instrumentation, ii) Vibrating wire instrumentation and iii) Mechanical type instrumentation.

Water Level and Pressure

Following are the different type of instruments used measure water level and pressure

- Water Level Gages
- Observation Wells
- Open Standpipe Piezometers
- Closed Standpipe Piezometers
- Twin-tube Hydraulic Piezometers
- Pneumatic Piezometers
- Vibrating Wire Piezometers
- Bonded Resistance Strain Gage Piezometers

Typical foundation piezometer and pressure cells are shown in fig. 2.1

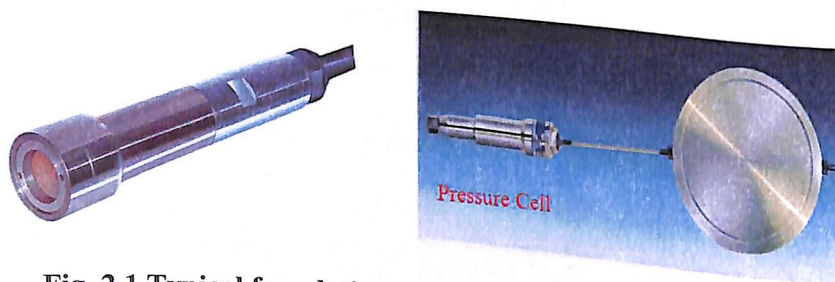


Fig. 2.1 Typical foundation piezometers and Pressure Cell

Seepage and Leakage

Seepage and leakage can be measured by

- Weirs
- Parshall Flumes
- Calibrated Containers

Movement

Some of different type of instruments used to measure different type of movements are listed below:

Surface Movement

- Level Surveys
- Alignment Surveys
- Triangulation and Trilateration

Internal Movement

- Plumblines

- Tilt meters
- Inclinometers
- Extensometers

Fig 2.2 shows typical plumb line, deflect meter and clinometers

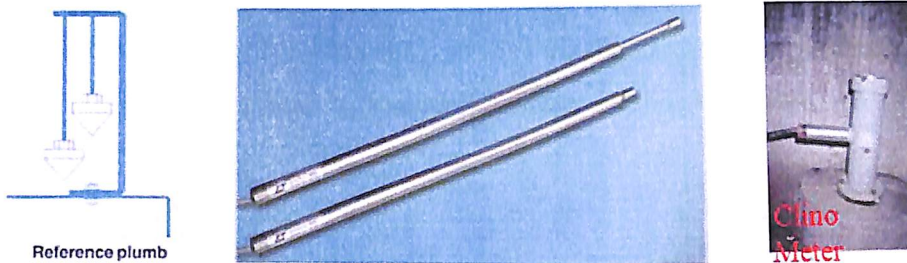


Fig . 2.2 Typical Plumb line, Deflect meter and clinometer

Crack and Joint Measuring Devices

Movement of one side of a crack or joint in a concrete structure relative to the other side of the joint or crack is commonly measured with reference points or crack meters. Grout or plaster patches can be used to evaluate whether or not movement is occurring. Reference points can be scratch marks in the concrete, metal pins, or metal plates on opposite sides of a joint or crack. The distance between the scratch marks is measured with a micrometer or dial gauge to determine movement. Sometimes three points are used in a triangle to measure both horizontal



Fig. 2.3 Typical Crack meter and joint meter

and vertical movement. Crack meters are commercially available devices that allow movement in two directions to be measured. A common device consists of two plastic plates. One plate is opaque and contains a grid. The other plate is translucent and contains a set of

cross hairs. One plate is fixed on each side of the crack or joint with the cross hairs set over the center of the grid. Movement is measured by noting the location of the cross hairs with respect to the grid. A variety of other crack meters including Carlson and vibrating-wire sensors, dial gages, and mechanics feeler gages may be used to measure movement of cracks. All these devices are simple to install and monitor. The accuracy and reliability varies depending on the details of the devices and measurements. Mineral deposits, iron staining, or efflorescence obscuring the instruments are a common problem if seepage or leakage flow is present. Some of the crack and joint meters are shown in Fig. 2.3.

Stress

Stress in concrete structures can be measured with total pressure cells designed to have stiffness similar to concrete. Fig. 2.4 shows

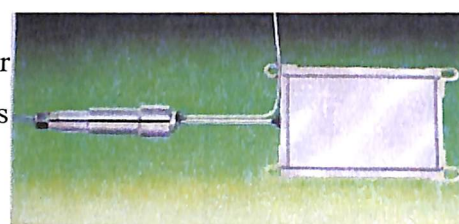
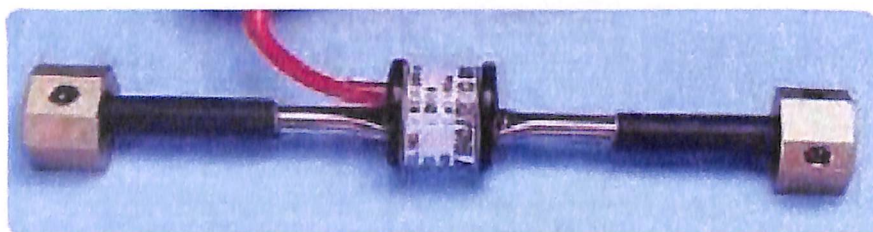


Fig. 2.4 Typical Stress meter

Strain

A variety of mechanical and electrical strain gages are used to measure strain in concrete structures. Some of the instruments are designed to be embedded in the dam during construction and others are surface mounted following construction. Strain gages are often installed in groups so that the three-dimensional state of strain can be evaluated, Fig. 2.5 shows a typical strain meter and strain spider.



Temperature

Temperature

meters or thermocouples.

Fig. 2.5 Typical Strain meter and Strain Spider

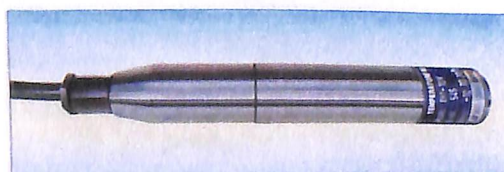


Fig. 2.6 Typical Temperature meter

2.2.4 Monitoring Program

A monitoring program includes responsibility assignments and procedures for data collection, reduction, processing, and presentation should be developed and documented.

Documentation

An instrumentation document should be developed that includes a discussion of the purpose of each instrument, expected ranges of data, threshold limits, manufacturers' literature, procurement and installation specifications, installation logs, calibration data and initial readings.

Maintenance and Calibration

A routine of regular maintenance of instruments, readout devices, and field terminals should be established. Detailed measurements and careful evaluation of data has little value, and may be misleading, if the data are inaccurate. The nature and frequency of calibration depends on the specifics of the instrumentation and should be developed on a case-by-case basis.

Monitoring Schedules

The schedules are considered to be generally applicable for all significant and high-hazard dams; however, since each dam is unique, the schedules should be applied using engineering judgement and common sense. Specific monitoring schedules should be developed on a case by-case basis.

Data Processing and Evaluation

The steps required to process and evaluate data, whether collected manually or automatically, are the same. Instrument data should be processed and evaluated according to the procedures established by the monitoring program. Accumulation of instrument data by itself does not improve dam safety or protect the public. Data collected manually should be recorded on the data sheets prepared as part of the monitoring program. Complementary data, such as air temperature, reservoir level, reservoir temperature, recent precipitation, and other information or observations that may be important in evaluating the instrumentation data should be noted on the data sheets. Most instruments require raw data to be converted into useable engineering units. The arithmetic calculations required to convert data are known as data reduction. The data reduction may be done in the field or office. It should be done under the supervision of the responsible engineer and it should be checked by someone other than the preparer to reduce errors. All reduced data should be summarized in graphical form. All

plots should include sufficient previous data to identify any long-term trends. Furthermore, the plots should be self-explanatory.

Data Interpretation

Data should be reviewed for reasonableness, evidence of incorrectly functioning instruments, and transposed data. Several checks for reasonableness can be made on all data. The magnitude of data should be near the range of previous data. Data that are significantly different may be incorrect. All data will have scatter from instrument error, human error, and from changes in natural phenomena such as temperature, wind, and humidity. The true accuracy of data will not be apparent until a significant number of readings have been taken under a variety of conditions. All data will follow trends, such as decreasing with time or depth, increasing with time or depth, seasonal fluctuation, direct variation with reservoir or tailwater level, direct variation with temperature, or a combination of such trends.

Dam Performance Evaluation

The purpose of instrumentation and monitoring is to maintain and improve dam safety. The data should be used to evaluate whether the dam is performing as expected and whether it provides a warning of developing conditions that could endanger the safety of the dam. All data should be compared with expected behaviour based on the basic engineering concepts. Variations from expected behaviour may suggest development of conditions that should be evaluated. All data should be compared with design assumptions. If no unusual behaviour or evidence of problems is detected, the data should be filed for future reference. If data deviates from expected behaviour or design assumptions, action should be taken. The action to be taken depends on the nature of the problem, and should be determined on a case-by-case basis. Possible actions include:

- Performing detailed visual inspection;
- Repeating measurements to confirm behaviour;
- Re-evaluating stability using new data;
- Increasing frequency of measurements;
- Installing additional instrumentation;
- Designing and constructing remedial measures;
- Operating the reservoir at a lower level; and
- Emergency lowering of the reservoir.

2.2.5 CWPRS Experiences

CWPRS has been involved in many dam monitoring projects in the form data monitoring and analysis of observed data C namely rack monitoring studies of Hirakud dam Orissa and Dharoi dam, Gujarat, Analysis of defelction data of Koyna dam, Maharashtra. Monitoring sluice gate rail tracks using distomat during rehabilitaion, Periodical Analysis and Interpretation of Data from Embedded Strain Gauges, Varahi H E Project, Karnataka. Recently CWPRS has taken up full pledge Analysis and Interpretation of Dam Instrumentation Data of concrete dams of Indira Sagar Dam, M. P and Omakreshwar Dam, M. P. The studies of the Analysis and Interpretation of Dam Instrumentation Data of Indira Sagar Dam, M. P, revealed the development of higher uplift in d/s of the dam. Further tracer studies from CWPRS confirmed the same and found out the source.

2.2.5.1 Analysis and Interpretation of Dam Instrumentation Data, IndiraSagar Dam, M.P.

The 653m long, 92m high Indira Sagar Dam concrete gravity dam (Fig. 2.7) is constructed across Narmada River in Madhya Pradesh. The dam consists of 27 blocks (1 to 3 and 25 to 27 forming non-overflow portion and block Nos.4 to 24 forming overflow (Spillway) portion). The reservoir, largest in India, with water storage of 12.22 Billion cubic meters a multipurpose project, with an overall hydropower generation of 1000 MW (8X 125 MW). To study post construction structural behaviour of dam, various frequency based vibrating wire type instruments have been installed in block 13 and 25 of the dam during construction (Fig. 2.8 and Table 2.2). Data from all the installed instruments are recorded by project officials every fortnight.

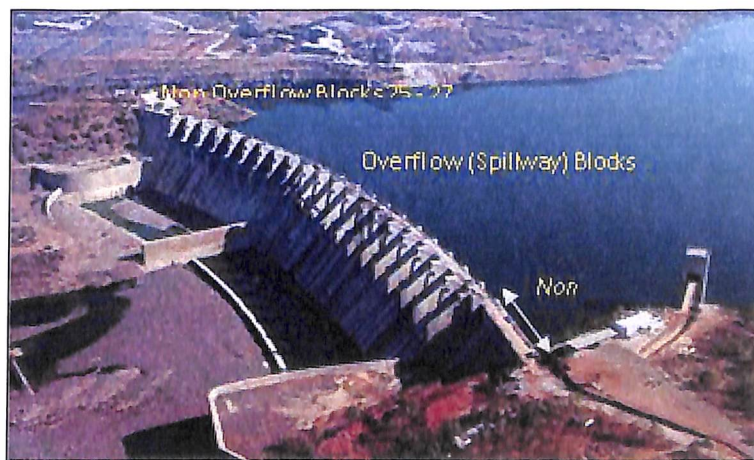


Fig. 2.7 View of Indira Sagar Dam

Table – 2.2 List of Instruments Installed in Indira Sagar Dam

Sr.No.	Type of Instrument	Nos. of Instruments installed	
		Non-over flow 24-25	Over flow Block No 13-14
1.	Foundation piezometer	08	11
2.	Uplift pressure pipe*	05	07
3.	Extensometer	01	01
4.	Reservoir Water level meter	01	01
5.	Pore pressure cell	04	07
6.	Joint meter	10	13
7.	Temperature meter	16	33
8.	Strain meter	08	05
9.	No stress strain meter	08	02
10.	Stress meter	07	07
11.	Clinometers*	02	03
12.	Plumb line*	01	01
13.	Seepage meter*	-	02

Analysis of data from the instruments has in general been made after applying necessary corrections, wherever applicable. In respect of strain meter data, necessary corrections have been applied to initial readings in consultation with project authority based on available installation records. The computed values based on measured data of “No stress strain meter” have been deducted from strain meter computed values to obtain net strain. The parameters have been plotted with time period and reservoir water level. Two Dimensional Stress analysis has been carried out by Finite Element Method with a view to obtain data on vertical stress, deflection, etc so as to facilitate comparison with respective observed values for different load combinations. Variation of water level with time is shown in Fig. 2.9 and variation of measured

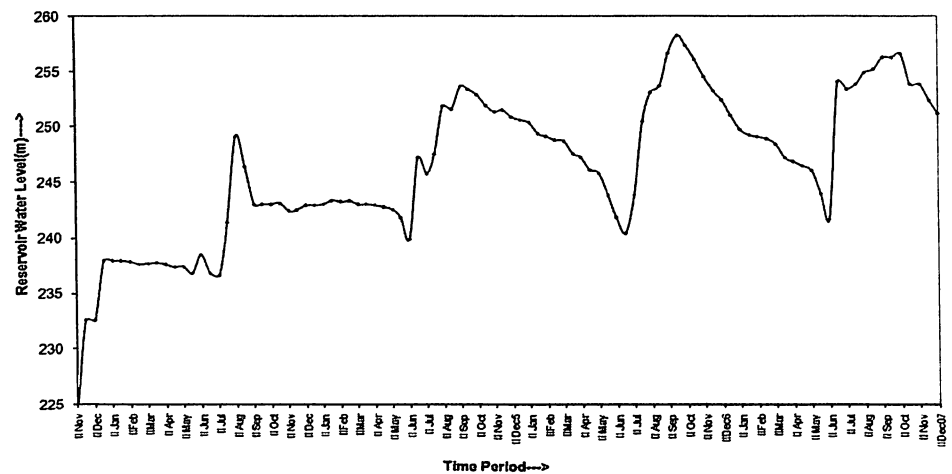


Fig . 2.9 Variation of water Level with time

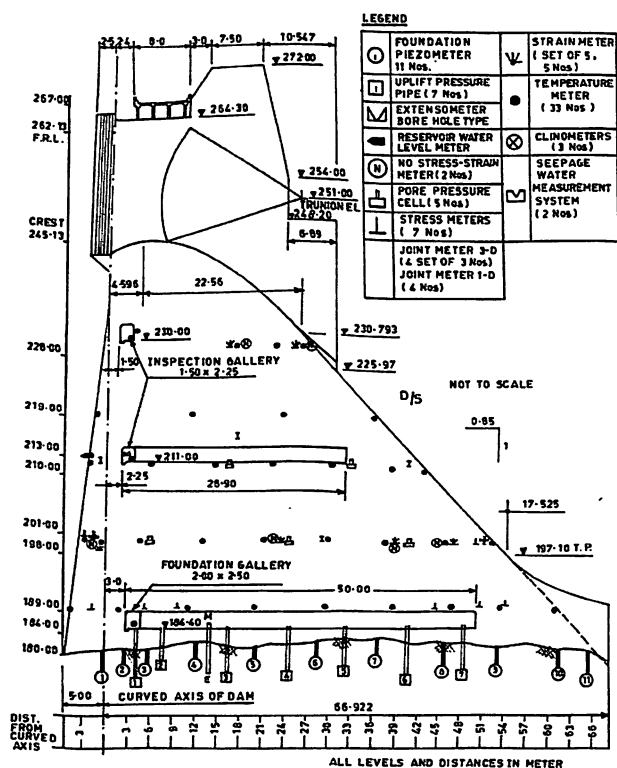


Fig. 2.8 Instrument Location In Block No. 25 and Block No. 13

uplift pressure at dam base shown in Fig. 2.10 and Fig. 2.11. From the Fig. 2.10 and Fig. 2.11 it can see the effectiveness of keeping galleries clean in reducing uplift pressure.

The measured uplift pressure is less than theoretically computed uplift pressure at all locations and poses no danger to the safety of the dam. However, in case of block no 25 uplift pressures is higher than theoretical value in downstream side (Fig. 2.11)

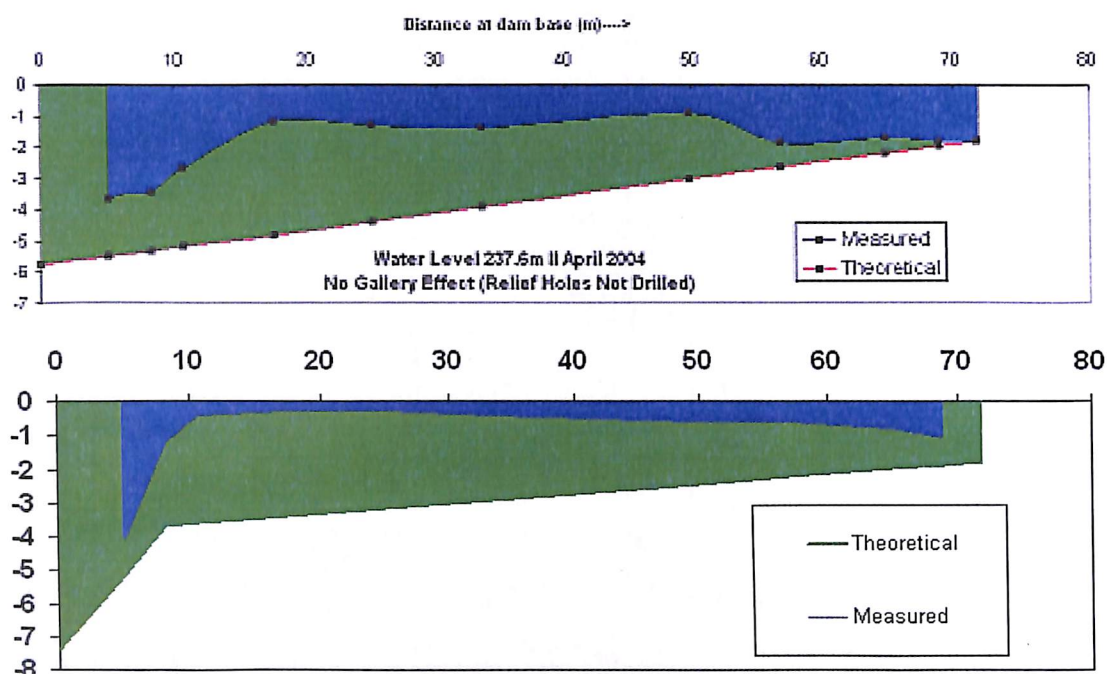


Fig. 2.10 Variation of uplift pressure at dam base in block 13

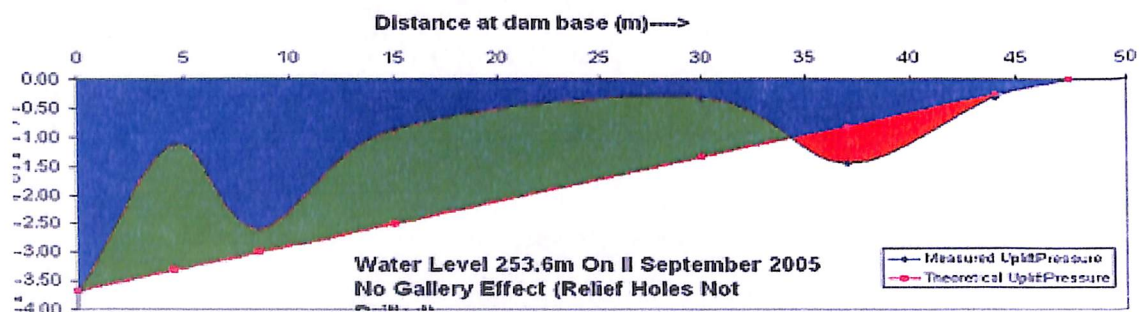


Fig. 2.11 Variation of uplift pressure at dam base in block 25

are attributed to cracks, honeycombing etc. Also change in leaking discharge of water should be monitored closely in the galleries. The pattern of joint displacement is cyclic and deformation at any location is not alarming (Fig. 2.12). It is found that the temperature is not much affected due to rise in reservoir water level. The heat flow from middle portion of the dam body towards upstream and downstream faces indicates the normal behavior (Fig. 2.13). It is also found that during winter, mass concrete cools in middle part of dam body while

during summer the temperature of very concrete rises in middle part. The pattern of strain is cyclic in nature and do not exceed the strain capacity of concrete (Fig. 2.14).

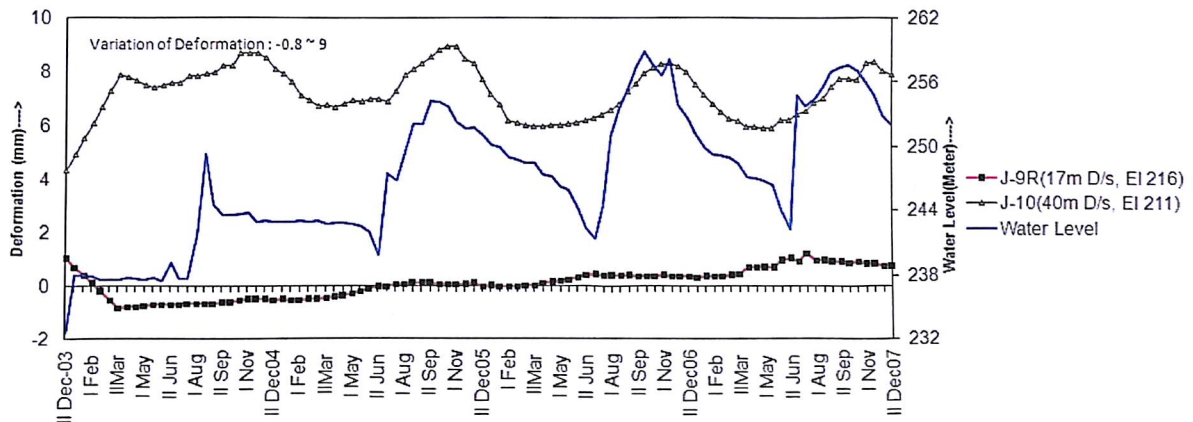


Fig . 2.12 Typical Variation of block movements

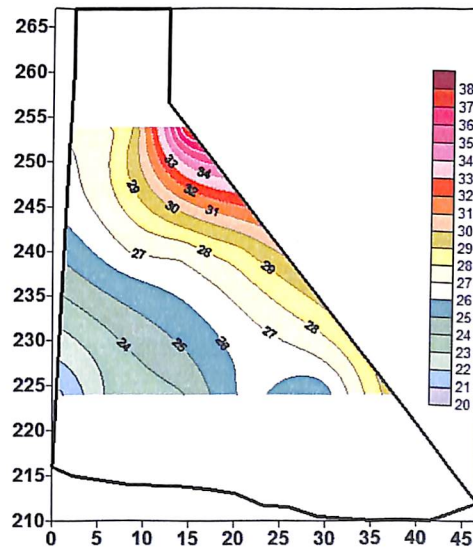


Fig. 2.13 Temperature

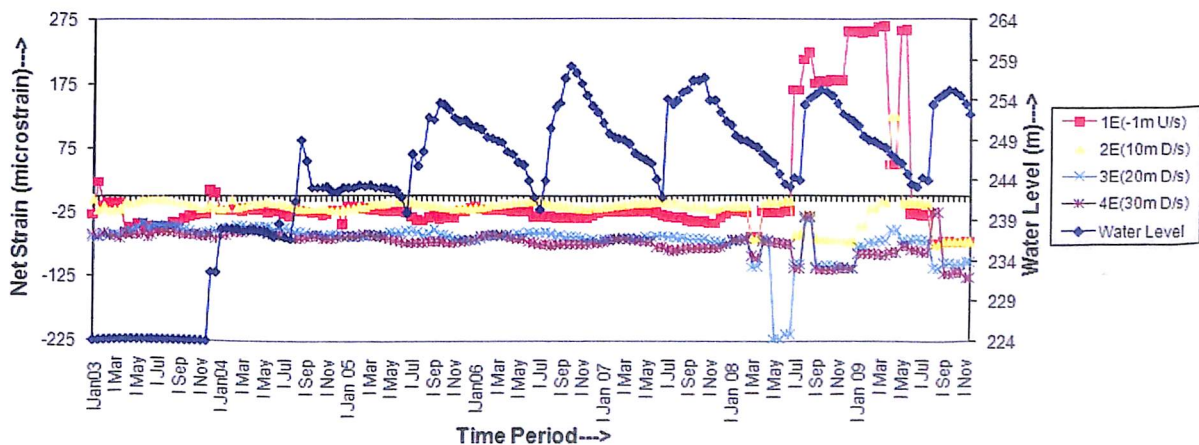


Fig. 2.14 Typical Strain Meter Plot

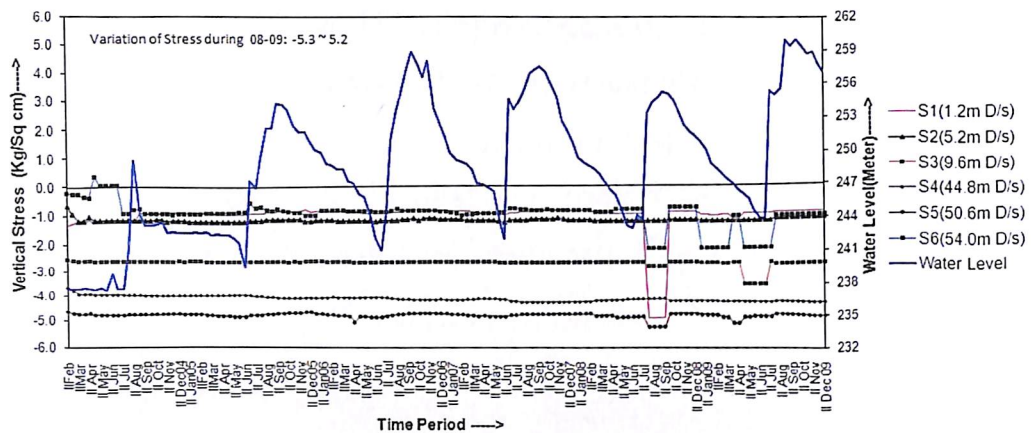


Fig. 2.15 Typical Stress Meter Plot

The variation of vertical stress with time- period (Fig. 2.15) is observed that at downstream side, stress meter shows almost zero compression, which indicates the possibility of initial settlement of foundation and reservoir base due to reservoir filling and stresses in other locations are well within limits.

The measured downward vertical settlement of foundation (Fig. 2.16) is not alarming and may be considered within allowable limits.

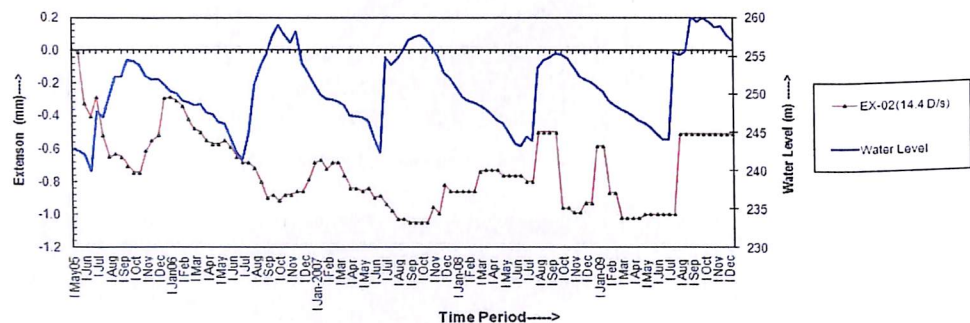


Fig. 2.16 Typical Extensometer

From the analysis of data obtained from the instruments installed in spillway dam block No. 13 and 25 overall behavior of the dam is studied. Studies shown that the effect of relief holes in reducing the uplift pressure in block no 13 and early detection of higher uplift pressure at d/s in block no 25.

2.3 SEISMIC TOMOGRAPHY

The current status of the structure, remaining service life and schedule for rehabilitation is very important in dam safety programme. The information required for the

evaluation of status of the structure is related to physical, chemical or mechanical properties. These techniques include visual inspection, instrumentation and non-destructive testing techniques. To confirm the visual observations, the engineers take some concrete samples by coring. All geotechnical tests provide information from point to point and the values are interpolated for in between places. These tests grossly under sample the subsurface and are frequently inadequate (Sarman & Palmer, 1990). In geophysical techniques like cross-hole and seismic tomography, a disturbance (mechanical, electrical or electromagnetic) is induced in the material to be investigated, and its response is registered to the given excitation. The response is then used to estimate the properties of the material (e.g. compressive strength, water content and porosity) and to detect and characterize internal defects. Geophysical methods are useful as non-destructive tools that can provide information over large volumes as compared to point measurements. The purpose of the survey is to provide basic data through imaging the internal structure for a precise safety and stability examination of the dam.

Cross-hole seismic studies give average velocities of compressional and shear wave velocities but it does not give information on distribution of velocities along the line. Cross-hole tomography can overcome this limitation. Seismic tomography survey gives two-dimensional velocity distribution i.e. a tomogram. The output of tomography method is an image of the seismic-wave velocity structure between the boreholes obtained by measuring the seismic-wave travel times between a large number of source and receiver locations at various depths within the boreholes.

Geophysical Tomography is a means of making a picture of a slice of the earth using geophysical data. It involves reconstructing a cross section or to find the interior distributions of values inside an object from the projections (sum of some interior value) measured outside the same object. The mathematical basis for tomography was established in 1917 by Radon (Radon, 1917) who showed that an object could be exactly reconstructed from a complete set of its projections. Seismic tomographic surveys use travel time data (Wadhwa, 2005, Kevin, 1988) because of ease and convenience.

Tomography is a type of inverse problem where measurements are first made of some energy which has propagated through a medium. The received character of this energy (e.g. amplitude, travel time or potential difference) is then used to infer the values of medium through which it has propagated. The two-dimensional velocity distribution plot i.e. a tomogram, shows the internal structure and properties of a medium located between a set of

measuring points. Thus, this technique makes the material “transparent” and allows detection of weak zones that require further analysis and/or rehabilitation. The benefit of this technique is that it gives an indication of the properties of one part relative to another one in a given structure. Laboratory tests on cored samples taken from the structure to be investigated can help in interpretation of the tomographic image in terms of the mechanical properties.

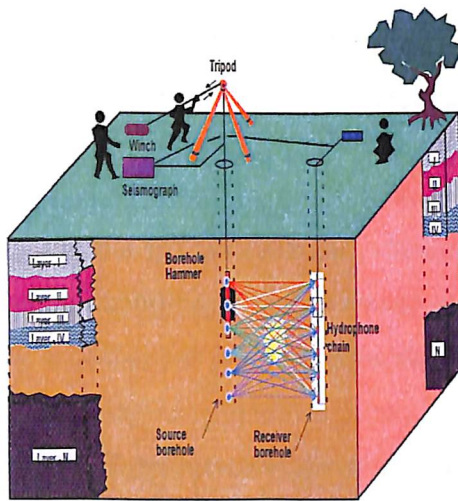


Fig. 2.17 Seismic Cross-hole Tomography - Field setup

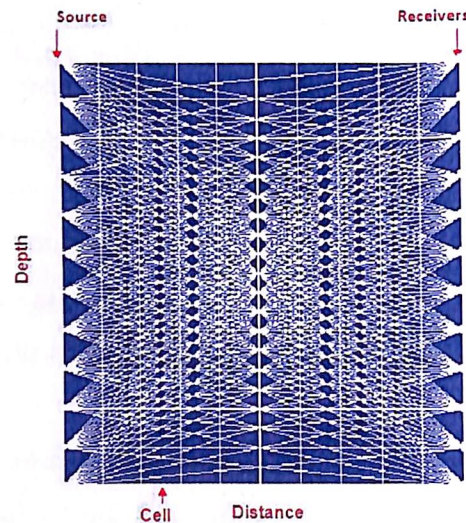


Fig. 2.18 Ray diagram for various positions of sources and receivers

In a typical field data collection 12 element hydrophone string is lowered in the receiver borehole in one depth location and in the source borehole hammer is lowered systematically from top to bottom and all the possible travel time are recorded (Fig. 2.17). First, seismic waves generated by the borehole hammer clamped at the elevation of first hydrophone of hydrophone string are picked up by hydrophone chain consisting of 12 hydrophone elements spaced 1 m apart. The hammer is then lowered further by 1 m and seismic waves produced at this depth were also recorded by keeping the hydrophone string at the same depth elevation. The process of collecting travel times data is repeated up to elevation of last hydrophone. Fig. 2.18 shows typical ray diagram for 12 source locations and 12 receiver locations.

The measured travel time in sonic measurements from shot to each receiver location reflects the average velocity along the path joining source and the receiver. Stated mathematically, each travel time 't' represents a line integral of wave slowness

$$t = \int_S^R \frac{1}{v} dl = \int_S^R p \cdot dl \dots\dots\dots(4)$$

'p' (inverse of velocity v) along the corresponding ray path from source to receiver where 'dl' is the path length increment.

By approximating continuous function $p(x, z)$ as a set of discrete elements or pixels, each with a uniform slowness p_j ($j=1,2,\dots,M$ where 'M' is the number of pixels) the above integral becomes

$$t_i = \sum_{j=1}^M p_j d_{ij} \quad (i = 1, 2, \dots, N) \quad \dots\dots\dots (5)$$

where 'd_{ij}' is the distance travelled by ray 'i' in pixel 'j'. For the entire set of rays, the above travel time equation can be expressed in matrix form

$$T = DP \quad \dots\dots\dots (6)$$

where 'T' the arrival time and 'P' the slowness are column vectors of order 'N' and 'M' and 'D' is an 'N' by 'M' rectangular matrix.

Using these average and constant velocities for all the pixels, the set of model arrival times for the various rays are calculated as follows:

$$T' = DP' \quad \dots\dots\dots (7)$$

Here, prime notation refers to the initial model. The model travel times so evaluated are subtracted from the field measured times to obtain the residuals dT

$$dT = T - T' \quad \dots\dots\dots (8)$$

These residuals are then back projected to obtain the correction factors 'dP' for slowness. The velocity model is then updated with the correction factor to change the slowness as under

$$P'' = P' + dP \quad \dots\dots\dots (9)$$

The procedure is repeated with the new slowness values and each pixel slowness is modified until convergence or set criterion limit (percentage RMS error) is reached.

To summarize the process of tomographic inversion, the distance between two boreholes is divided into small rectangular grids (pixels). The total P-wave arrival times recorded in the field decide the number of pixels. The only condition in deciding the number of pixels is that their number should not be greater than the number of recorded arrival times. After fixing the pixel size, it is assumed that the P-wave velocity in each pixel is constant. With these initial average velocities, first arrival times of the rays for all possible positions of sources and receivers are calculated using straight ray tracings. These synthetic travel times are compared with the field measured timings and the differences or residuals are inverted to obtain perturbations to the velocity model using the algorithm (Jackson, 1992). The procedure is repeated and the velocity model is perturbed until, either there are no differences between the model travel times and the measured arrival times or RMS error is within the set limit (Singh and Singh, 1991). It is difficult to quantify the resolution of the final velocity model. The resolution is determined primarily by the angular coverage of the data (Nolet, 1985; Bregman et al., 1989). Features situated in the central part of the plane are intrinsically better resolved than those at the top or the bottom where there is a small ray aperture. In the central part of the tomogram the resolution is consistent in the horizontal and vertical direction. Unlike CAT scanning in geophysical tomography source and receiver can rarely be placed on all sides of the imaging region resulting in some sub regions having very poor angular ray coverage.

2.3.1 CW&PRS Experiences

2.3.1.1 Seismic Tomography Studies for VPHEP, Uttarakhand

Vishnugad Pipalkoti Hydroelectric Project (VPHEP) a run-of-the-river scheme on river Alaknanda consists of construction of concrete gravity dam of 65 m height, underground tunnel, desilting chambers, head race tunnel and underground powerhouse (CWPRS Technical Report No. 4841). A 650 m long adit has been made in the hillock at Hut village to assess the rock quality as also to decide the exact location of the powerhouse cavity. To assess the quality of rock with depth as also to delineate the weak zones, if any tomographic studies in ten NX size holes drilled 10 m apart in the adit of powerhouse up to 47 m depth were carried out. The proposed underground powerhouse is situated in Himalayan region. The rocks in general, are

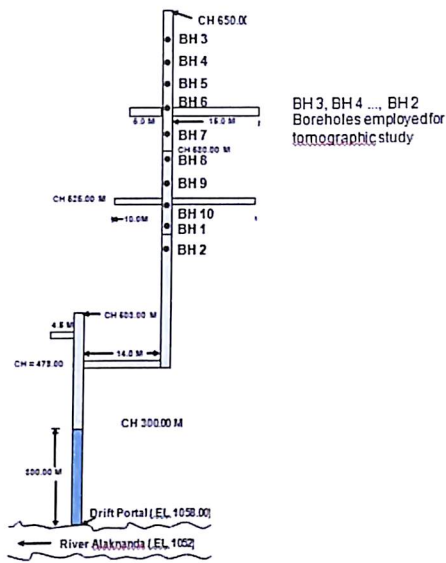


Fig. 2.19 Location of boreholes for Tomography studies

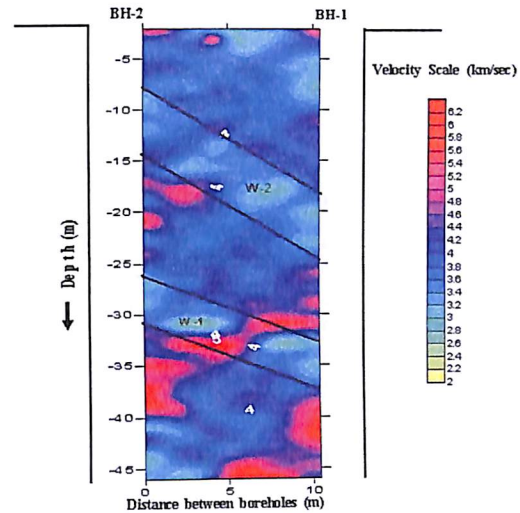


Fig. 2.20 Contoured P-wave velocities obtained between pair of boreholes BH-2 and BH-1

tightly folded and are in a highly deformed and crumbled state. A hydraulically clamped borehole hammer was used as a source for generation of compressional (P) waves. The hammer was clamped at a particular level in the NX (inner dia. 76 mm) sized hole and striking it up or down produces P-waves. The borehole receivers consisted of a 12-channel hydrophone string with hydrophones spaced 1 m apart. A 24-channel signal enhancement seismograph, 'Terraloc model MK6' was used for data acquisition and recording.

Fig. 2.19 shows the site plan of the ten NX sized rotary drilled boreholes at intervals of 10 m drilled up to depths of 48 m each in the adit of the power house. The P- wave data were acquired by moving the hydrophone string in the receiver borehole and borehole hammer in the source borehole. Travel time data was collected between all the boreholes by collecting travel time data between two adjacent boreholes at a time. Nine tomograms depicting lateral and vertical variations of P-wave velocities for the planes between two successive boreholes were obtained by inverting the arrival times using 'GEOTOM' software package.

Fig. 2.20 shows the contoured image of P-wave velocities for the plane between boreholes BH-2 and BH-1. The tomographic reconstructions were constrained by limiting the upper P- wave velocity to 6000 m/sec and a lower bound velocity of 1800 m/sec.

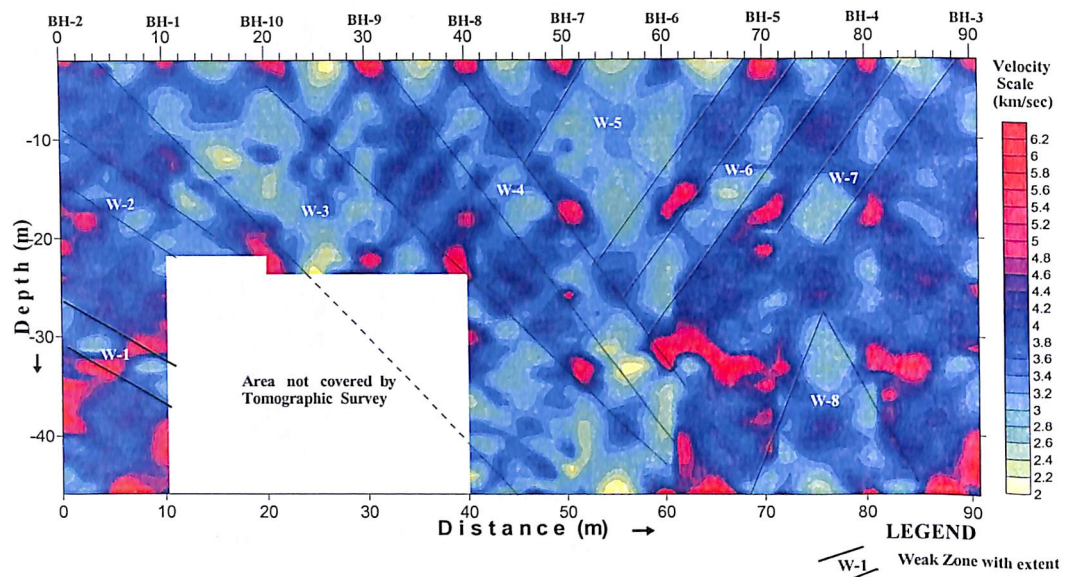


Fig. 2.21 Contoured P-wave velocities between pair of boreholes 2-1, 1-10, 10-9, 9-8, 7-6, 6-5, 5-4 & 4-3

A combined tomogram was obtained by taking velocities of individual pixel from the nine individual tomograms and is shown in Fig. 2.21. In the combined tomogram eight isolated weak zones of varying dimensions and orientations were inferred. The P- wave velocity of good quality dolomite rock varied between 3500 and 6000 m/sec while the velocity less than 3500 m/sec was interpreted as weak zone in dolomite. In the combined tomogram two sets of weak zones are observed. One set of weak zones i.e. weak zones W-1, W-2, W-3 and W-4 dip from top to bottom and from left to right. The second set of weak zones i.e. weak zones marked as W-5, W-6 and W-7 dip from top to bottom and from right to left. The weak zone W-8 is triangular in shape. This zone exists between boreholes BH-5 and BH-3 and extends from 30 m depth to 46 m depth.

2.3.1.2 Seismic Tomography Studies for Dimbhe Dam, Maharashtra

Dimbhe dam, constructed on River 'Ghod' in Ambegaon Taluka, Pune, is a masonry dam with a height of 72.1 m and length of 852.0 m. The dam was constructed in the year 2001, with a total storage capacity of 13.5 TMC. Subsequently after the impoundment of the dam, heavy leakages were observed through the masonry in both the galleries as well as in the downstream face of the dam. To assess weak zones, prone to seepage, seismic tomography was carried out in the masonry portion of the dam. The weak zones, if present,

will be represented by low velocity values and hence can be delineated (CWPRS Technical Report No. 4992).

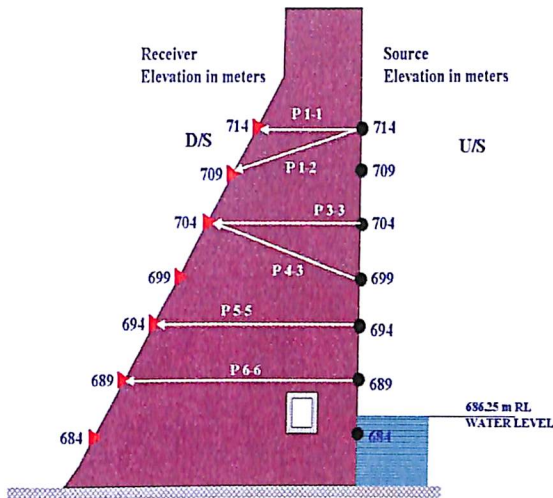


Fig. 2.22 Planes covered by Seismic Tomography Survey

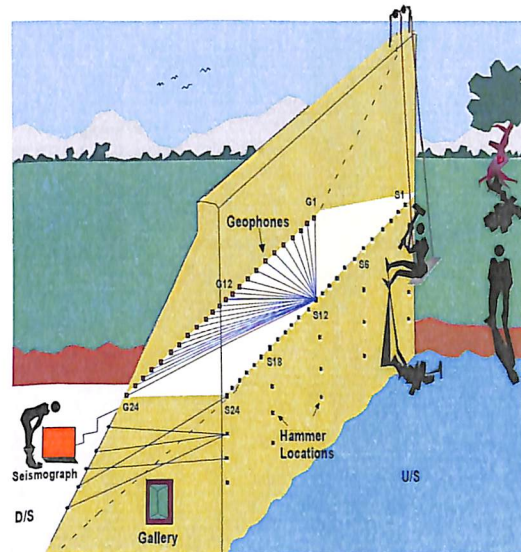


Fig. 2.23 Tomography Survey setup for a plane

The instrument used for data acquisition and recording was a 24-channel signal enhancement seismograph 'McSeis-SX' manufactured by M/s. OYO Corporation, Japan. Elastic waves were produced by hitting with a 10 Kg sledge hammer at preset locations on the upstream face of the dam and the seismic wave arrivals are picked up by different geophones having 10 Hz natural frequency placed on the downstream face of the dam at known locations. The survey was conducted along six planes from ch. 630 m to ch. 676 m at six elevations separated by 5.0 m (Fig. 2.22). Tomographic set up for collection of travel time data between different source and receiver locations for a plane is shown in Fig. 2.23.

For collecting travel time data 24 geophones spaced 2 m apart were placed on the downstream face of the dam and 47 hammer points spaced 1.0 m apart were used for generating acoustic waves on the upstream face of the dam yielding 1128 ray paths. The velocity field between the planes was discretized on a 230 square grid points. This means 230 unknowns against 1128 equations. Therefore, for this type of set-up, where the number of equations is much larger than the number of unknowns, a near unique inversion is possible.

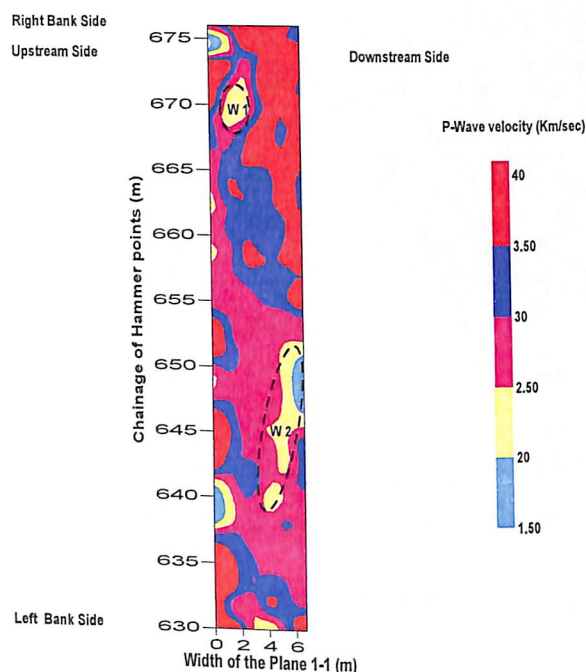


Fig. 2.24 P-wave velocity distribution along Plane 1-1 (714-714)

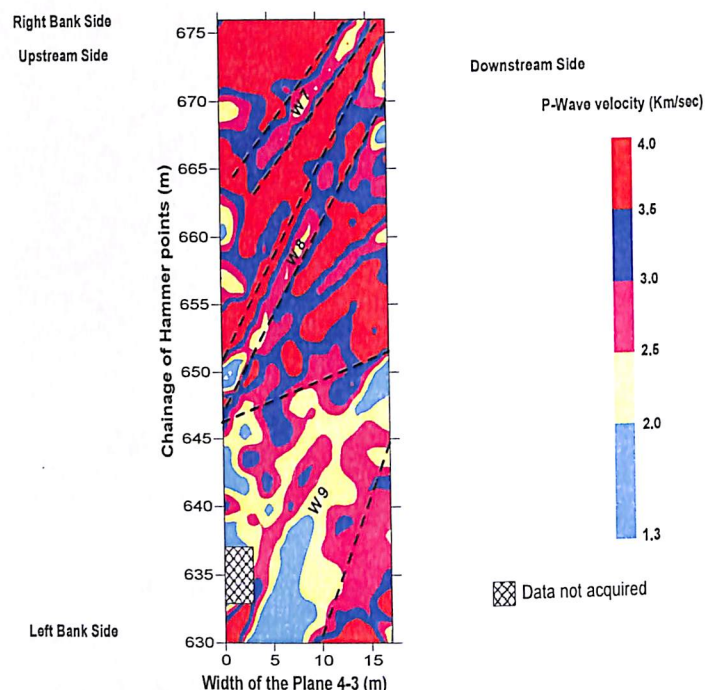


Fig. 2.25 P-wave velocity distribution along Plane 4-3 (699-704)

Travel time against 1128 equations. Therefore, for this type of set-up, where the number of equations is much larger than the number of unknowns, a near unique inversion is possible. Travel time data for tomographic analysis was collected one plane at a time. The six tomograms depicting P- wave velocity distribution were obtained by inverting the arrival times using 'MIGRATOM' software package. The reliability and uniqueness of the velocity tomograms were improved by putting the lower (1000 m/sec) and upper (4000 m/sec) limits on the velocities.

Out of the six planes covered, two isolated weak zones each are identified in planes 1-1, 1-2 and 3-3 and the lateral and vertical extent of plane 1-1 is shown in Fig. 2.24. In plane 4-3 three weak zones are identified in the form of strips. These zones extend from upstream side to downstream side of the dam (Fig. 2.25). In the remaining two planes, 5-5 and 6-6, the quality of the masonry inferred was good. Tomogram obtained for the plane 5-5 is shown in Fig. 2.26. In all 9 weak zones are delineated.

Results of the seismic studies revealed that the velocity of the masonry generally, varies from 2500 m/sec to 4000 m/sec. From these values of wave velocities it was inferred

that the masonry was of good quality. However, in some planes velocities less than 2500 m/sec are observed indicating that the masonry is of inferior quality. It is recommended that these zones may be grouted with suitable material to strengthen masonry of the dam so that seepage can be controlled.

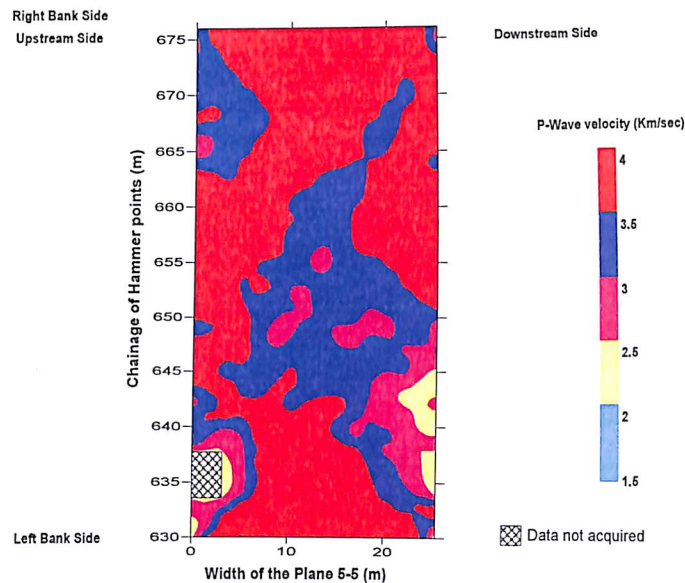


Fig. 2.26 P- wave velocity distribution along plane 5-5 (694-694)

2.4 NUCLEAR AND SONIC LOGGING TECHNIQUES

“Dam failures have not only occurred in dams built without application of engineering principles; but also in dams built to accepted state of art of dam engineering”(CWC Report). Dam failures usually result from poor design, improper construction, ageing, inadequate maintenance, or a combination of the above. Uncontrolled seepage is one of the major causes of dam failures. Excessive and uncontrolled seepage in dams generally occurs through the body of the dam; the geological formations in the vicinity of the reservoir; or through the structure-foundation interface. Among the different techniques, borehole logging techniques are being increasingly used in seepage investigations of dams for identifying cracks / voids and permeable zones. The techniques are also applied for detecting engineering parameters of dam like bulk density, sonic velocities and mechanical properties of the rock and body of the dam.

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. It includes techniques of lowering sensing devices in a borehole and recording some physical

parameters that may be interpreted in terms of the physical and chemical characteristics of the rocks and the fluids contained in them. The subsurface geologic conditions and engineering characteristics can be determined directly or indirectly from the properties measured by these techniques.

Borehole logs can be interpreted to determine the lithology, geometry of the formation, resistivity, bulk density, porosity, compressional and shear wave velocities (V_p & V_s), moisture content, water bearing strata and movement of water (Scott Key, 1971).

2.4.1 Borehole logging

Based on the parameter to be measured, borehole logging is classified into various types. Table- 2.3 below shows the various logging methods and the parameters measured to characterize the subsurface. Among them, nuclear logging and acoustic logging techniques are important in evaluation of parameters related to dam safety and rehabilitation. These techniques are discussed below.

Table – 2.3
Types of Logs

Logging Method	Parameter Measured
• Resistivity • Spontaneous potential (SP) • Natural Gamma-ray • Gamma-Gamma • Neutron • Sonic • Temperature • Caliper	• Electrical resistivity • Electrical potential • Natural gamma radiation • Bulk density • Porosity • V_p & V_s, Mechanical properties of rocks • Fluid Temperature • Borehole diameter

2.4.1.1 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. Both gamma radiation and neutrons possess appreciable penetrating power and are measured in nuclear/radioactive logging (Mayers, 1992). 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.

Well logging instruments which measure the radioactivity of nearby formation may be considered under three headings (i) those which detect Gamma- radiation resulting from

the natural radioactivity of the U, Th, and K in the rocks (Natural gamma) (ii) those which employ artificial gamma rays (gamma-gamma/density) (iii) those which use neutron sources to induce nuclear processes (Neutron-neutron).

Natural Gamma Log

Natural gamma logs are records of 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 probe used for this logging consists of a detector and amplifier; the detector mostly is a scintillation counter, which employs thallium activated sodium iodide crystals to detect gamma radiation. The gamma emitting radioisotopes normally found in rocks are potassium-40 and daughter products of the uranium and thorium decay series. The gamma radiation from either the uranium and thorium series is much larger than that of potassium-40.

Gamma-gamma Log

Gamma-gamma logs are records of the intensity of gamma radiation from a gamma source in the probe after it is back scattered and attenuated within the borehole and surrounding rocks. The main use of gamma logs is for the measurement of bulk density and porosity of rocks.

A radioactive source contained in this logging probe emits medium energy gamma rays into the formations. These gamma rays collide with the electrons in the formation. At each collision, a gamma ray loses 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).

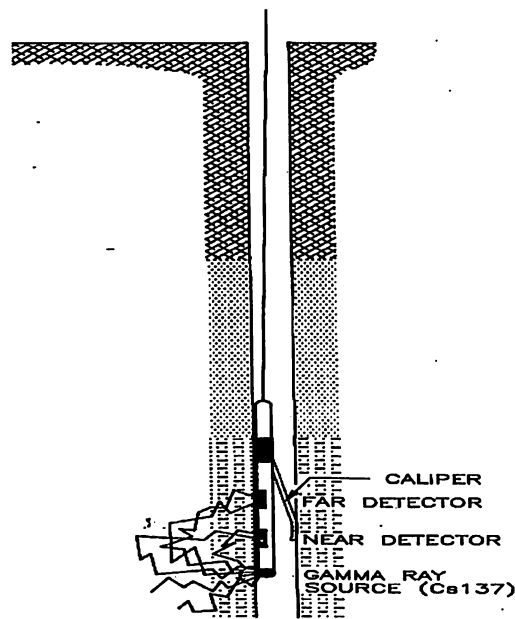


Fig. 2.27 Set up of Density Probe

Sonde used for density logging contains a concentrated source of mono-energetic gamma rays, a Caesium-137 (^{137}Cs) or Cobalt-60 (^{60}Co), the detector usually a scintillation counter. Since the tool is short, adjacent beds or bed thickness have little distorting effect. The instrument must be calibrated for different formation densities and hole diameter. The errors in bulk density obtained by this method are of the order of $\pm 5\%$.

The probe contains a Gamma source (370 MBq ^{137}Cs) and two high-sensitivity scintillation gamma detectors. Gamma radiation from the source is backscattered (Compton Scattering) and reaches the detectors where the relative count rates provide a measure of the formation bulk density (Fig. 2.27).

The probe provides a continuous measurement of density using two source-to-detector spacing. The near detector (High Resolution Detector) provides good resolution and bed thickness and boundary while the far detector (Long Spacing Detector) offers deeper formation penetration and greater immunity from borehole influences. A natural gamma detector is also provided and situated near the top of the probe away from the radioactive source.

Neutron - Neutron 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 are continuously emitted from a radioactive source such as Americium-241-Beryllium, which is mounted in the probe used for neutron logging. These neutrons collide with nuclei of the formation material; with each collision the neutron loses some of its energy. The rate at which a neutron loses energy in elastic collisions varies inversely with mass of the target nucleus. Thus the slowing down of neutrons depends largely on the amount of hydrogen in the formation. Within a few microseconds the neutrons are slowed down by successive collisions to thermal velocities, corresponding to energies of around 0.025 eV. They then diffuse randomly without losing any more energy until they are captured by the nuclei of atoms such as chlorine, hydrogen etc. which then emit characteristic gamma rays.

Depending on the type of neutron logging tool, either these capture gamma rays (neutron-gamma) or neutrons (neutron-neutron log) themselves are counted by a detector in the probe. Modern neutron tools most commonly count thermal neutrons with a He-3 detector. 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.

The probe contains a Helium proportional detector and a detachable sealed neutron source (37GBq $^{241}\text{Am-Be}$) (Fig. 3.28A & 3.28B). Fast neutrons from the source are scattered and slowed, principally by the hydrogen in the formation, until they reach thermal energy levels and are absorbed. The flux of the thermal neutrons reaching the detector is related to the hydrogen content of the formation. The probes are pre-calibrated by the manufacturers and the calibration file is provided for converting the log response to the respective bulk density and porosity. It is also possible to calibrate the probes in the laboratory using calibration pits filled with different materials of known density and porosity.

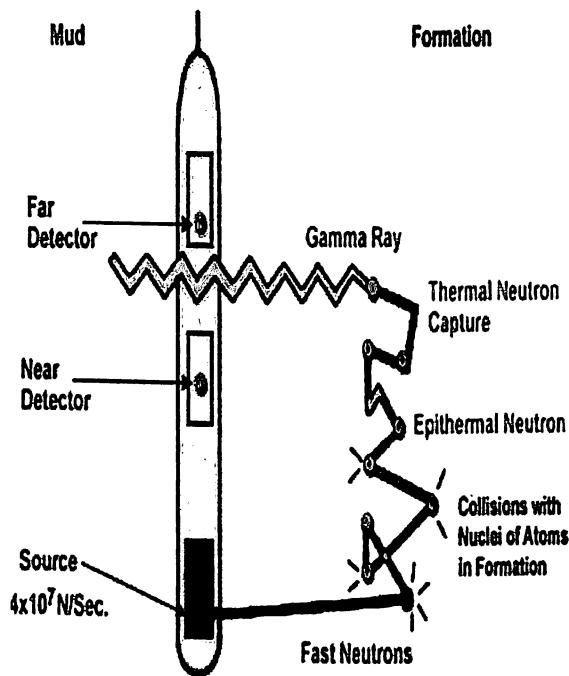


Fig. 2.28A Principle of Neutron Log

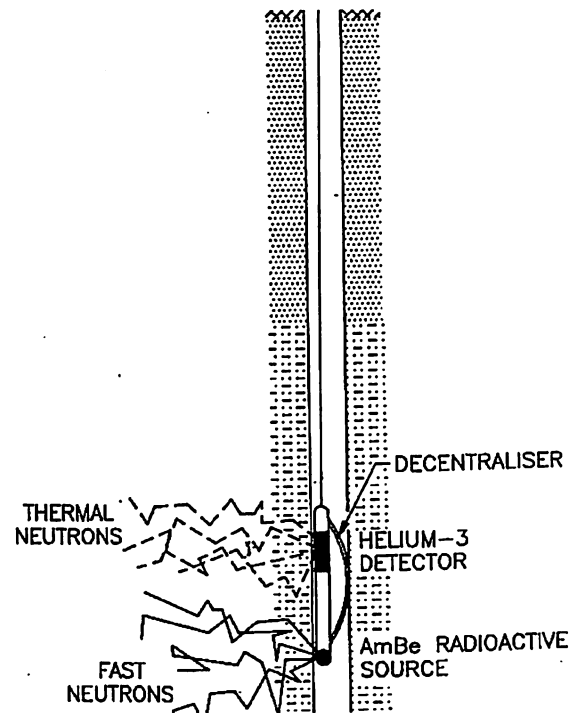


Fig. 2.28B Set up for Neutron-Probe

Calibration of Nuclear Logs

The response obtained in gamma-gamma log is in terms of count rate (counts /sec) a unit of radioactive disintegration and it is required to be converted into known parameters like density, by calibrating the probe in the laboratory. Calibration of nuclear logs can be carried out either in standard pits constructed with materials of known density values or with laboratory calibration of the cores samples. In calibration pits the response of density probe for different known density values is obtained by filling the pits with soil, murrum, sand etc. in various proportions by simulating field conditions. The calibration curve is then obtained by plotting density against counts /sec and applying best-fit equations. This calibration curve is used while evaluating density from gamma-gamma logs. In general, a low value of count rate corresponds to a high value of density and vice-versa.

2.4.1.2 Acoustic logs

Acoustic logging systems use medium to high-frequency acoustic (sonic) energy emitted from a sonde to obtain seismic velocities of the geologic material in the borehole. The frequency spectrum of the wave depends on the source frequency spectrum and is usually in the 5 to 30KHz range (Crain, 2004). An acoustic log is thus a record of transit time

of an acoustic pulse between a transmitter and receiver in a probe. Sonic logging is generally used for porosity measurement and identification of fractures.

Basically, all acoustic logging devices contain one or two transmitters that convert electrical energy to acoustic energy, which is transmitted through the environment as an acoustic wave. The receivers then convert the acoustic energy back into electrical energy for transmission up the cable. (Fig, 2.29). The propagation velocities of the seismic waves can be calculated by travel times and distance traveled from transmitter to receiver. The devices must be operated in a fluid-filled borehole. The types of acoustic waves received after transmissions through the rock are classified as Compressional or P waves, Shear or S waves, and surface waves. Compressional waves have the highest velocity and arrive first, followed by the shear waves and the surface waves. Devices with two receivers cancel the borehole fluid travel times so that only the refracted wave paths through the borehole wall are measured.

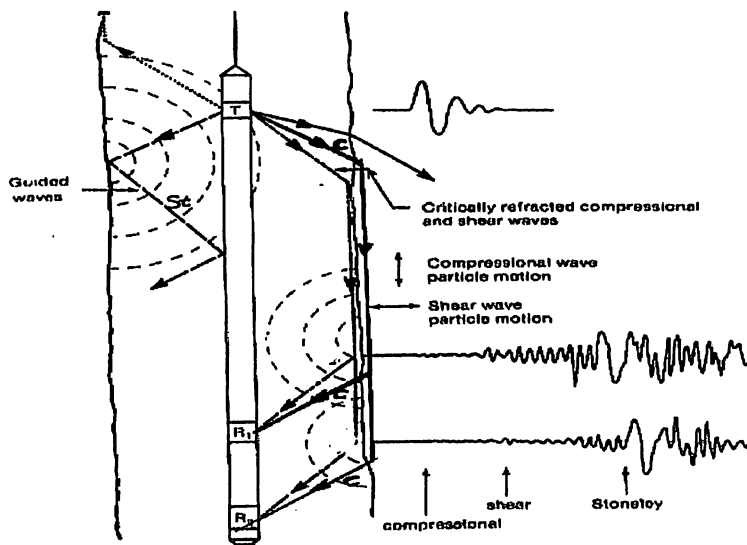


Fig. 2.29 Set up of Acoustic Log

The acoustic pulses generated at the transmitter are in the lower ultrasonic range around 23 kilohertz (kHz). The radius of investigation of the sonic tool varies with frequency and wavelength of the elastic wave as well as with the sonic velocity. It is reported to be 3 times the wavelength. (Pirson, 1963).

A sonic log is conventionally recorded in terms of "Transit Time"- the time in microseconds taken for the compressional wave to travel through 1 foot of the formation and is reciprocal of velocity.

Determination of dynamic properties of the body of the dam

The dynamic properties of the material surrounding the borehole is obtained from nuclear density and sonic logs. The P-and S-wave velocities obtained can be used directly to calculate the Poisson's ratio, Young's modulus, and shear modulus. The, Poisson's ratio is calculated using the equation

$$\sigma = \frac{\frac{1}{2} V_p^2 - V_s^2}{V_p^2 - V_s^2} \quad (1)$$

where V_p and V_s are the compressional and shear wave velocities, respectively. The effective porosity of the strata can also be evaluated. The young's modulus is represented by the relation

$$E_d = \rho V_p^2 (1 + \sigma)(1 - 2\sigma) / (1 - \sigma) \quad (2)$$

The shear modulus is defined in terms of density and S-wave velocity, as given in the following equation:

$$G = \rho V_s^2 \quad (3)$$

2.4.1.3 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 provide the physical size of a drill hole and should be run in all boreholes in which other logging is anticipated. Caliper logs provide indirect information on subsurface lithology and rock quality. Borehole diameter varies with the hardness, fracture frequency, and cementation of the various materials penetrated. Borehole caliper surveys can be used to accurately identify washouts or swelling or to help determine the accurate location of fractures or solution openings, particularly in borings with core loss.

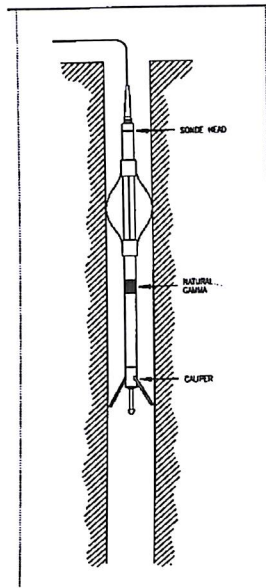


Fig. 2.30 Set up of Caliper

Caliper logs can also identify porous zones in a bore by locating the intervals in which excessive mud filter cake has built up on the walls of the borehole. One of the major uses of borehole caliper logs is to correct for borehole diameter effects. Caliper logs also can be used to place water well screens, position packers for pressure testing in foundation investigations for dams or other large engineering structures, and help estimate grout volumes in solution or washout zones.

Multiple-arm calipers convert the position of feelers or bow springs to electrical signals in the probe (Fig. 2.30). The electrical signals are transmitted to the surface through an armored cable. Some caliper systems average the movement of all the arms and record only the change in average diameter with depth, and others provide the movements of the individual arms as well as an average diameter. The shape or geometry of the borehole cross section can be determined with the individual caliper arm readings.

3.4.1 Instrumentation

The well logging unit consists of three parts: i) the down hole probe or sonde, ii) cable and winch, and iii) surface system for signal processing and recording. The output, electronic signal of the sondes 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.

Central Water & Power Research Station is equipped with portable well logging unit manufactured by M/s Robertson Geologging Ltd., U.K. The Robertson Geologging (RG) equipment 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 VIZ. Nuclear (consisting of Natural Gamma, Gamma-Gamma Density, Neutron), Caliper, Sonic. The general set up of RG well logging unit is shown in Fig. 2.31.



Fig. 2.31 Schematic Setup of Borehole Logging

2.4.3. CWPRS Experiences

Borehole logging has been efficiently used for solving problems related to dam safety and rehabilitation of structures at CWPRS. A few typical case studies are described below.

2.4.3.1 Determination of Dynamic Properties at Kolkewadi Dam, Maharashtra

Kolkewadi dam is a part of Koyna Hydroelectric Project Stage-III. It is constructed in the year 1975, across Boladwadi stream situated in Kolkewadi village of Ratnagiri District, Maharashtra. The total length of the dam is 497 meters and maximum height above the foundation is 66.30 meters. The dam is constructed in concrete and UCR masonry with different proportion of cement mortar of UCR masonry with 1:3, 1:4 and 1:5 and cement mortar is 10%, 5% and 85% respectively. As the Kolkewadi dam was in the seismic epicenter zone, it was required to undertake its strengthening. Dynamic analysis of hydraulic structures is carried out for its strengthening. For identifying the zones of stress concentration, the in-

situ properties of masonry / concrete were evaluated by conducting nuclear and sonic logging of boreholes drilled in the body of dam. The dynamic properties determined were insitu bulk density, shear and compressional wave velocities, Poisson's ratio, Shear and Young's modulus of masonry in different proportion of UCR. (CWPRS Technical Report No. 4483)

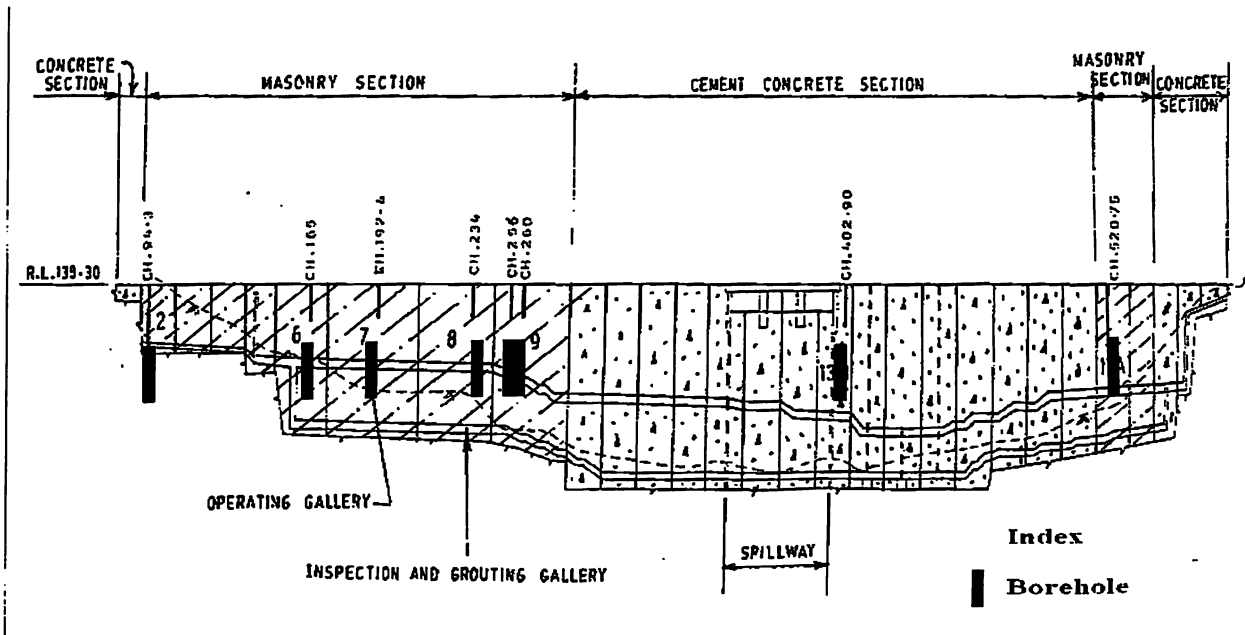


Fig. 2.32A Index Map of Kolkewadi Project showing Borehole

Nuclear density and sonic logging were employed in eight boreholes of diameter 76 mm, 150 mm and 250 mm drilled in the body of dam covering different portions of UCR/cement ratio viz. 1:3, 1:4 and 1:5 and also in the concrete portion of the dam. The depths of boreholes were varying from 6 m to 18 m. Fig. 2.32A shows the index map of Kolkewadi dam along with the location of boreholes.

The bulk density of the masonry was determined from gamma-gamma logging using calibration curves and applying the necessary borehole diameter corrections, which were obtained by caliper logging. The compressional and shear wave velocity in the borehole were obtained from the Sonic logging. The Poisson's ratio, Young's modulus and shear modulus values obtained were evaluated with determined values of densities, compressional and shear velocities in the boreholes obtained and were used for evaluating the dynamic properties of

the masonry. Fig. 2.32 B shows the typical plot of dynamic properties of the masonry obtained from nuclear and sonic logs.

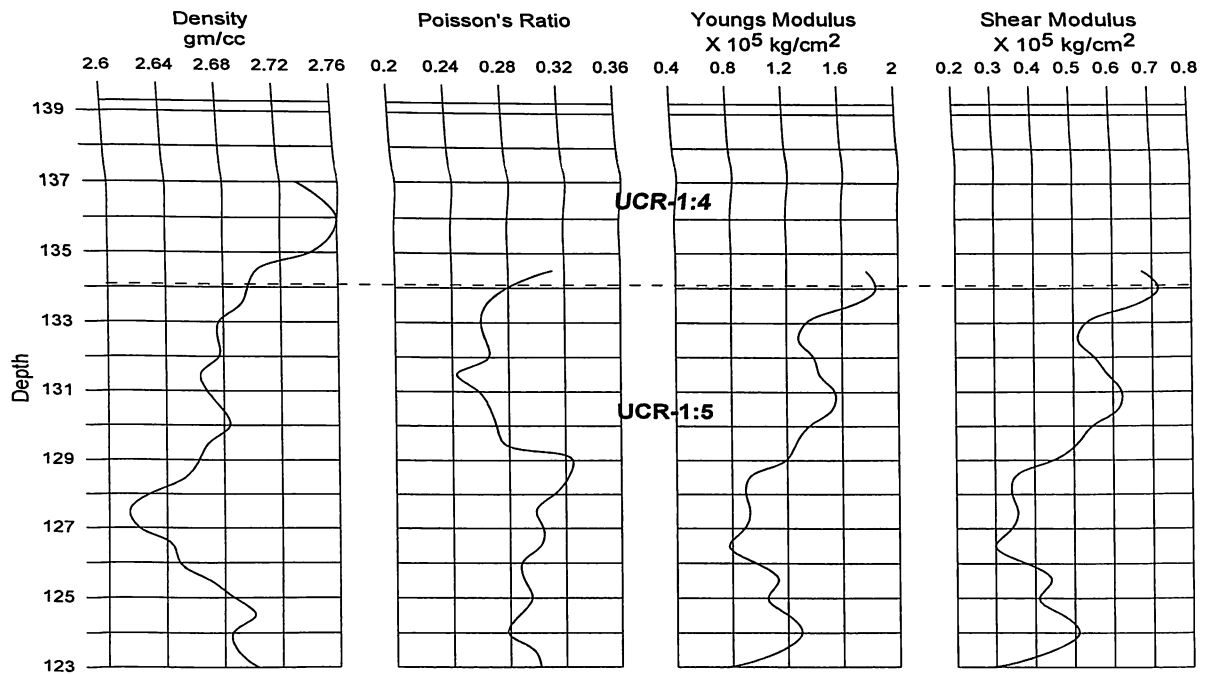


Fig. 2.32B Typical plot of dynamic properties of the masonry

2.4.3.2 Determination of Insitu Bulk Density of Barvi Masonry Dam:

Barvi Dam, located in Ulhasnagar, Maharashtra on Barvi river is 38.92 m high and 750 m long dam. In the left flank portion of the main dam, 1551 lts/min of seepage was observed from the body drainage holes (porous pipes). Grouting was carried out between Ch. 91.5 to 51.5 m, Ch. 170 m to Ch. 210 m, and Ch. 269.5 m to Ch. 281.5 m to reduce the leakage and strengthen the dam. After grouting, the seepage was reduced to 328.25 lts/min. Therefore to evaluate the efficacy of grouting and guniting, nuclear logging studies were conducted to determine the insitu bulk density of the masonry. The bulk density of the masonry was determined from gamma-gamma logging using calibration curves. It was found that the bulk density of the masonry varied between 2.3 gm/cm³ to 2.7 gm/cm³. It was also observed that the post grout density was more than the pre-grout density which indicated improvement in grouting. In the left flank portion, a weak zone was identified between RL 58 and RL 48 along with presence of cracks.(CWPRS Technical Report No. 2687)

It was recommended to conduct grouting under controlled pressure in the left flank portion in the zones where the post grouting density was around 2.4 gm/cm³, particularly at RL 57.61 m at the kink of the downstream face. It was recommended that the actual measured value of the density should be used for stability analysis.

2.5 TRACER TECHNIQUES

The hydraulic structures viz. dam, reservoir, canal etc. are although designed not to seep or leak, still minimum seepage or leakage occurs through these hydraulic structures. If the seepage exceeds, it may cause softening or weakening of rock or soil mass and leading of excessive hydrostatic uplift pressure. So, seepage through such structures is a potential threat to structure, public welfare and wastage of water (Nilsson, et. al, 2004).

Tracers have become a primary tool for process investigation, qualitative and quantitative system analysis and integrated resource management in the hydrology. 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. The multi parameter detecting technology which synthesizes the advantage of the nature tracer, manpower tracer and isotope has gained great effect on detecting the leakage passage of dam (Lin. T, et. al, 2008), canal or any hydrological system. In most cases, the tracer is used to track the movement of water (Flury. M, et. al, 2003), analysis of flow pathways, velocities and travel times, hydrodynamic dispersion, recharge, and discharge etc (Moser. H, 1995). This means that the nature and magnitude of seepage flux is inferred or calculated from the measurements of other parameters such as hydraulic head, hydraulic conductivity, temperature or isotopes (Ancid, 2000). The need for tracers is mainly a consequence of the large variability with time and space of relevant parameters of water systems. This variability frequently makes it difficult to obtain reliable values of the parameters which are responsible for the behaviour of water in the system under investigation, especially when precise information is absolutely necessary. In these cases, the use of tracers may provide the only available solution (Plata-Bedmar. A, 1988). The change of permeability of the dam body media is interpreted as a result of interaction with seeping water after reservoir impoundment (Hien. P. D et. al, 1996). The tracer method can be adapted in the best way to the hydrological problem and situation, but there are limitations regarding the size of the investigation area and the time period of the experiment (Moser. H, 1995).

By definition, tracers are 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. The tracer is conservative in behavior. It moves in a manner similar to water without sorption to soils, sediments, or rocks, without degradation during the time frame of interest (Markus Flury, et. al, 2003, Aeby, P, et. al, 2001) and must be detectable at low concentrations to ensure high recovery rates, as well. Tracer must be sufficiently economical but not alter the environment or subject to leaching, abrasion or other alteration; or hazardous to public health or the environment. The tracer has low background concentration and is clearly discernible from the background of the system. The tracer is detectable either by chemical analysis or by visualization. The tracer generates a low toxicological impact on the study environment.

Ideal tracer has properties such as no loss, no delay and having the same compound as traced fluid. An ideal tracer is nontoxic, inexpensive, moves with the fluid in contact, easily detectable in trace elements, does not alter the natural flow direction, is chemically stable for the desired length of time and for most purposes is neither filtered nor sorbed by the solid medium through which the fluid moves (H. Moser, 1995, Hotzl, H. et. al., 1992, Peters, N. E, et. al, 1993).

Objectives

The general applications or objectives of Tracers are:

- Determination of sediment transport rates, hydraulic connection, direction, velocity and subsurface flow etc.
- Determination of aquifer characteristics like filtration velocity, porous velocity, porosity, permeability, transmissivity etc., linking sediment transport to hydrodynamic mechanisms.
- Stratification of aquifers
- Interconnection between solution cavities in karst areas (Turkmen S, et. al. 2002)
- Seepage through hydraulic structures
- Assessment of seepage losses through irrigation canals
- Selection of waste disposal sites, monitoring of sediment plume behaviour, assessment of the influence of man-made structure on sediment movement, wastewater treatment process studies etc.

- Estimation of infiltration for recharge studies.

2.5.1 Types of Tracers

Depending on application, tracer can be used to characterize properties of large subsurface volumes or investigate small-scale transport phenomena (Craig E, et. al, 2005). A tracer can be entirely natural like the heat carried by a plume of geothermal water or intentionally introduced like dyes placed to determine leakage source and establish interconnection.

Tracers can be broadly divided into two groups:

- i) Conventional tracers and
- ii) Isotope tracers

2.5.1.1 Conventional Tracers

Conventional tracers include strong electrolytes like Sodium Chloride (NaCl), Ammonium Chloride (NH₄Cl) and organic dyes like Sodium Fluorescein, Rhodamine-B, Rhodamine-WT, etc (Olaf Huseby, et. al, 2009) which moves with the same velocity as the ground water, and its concentration is affected only by hydrodynamic dispersion. Major ions such as chloride and bromide have been used as they behave conservatively and rarely sorb onto geological material.

Dye tracers (viz. Sodium fluorescein) are successfully used in karst and other high-permeability terrains where other types of tracers have limited use (Mull, D.S, et. al, 1988). Dyes or organic dyes have proven to be powerful tracers (Davis et al., 1980) due to their low toxicity, detectability at low concentrations and over long distances, relatively low sorption tendencies, good solubility in cold water and low cost, as well (Aeby, P. et.al, 2001). Since dye tracer is water soluble, the tracer response curve reflects the flow model in the porous media and dilution processes in the in-situ water. The dye provides information on the nature of the groundwater flow that is not produced by more conventional tracing methods. Sodium Fluorescein is orange in color, low toxicity, sensitive to Ultra-violet light and characteristic bright yellowish green colour in dilute concentrations (Divine C. E, et. al, 2005). Congo red is a red dye that may turn blue in acid waters and Rhodamine-B is a red dye similar to Fluorescein.

2.5.1.1.1 Methods of application of conventional tracers

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. The best injection point is at the well head or on the injection flow line very close to the wellhead.

The tracer techniques can be employed by utilizing two methods.

- i. Single Well or Point Dilution Technique
- ii. Multiwell Techniques

Single Well Technique (Point Dilution Technique)

The aim of point-dilution method is to inject a tracer into a well, monitor one or more down-gradient wells by collecting and analyzing ground water samples, 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 faster the ground water flow, the faster the tracer is swept from the well bore.

The concentration of a tracer decreases as a result of horizontal flow of water or by diffusion. 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. When a tracer like common salt or organic dye or any radioactive water-soluble tracer is introduced in a borehole, the decrease in the concentration of the tracer is related to the filtration velocity of the undisturbed ground water flow as shown in figure 2.33. A change in the concentration of the tracer is caused either by flow or by diffusion (Rao. S. M, 1984). The flow in the borehole consists of horizontal flow, vertical flow, density currents and flow due to artificial mixing.

In a single borehole, many hydraulic coefficients, such as flow velocity, direction of flow, vertical flow etc., can be obtained by using isotope tracer method. There are three conditions if a fissure intersects a borehole: vertical, parallel and tilt. According to each different condition, the formulation of flow velocity deduced by isotope dilution method is different (Moser, H., et. al, 1989).

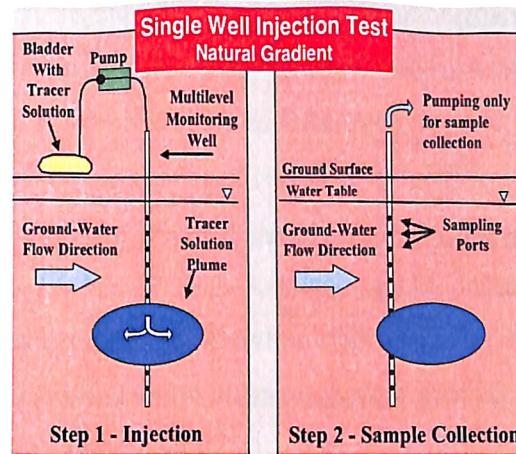


Fig. 2.33 Conceptual model showing the tracer migration in single well injection test

If only steady horizontal flow is dominant and the tracer is homogeneously distributed throughout the volume at all times, then the relation between apparent dilution rate and the concentration of the tracer is given as:

$$V_a = -\frac{V}{Ft} \ln \frac{C}{C_0} \quad (1)$$

Where:

V_a = apparent velocity

V = volume of water in the borehole in which dilution takes place

F = Area of cross section of measuring volume perpendicular to the direction of the undisturbed groundwater flow

t = time required for concentration to fall from C_0 to C

The horizontal flow pattern is distorted due to the presence of borehole. Thus, the measured velocity (V_a) has to be related to actual filtration velocity (V_f) by some additional terms, which account for the hydrodynamic distribution (Kaufman et al 1969).

The correction factor ϕ , which accounts for the distribution of the flow lines due to the presence of the borehole, is given by.

$$\phi = Q_h / Q_f \quad (2)$$

Where Q_h = horizontal flow rate in borehole in cm/sec

Q_f = the flow rate in the same cross section of formation in the absence of borehole.

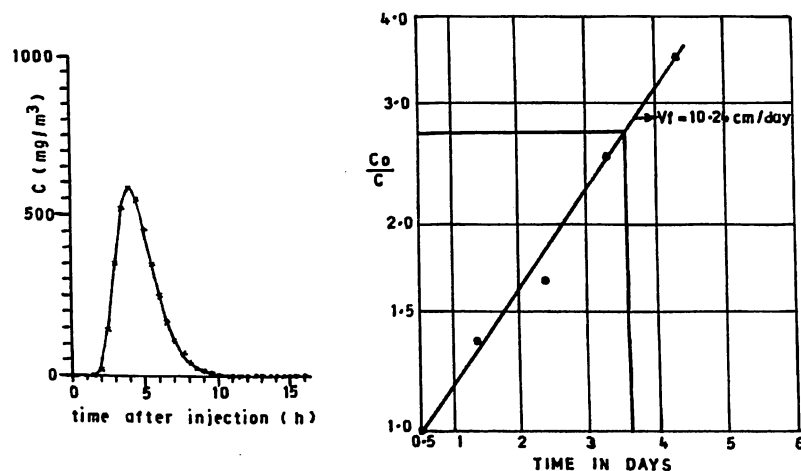
If only horizontal flow exists then, V_f is given by

$$V_f = -\frac{V}{Q_f t} \ln \frac{C}{C_0} \quad (3)$$

If packer system is used with a detector probe, then

$$V_f = -\frac{d^2 - d_s^2}{4Q_f t} \ln \frac{C}{C_0} \quad (4)$$

Where d & d_s are diameters of boreholes and detector probe respectively.



In this technique, the tracer is injected into the borehole by different methods like pouring it through a thin pipe, using a special syringe or a pump or by crushing an ampoule in the borehole at any desired depth. The tracer is then thoroughly mixed and then insitu measurements made at desired depths. Alternatively, samples are collected and then dilution of tracer is measured (Lamontagne S, et. al, 2002). A typical curve showing the change in tracer concentration with time for point dilution technique is depicted in figure 2.34.

Multi-well Technique

Tracers used for Multi-well Technique may be chemical or radioactive. However, radioactive tracers are suitable for small volume injection. This multi-well approach can yield a direct estimate of ground water flow velocity.

The method involves injecting a predetermined quantity of tracer in the form of a solution in one of the boreholes and monitoring its appearance in a number of boreholes

located at the downstream, in the anticipated direction of flow. Use of tracers through inter-well tracer testing has been established and proven as an efficient technology to obtain information on well-to-well communication, heterogeneity and fluid dynamics (Olaf Huseby, et. al, 2009, Moser, W., et. al, 1989), determine direction of flow and seepage velocity through porous medium etc. The injected quantity of tracer lasts long enough to be detected in the monitoring boreholes. The seepage velocity can be determined by knowing the arrival of the peak in the concentration versus the time curve and the distances between the injection and observation holes. Thus hydraulic interconnection between two water bodies, if any, can be established as shown in figure 2.35.

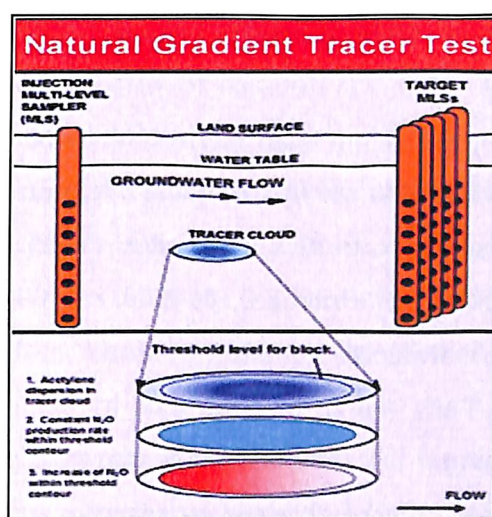


Fig. 2.35 Conceptual model showing 2tracer migration in multi well injection test

Advantages and limitations in use of conventional tracers

Advantages of conventional tracers are already discussed earlier. Chemical and dye tracers are generally affected by absorption and adsorption processes. While detecting these tracers diffusion, dispersion, are predominant and therefore many times exact detection is not possible (Rao. S. M, 1984). The sampling intervals need to be carefully and accurately planned. Further, these tracers cannot be detected over long distances. Also, their detection with leached material is difficult many times.

2.5.2 Isotope Tracers

Isotope tracers can be subdivided into stable and unstable isotopes. Stable isotopes are those isotopes that do not undergo radioactive decay with time (Ahmad. M, et. al, 2003); so

their nuclei are stable and their masses remain the same. However, they may themselves be the product of the decay of radioactive isotopes. This alters the mass of the atom but not its chemical nature. The most commonly occurring isotope has the lowest mass in the case of ^{13}C , ^2H , ^{18}O as well as ^{15}N . Stable isotopes include environmental isotopes while unstable isotopes are radioactive (e.g. ^3H , ^{51}Cr , ^{60}Co , ^{82}Br , and ^{131}I) (Gaspar et. al, 1972, Gibson, et.al, 2005).

The potential contribution of isotope methods are: (1) determination of physical parameters related to flow dynamics and system structure, (2) delineation of processes involved (process tracing) during flow and circulation of water, (3) study of origin (genesis) of water, mixing ratios of component flows (component tracing) and (4) study of "Time-scale" of events (Y. Yurtsever, et.al, 1993, Kendall, C. et. al. 1998).

2.5.2.1 Environmental Isotopes

Human-applied tracers are different than environmental tracers. Isotopes which are naturally produced and incorporated into the hydrological cycle or are released inadvertently to the environment through human activities, are often referred to as "Environmental Isotopes" (Markus Flury, et. al, 2003, Pritchard, J, et.al, 2000). Environmental isotope techniques are based upon measurements of the variations in the environmental isotope compositions of natural water. They cannot be controlled by man, but can be observed and interpreted to gain valuable regional information on the origin, turnover and transit time of water in the system which often cannot be obtained by other techniques (Coplen T.B, 1993). Analysing and interpreting the chemistry of water can provide valuable insights into groundwater-surface water interactions. Dissolved constituents can be used as environmental tracers to track the movement of water. This relies on the measurement and interpretation of background concentrations of the chemical constituents of water, such as major ions, stable or radioactive isotopes.

The application of environmental isotope in seepage studies comprises the use of stable and radioactive isotopes of the water molecule and its solutes (Gasper et al. 1972). They have same physico-chemical behavior as normal water molecules and are ideal geochemical tracers of water because their concentrations are usually not subject to change by interaction with the aquifer material. So by comparing the isotopic composition of seepage water and its suspected sources, it is possible to confirm or rule out the possibility of interconnections (IAEA, 1983, Y. Yurtsever, et.al, 1993, Clark and Fritz, 1997; Scanlon et al., 1997; Ka"ss, 1998).

2.5.2.2 Radioactive (unstable) Isotope Tracers

Radioactive isotopes spontaneously emit radiations and in a chain process, it is transformed to atoms of other radioactive elements, till a stable atom is formed. Radioactive tracers should have an optimum half-life period; i.e. long enough duration for the experiment to be conducted and short enough so that the radioactive contamination is minimum for the period of the experiment and later should be negligible (Plata-Bedmar. A, 1988). The example of radioactive tracers is Bromine-82, Iodine-131, Cobalt-60, Rubidium, Hydrogen-3 (Tritium) etc. Injected radioactive tracers like Tritium, $K^{131}I$, $NH_4^{82}Br$ and $K_3[^{60}Co(CN)_6]$ satisfy the physico-chemical behavior requirements of a good water tracer. Ease and speed of measurement are the most significant advantages of the radioactive tracer technique (Lichner. L, 2001). As gamma emitters can be detected in situ, ^{82}Br is the most commonly used artificial isotopic tracer in view of its short life span (36 hrs) and high-energy gamma emission. Sometimes, radioactive tracers (^{198}Au and ^{140}La) are removed from the water and adsorbed on the soil matrix as the water enters the soil during seepage and are particularly useful to locate areas of high seepage (Kamble et al., 2003, Gibson, et.al, 2005).

Radon gas can be used as a natural tracer of ground water seepage as the groundwater has a higher concentration of ^{222}Rn than surface water (Kraemer et.al, 1998).

Radioactive tracer techniques present advantages:

First, their concentration can be measured with detectors located outside pipes or vessels. Second, measurement of samples taken from a flow is simple and independent of the matrix of samples. Third, a radioactive tracer is unique for labeling specific elements or chemical species.

Methods of application of Radioactive Isotope Tracers

The different methods of application of radio-active isotope tracers are

1. Point Dilution Technique
2. Point Injection Technique
3. Labeling of whole water column
4. Radioactive Cloud Migration Method
5. Radioactive Tracer Adsorption Method

among which only Point Dilution and Point Injection methods are discussed here.

(1) Point Dilution Technique: The Point Dilution Technique, the horizontal component of the groundwater flow velocity is obtained from the decrease in the concentration of the tracer injected into the borehole due to the flow perpendicular to its axis, as a function of time.

In principle, the method is based on (1) the groundwater flow at the borehole site is in a steady state, (2) tracer concentration at any given moment in all points within the volume is same, (3) absence of significant losses of tracer (Lichner. L, 2001).

(2) Point Injection Technique: This method is based on the execution of tracer injection at different points of the seepage prone area and measurement of the eventual break through curves at the existing spring or downstream boreholes. Each tracer injection provides information of the water leakage corresponding to a small area close to the injection point (Kimball, B.A., 1997). This technique is particularly useful in quantifying sources of pollutants that may be entering the stream from tributaries, seeps, or from ground water.

2.5.2.3 Advantages of Isotope Tracers

The unique properties of isotopic tracers make them an ideal tool to trace the water movement in hydraulic structure, used for flow measurements in rivers and canals and estimate groundwater velocities, as well. The ability to study widespread effects has generally made naturally occurring tracers more useful and more environmentally accepted than artificially introduced tracers.

Isotopes are suitable for studying most sorts of pollution because of wide range availability. Sometimes they are a quite unique tool with significant advantages in comparison with other tracers. One important advantage of isotope techniques is that they provide the chance to estimate, qualitatively and quantitatively, what are called "retardation parameters" of geological media with respect to the movement of pollutants (Dubinchuk, et.al, 1990). Isotopes for hydrological studies include the stable isotopes of water (^{18}O , ^2H), exhibit systematic spatial and temporal variations of isotope fractionations that accompany water-cycle phase changes and diffusion. Isotope fractionation produces a natural labelling effect within the global water cycle that has been applied to study a wide range of hydrological and climatic processes at the local, regional, and global scales (Gibson, et.al, 2005).

2.5.3 Dam seepage

Dams have been a part of the economic development model of almost all nations of the world. At some stage of their development, most countries with water resources that can be economically exploited have built dams for energy, irrigation, and drinking water. In developing countries like India, constructing dams form a critical component in building the infrastructure of the nation. There are approximately 5125 large dams (4728 completed and 397 under construction) and several thousand smaller dams. However, like all pieces of structures, dams age and deteriorate, posing a potential threat to life, health, property, and the environment. Although, in India a sound foundation was laid for a nation-wide systematic dam safety surveillance programme in 1979, and maintenance and upkeep of the dams have been started recognizing dam safety as an important activity, significant funds are not provided for dam safety rehabilitation.

The hydraulic structures viz. dam, reservoir, canal etc. are although designed not to seep or leak, still minimum seepage or leakage occurs through these hydraulic structures when founded on pervious foundations.

2.5.3.1 Tracer techniques for dam seepage investigation

Tracer injection studies in dams are usually carried out by injecting a predetermined quantity of tracer, preferably dye tracer like Rhodamine-B ($C_{28}H_{31}ClN_2O_3$) or Sodium Fluorescein ($C_{20}H_{10}Na_2O_5$) at specific depth within the reservoir or in borehole drilled through the body of the dam. The depth at which the tracer is to be injected is deciphered based on borehole logging test results and or any other NDT (Non Destructive Test) results. Predetermined quantity of desired tracer is tied to a weight (cylindrical pipe or rod) and is lowered to the desired depth either in the reservoir or borehole drilled through the body of the dam. A conceptual model showing the field setup of tracer injection at dam site is shown in Fig. 2.36 along with the probable path through which the migration of tracer.

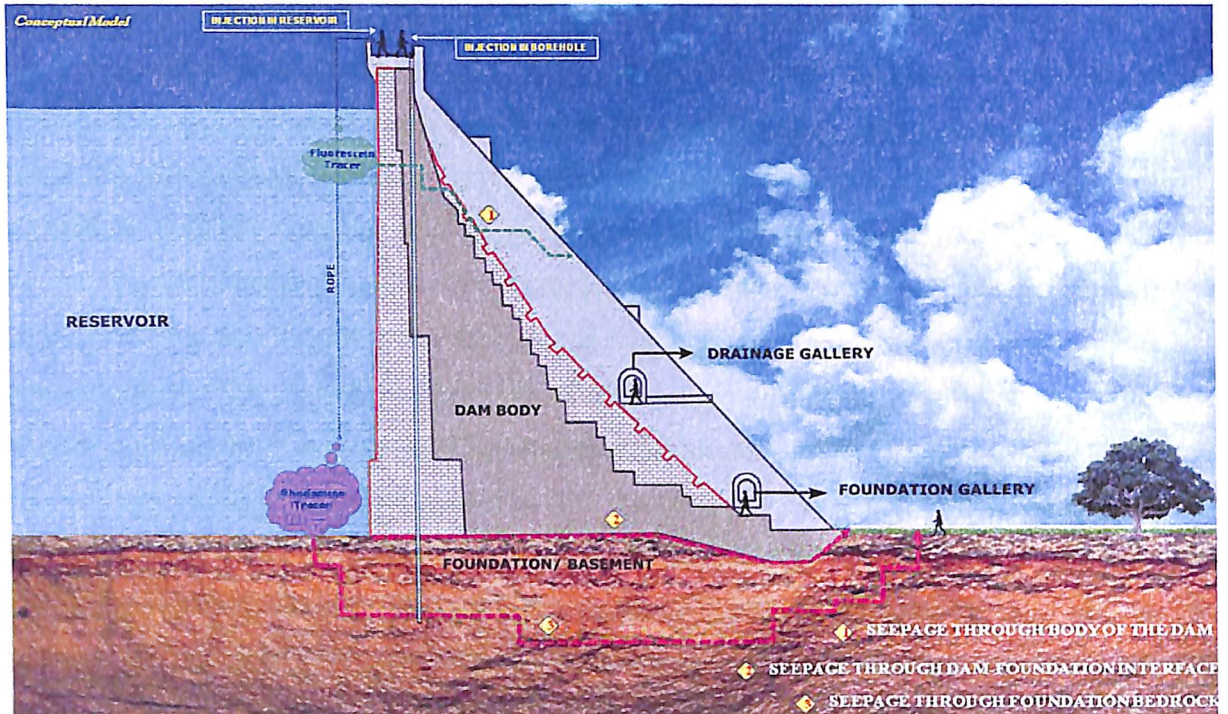


Fig. 2.36 Conceptual model showing the field setup of tracer injection at dam site.

A typical graph depicting the tracer (sodium fluorescence) arrival at a known seepage location at different times is shown in figure 2.37. The velocities were calculated from the peak concentration and the seepage losses were estimated.

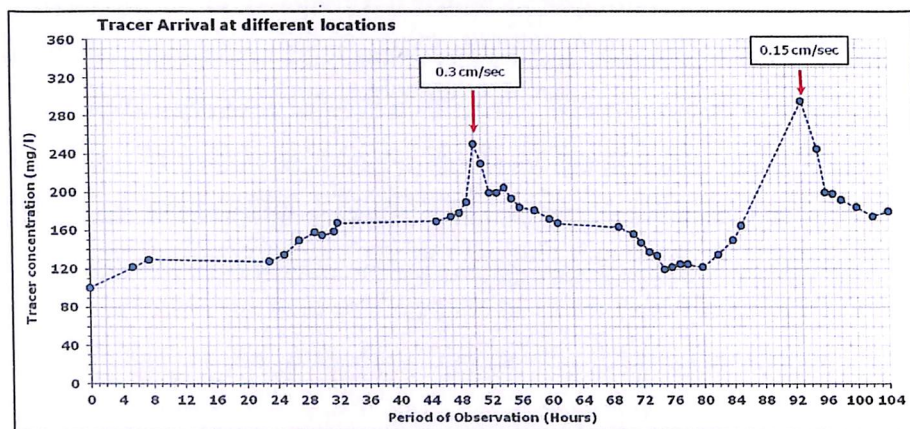


Fig. 2.37 Tracer Arrival curve; concentration (vs) time

2.5.4 CW&PRS Experiences

CWPRS is actively involved in seepage investigations for hydraulic structures since 1962 and had given solutions and remedial measures to the problems. Following, few typical case studies are cited to provide insight and approach to the seepage problems in the hydraulic structures.

2.5.4.1 Delineating path of seepage in the masonry at Pawana Dam, Maharashtra

The Pawana Dam of height 38.1 m was constructed across River Pawana, Maharashtra. It is a composite dam comprising 414 m long masonry dam with overflow and non-overflow portion and a 903 m long earthen dam. Excessive seepage which was related with the rise of the reservoir level was observed in the drainage gallery and body of dam. Hence grouting at a few places and guniting of the entire upstream face was done to reduce the seepage which was actually reduced by about 50%. For the purpose of strengthening the dam and to raise the Full Reservoir Level by 0.5 m, tracer studies were carried out to delineate the path of seepage. Figure.2.38. shows the layout of dam and excessive seepage observed at drainage gallery and downstream of the dam. Seepage was observed like water jets on the downstream slope of Left Hand Side (LHS) of masonry between Ch.725 ft. and Ch. 849 ft. in monoliths 5, 6 & 7 and between RL 1935 ft and 2004 ft. The leakage was observed from a number of pores holes on the RHS gallery but no appreciable seepage was seen in the LHS drainage gallery. This is an example of seepage through structure. Tracer studies were carried out in two phases during the year 2003, viz. May-June when the reservoir level was between 1975 ft. and 1976.30 ft. and in September when lake level was around 2010 ft. The appearance of Potassium Permanganate dye tracer was monitored at downstream observation points at every 10 minutes interval for about two hours after injection.

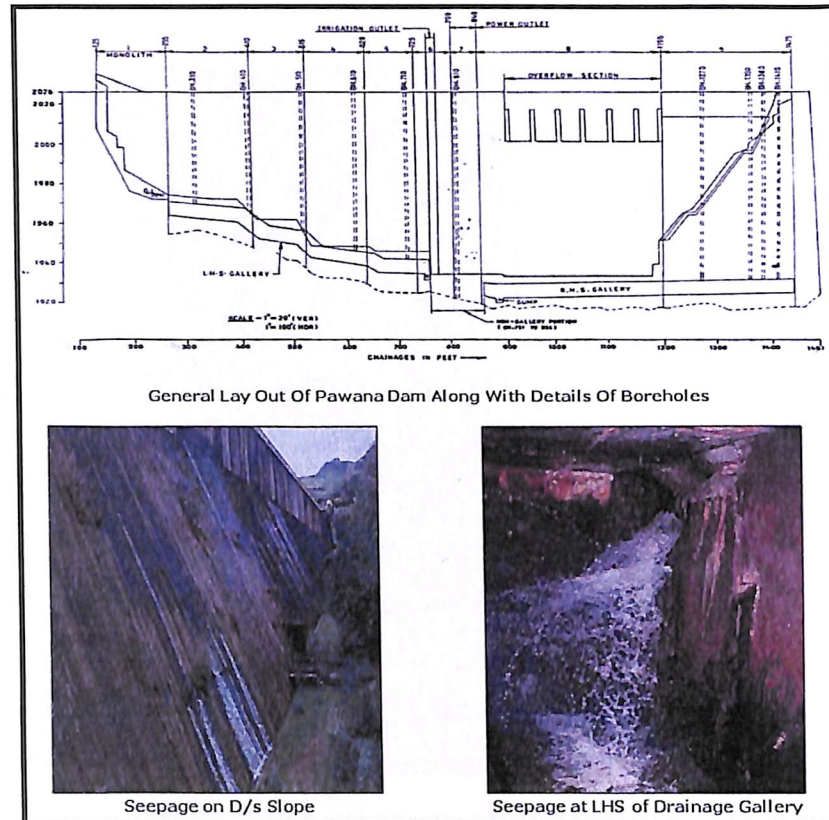


Fig.2.38 Dam layout with visible seepage sites along the downstream

The results indicated that the tracer injected at Ch.225 m (RL 590 m) had a direct connection with water jet in the LHS drainage gallery and small quantity of tracers were also observed at the downstream slope for the tracer injections at Ch.250 m (RL 598 m & 594 m), Ch. 259 m (RL 598 m). The tracer injected at other locations has not arrived at the downstream monitoring points, indicating that there is no direct interconnection between these points. The direct arrival of dye for injection in borehole at Ch.247 m (RL 592 m, 594 m and 598 m) may be due to the presence of gaps in masonry dam. The tracer also appeared distinctly for the injection points in a well at Ch. 226 m (RL 609 m). It was observed that except for injections on the upstream face of dam at Ch. 226 m and Ch. 230 m, dye appeared at the remaining injection points. From the nuclear and caliper logging of boreholes, 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.

Tracer studies revealed that dye injected in the boreholes, where prominent cracks/ voids were detected by nuclear logging, arrived at leakage points in the downstream slope of dam. It was inferred from the studies that cracks / voids in masonry might have occurred due to process of leaching and seepage also was occurring through these portions. It was

recommended to undertake controlled grouting in the body of dam where prominent cracks/voids were located to reduce/ stop seepage.

2.5.4.2 Ascertaining seepage path in the foundation at Nagarjunasagar Dam, Andhra Pradesh

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 (3418 m in length) in the centre of the gorge flanked by earth dams. The reservoir formed upstream of the dam is the largest man-made lake in the country and third largest in the world (Fig. 2.39).

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. Technical Experts Committee (TEC) was appointed by A.P Govt. to examine the cause of the formation of the cavity. As suggested by TEC tracer studies were conducted for determining the interconnection if any, and the seepage velocity through the earthen dam. The area under study lies in the sedimentary terrain of Cuddapah group with quartzite as a predominant rock type.

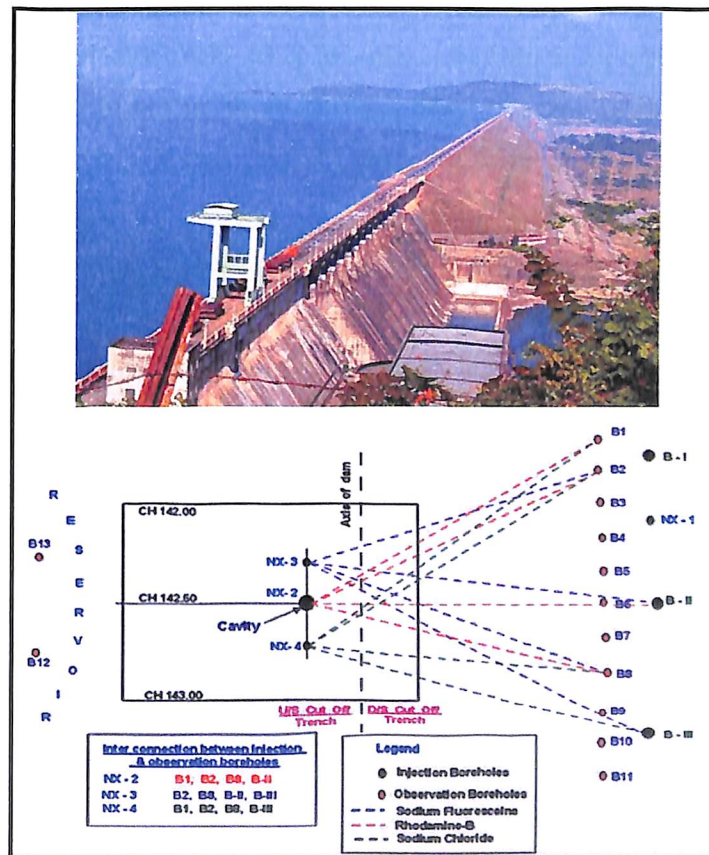


Fig. 2.39 Plan of Location of Boreholes for Tracer Studies at Right Earth Dam, Nagarjunasagar Project, A.P.

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 (B1.....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 , Nx-5 was drilled to monitor the flow of the injected tracer. B12 and B13 were drilled at the reservoir of upstream side to monitor movement of the tracer towards upstream. Tracers used were sodium fluoresceine, Rhodamine-B and common salt.

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 as shown in figure. 2.40. Borehole logging also revealed that the foundation rock below cavity (RL 544 – 539ft.) was prone to the excessive seepage. It was recommended that a suitable treatment should be given for the foundation at cavities zone to reduce the seepage through them.

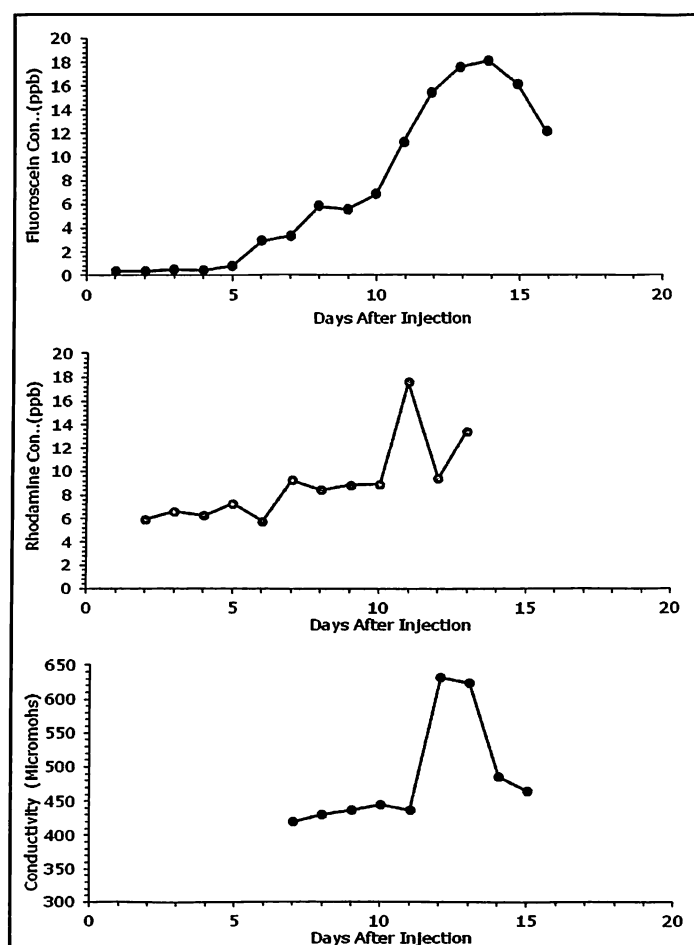


Fig. 2.40. Arrival of Sodium Fluorescein, Rhodamine and salt tracers in Nx Bore Hole B1 and B2 respectively

2.6 TESTING OF CONCRETE/MASONRY CORES IN LABORATORY

The laboratory concrete mix strengths are determined in controlled condition and are always within the range of design value. But in-place concrete strengths are a function of several factors such as concrete mixture proportions, curing conditions, degree of compaction, unforeseen conditions at placing site and deterioration over time. Hence the strength of in place concrete is likely to fall below the design value though it is not expected even when a good quality control is exercised. The condition of the in place concrete and extent of distress is indirectly assessed by its strength testing. Hence even though the strength of the concrete is measured regularly during construction by casting cubes at site, determining strength of insitu concrete of structures is also necessary. A number of methods are available to determine strength properties of insitu concrete such as non-destructive

method, insitu density method, core extraction method etc. each having its merits and demerits. However, for the structures using mass concrete mixes extraction of cores and determining their properties such as density, compressive strength, & static & dynamic modulus of elasticity is a realistic and acceptable method.

A core is extracted by means of a rotary cutting tool fitted with diamond bits driven by a electric motor or petrol/diesel engine. The physical properties such as density (ρ), static values of Young's modulus of elasticity (E_{static}), Poisson's ratio (ν_{static}) unconfined Compressive strength, Split tensile strength, and dynamic values of the Young's modulus of elasticity ($E_{dynamic}$) and Poisson's ratio ($\nu_{dynamic}$) of concrete/masonry are an important consideration in the analysis and review of the safety of concrete/masonry dams to predict the behavior of a structure under various static and dynamic loading conditions.

The Laboratory test procedure used for measuring physical properties in Laboratory are described below;

2.6.1 Concrete core

Normally concrete core of 15 cm diameter is extracted from dam body for laboratory evaluation.

Density (ρ)

The density, defined as mass per unit volume, was obtained simply by taking the weight of the core and calculating its volume from the dimensions.

Static Young's Modulus of elasticity (E_{static}) and Poisson's ratio (ν_{static})

Cylindrical core with length/ diameter of 2 shall be prepared such that the circular end faces were plane and perpendicular to the axis of the cylinder. The Young's modulus of elasticity and the Poisson's ratio of core specimens shall be then determined as per the procedure laid down in IS: 516-1959. For this purpose, the axial and the lateral deformations of the specimen were measured for each load step by gradually loading the specimen in a compression testing machine up to a maximum load of $1/3^{rd}$ of the its design compressive strength, obtained by testing of some concrete core samples. Deformations were measured either by fitting an extensometer or by strain gauge to the core specimen (Fig. 2.41). The

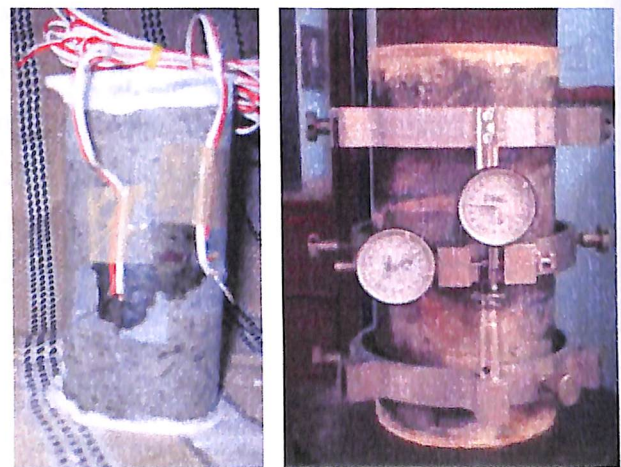


Fig. 2.41 Specimens with strain gauges & with Combination Extensometer

modulus of elasticity was determined from the stress–strain plot, where stress was taken as the load per unit area of cross section of the test specimen and longitudinal strain was taken equal to change in length for a given load divided by the length of the specimen. The Poisson's ratio (ν_{static}) was obtained as the ratio of the lateral strain and the longitudinal strain, where lateral strain was taken as the lateral deformation divided by the diameter of the test specimen.



Fig. 2.42 Compression test set up

Compressive Strength

The cores shall be tested under compression testing Machine. Load shall be applied gradually till the failure of the specimen as per procedure laid down in IS: 516-1959. The compressive strength of the core material was taken as load per unit of the area of cross section of the specimen at failure (Fig. 2.42). Correction factor was applied as per graph given in IS:516 wherever L/D ratio was less than 2.

Split Tensile Strength

The split tensile strength shall be determined as per the procedure laid down in IS: 5816-1970. Accordingly, a core sample of length equal to about its diameter shall be subjected to increasing load along a diameter till failure using compression testing Machine (Fig. 2.43).

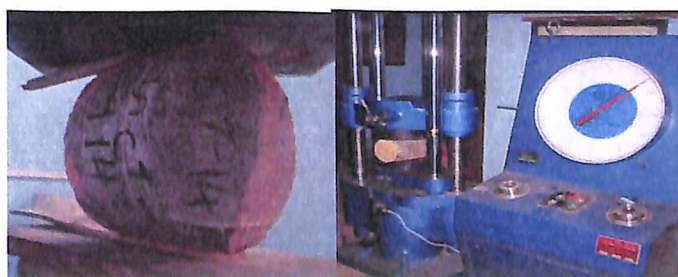


Fig. 2.43 Split tension set up

The split tensile strength (T_{sp}) is calculated from the formula $T_{sp} = 2P/\pi dl$
Where, P is the maximum load, d the diameter and l the length of specimen.

Dynamic Modulus and Poisson's Ratio

Dynamic values of the Young's modulus and Poisson's ratio shall be estimated using resonance frequency method as suggested in ASTM C 215-02 guidelines. The method is based on measuring the fundamental resonance frequencies of the test specimen in longitudinal, transverse and torsional vibrations. Figure shows the photograph of the test setup used for measurement of resonant frequency of test specimen. The fundamental longitudinal frequency (n') of a cylindrical test specimen of diameter d , length L , and mass M is related to the dynamic Young's modulus of elasticity (E_d) as follows:

$$E_d = 5.093(L/d^2)Mn'^2$$

Also, the fundamental torsional frequency (n'') of a cylindrical test specimen of cross sectional area A , length L , and mass M is related to the dynamic modulus of rigidity (G_d) as follows:

$$G_d = \frac{4L}{A}Mn''^2$$

The dynamic Poisson's ratio (ν_d) can be obtained from the modulus of elasticity (E_d) and modulus of rigidity (G_d) as follows:

$$\nu_d = \frac{E_d}{2G_d} - 1$$

To obtain the resonant frequencies in longitudinal and torsional modes, the specimen is supported at the center on the test bench and vibrations were imparted to the specimen at one end of the specimen by the vibrator through a physical contact and the response is picked up at the other end with an accelerometer (Fig. 2.44).

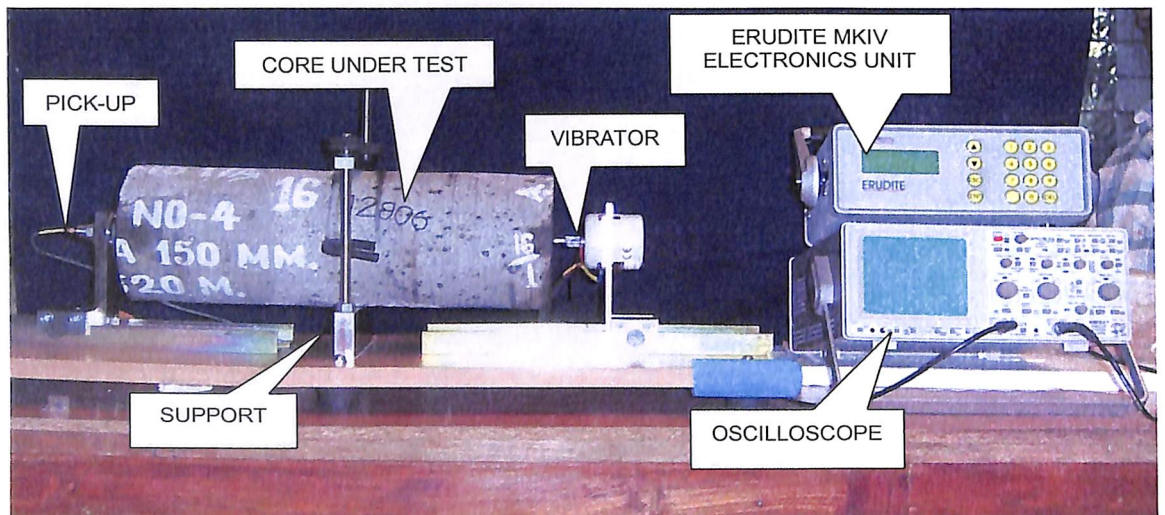


Fig. 2.44 Test setup for longitudinal mode of vibrations for concrete core.

2.6.2 Masonry core

For masonry concrete, normally compressive strength testing is performed on drilled cylindrical cores of size 90 cm dia x 90 cm height ($L/D=1$) from different zones of masonry portion and then the strength obtained is converted for $L/D=2$ using conversion curve given in IS-516-1959. Whereas in order to determine the strength of masonry of the dam, it is required to extract cores of size 45 cm dia. x 90 cm height, vide IS 6512 – 1984 – Criteria for design of solid gravity dams as detailed below:

“5.13.2.2 Compressive strength – The compressive strength of masonry shall conform to the following requirements:

(b) Masonry- The compressive strength of masonry is determined by compressing to failure 75 cm cubes of the masonry fabricated and cured at temperatures approximating to those expected in the structure (or 45 x 90 cm cylinders cored out of the structures or blocks made for the purpose). This strength should satisfy early load and construction requirements and at one year it should be five times the maximum computed stress on the dam or 12.5 N/mm^2 whichever is more.”

2.7 LABORATORY TESTS FOR ALKALI AGGREGATE REACTION

Concrete is a basic structural component of almost all dams. Concrete structures for dams include the intake structures, outlet conduits, and outlet works (most all dams); service and emergency spillways (embankment and concrete dams); and the main body or in some cases an impermeable core (concrete and embankment dams). Concrete deterioration encompasses physical processes that are constantly acting on the concrete with time, or an internal condition that may affect the physical structure of the material itself (Dolen, 2003). The majority of the old concrete dams exhibit ageing problems and causes Alkali Aggregate Reaction (AAR).

There are two types of AAR --- Alkali Silicate Reaction (ASR) and Alkali Carbonation Reaction (ACR). Among two, ASR is more common in the concrete structure.

Alkali Silica Reaction (ASR)

Alkali-silica reaction (ASR) is chemical reaction between the reactive silica present within aggregates and alkalis in the cement. The product is alkali-silica gel. The mechanism is the silica gel imbibes water and expands. As a result enormous pressure is generated within concrete which develops cracks (Fig. 2.45).

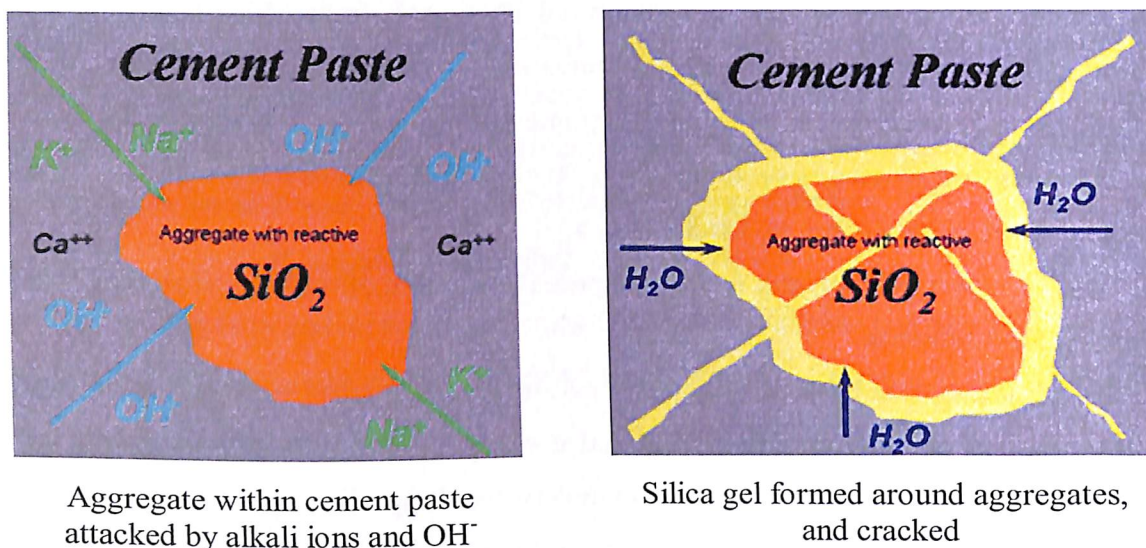


Fig. 2.45 ASR activity within cement paste and aggregate

The major controlling factors of ASR are (1) reactive silica present within the aggregates, (2) sufficient alkali ions present within the pore solution and (3) sufficient moisture i.e., relative humidity should be higher than 80%. This chemical reaction takes place over a period of time. The external water which passes through the major and micro cracks as a result of ASR, reach at the reaction sites. So the chemical reaction, swelling, expansion and cracking of concrete are continued with time which ultimately causes deterioration of the concrete structure after a certain period of time, if not properly maintained in time. ASR reduces the load bearing capacity and the fatigue life of concrete, enhances the shear capacity of reinforced concrete with and without shear reinforcement. The losses in elastic modulus between 20% and 50% were found for expansions of 1-3mm/m.

Alkali Carbonation Reaction (ACR)

ACR is a reaction between the alkali hydroxides in cement and certain dolomitic limestone aggregates (i.e. reactive aggregates) under low pH or acidic environment. The important constituents are CO₂, Ca⁺² and H₂O. This is a continuous and slow process. They promote corrosion of steel reinforcements.

2.7.1 Reactive Aggregates

Aggregates are naturally occurring rocks and minerals. The siliceous, porous and glass type aggregates are reactive in nature and undergo more chemical reactions. The

chemical reactions between alkali ions and the dissolved silica occur very rapidly. Table- 2.4 shows Alkali reactive (A) silica minerals and rocks and (B) carbonate rocks.

Table – 2.4

Classification of Alkali Aggregate Reactivity
A. Alkali reactive silica minerals and rocks

A₁. Alkali-Reactive Metastable Silica Minerals and Volcanic Glasses	
Reactant	Opal, tridymite, cristobalite, volcanic glasses
Rocks	Opal-bearing rocks such as shales, sandstones, silicified carbonate rocks, chert; rhyolites, dacites, latites, andesites, glassy groundmass basalts.
Expansion	Mainly caused by hydraulic pressure from expansion.
A₂. Alkali-Reactive Quartzose Rocks	
(a) Silica Rocks	
(b) Silicate Rocks Rich in Quartz	
Reactant	Chalcedony, quartz; microquartz (0.01 to 0.05 mm in size, i.e., fine to coarse)
Rocks (a)	Chert, chalcedony and/or microquartz; vein quartz, quartzite, quartzarenite.
(b)	Volcanic rocks as under A ₁ , argillites, siltstones, greywackes, arenites, arkoses, slates, phyllites, (quartz-mica) schists, granites, granite and grano-diorite gneisses, charnockites.
Expansion	Caused by hydraulic pressure from expansion.

B. Alkali-reactive carbonates rocks

Reactant	Dolomite (metastable; process dedolomitization)
Rocks	Dolomitic Lst., dolostones; quartz-bearing argillaceous; calcitic dolostones
Expansion	Due to water uptake by dry clay minerals and other micro- constituents, or other physico-chemical causes.

BIS Code 383-1970 – Specification for Coarse and Fine Aggregates from Natural Sources for Concrete (ICOLD, 1991, BIS 383, 1970).

In view of recommendation of BIS 383, 1970 under note in para 3.2 (Page 6), “The aggregates which have reactive tendency should be avoided or used only with cement of low alkali content (not more than 0.6 percent as Na₂O) after detailed laboratory studies”.

2.7.2 Supplementary Cementing Material (SCM) or Pozzolan

Supplementary Cementing Material (SCM) or Pozzolan is nothing but mineral admixture. They are highly reactive, siliceous or siliceous – aluminous fine material. The major activity of SCM within concrete is reduction of (a) density of cement, (b) calcium within pore solution, (c) pore solution alkalinity (d) ionic mobility and water permeability and also (d) expansion.

The common SCM presently used are silica fume (sub-product from ferrous-silica alloy industry); granulate blast furnace slag (sub-product from steel fabrication) and fly ash (sub-product from coal fired power stations). Besides artificial sources, the natural sources of SCM are clays (metakaolin) and natural volcanic pozzolans etc. Presently Lithium is the most effective SCM as they react with silica gel rather than free lime and form lithium-silica gel which is non-expansive in nature in the presence of moisture and forms a protective shield around the aggregates to prevent any sort of contact with the alkaline pore solution. But since the availability of lithium in the free state is very rare and extraction of the same from other rock (Ex. Pegmatite) is quite costly, its use is very restricted in this world.

2.7.3 Analysis of AAR

As AAR is a slow but continuous process, it is very difficult to identify at the initial stage. For analysis of AAR both field and laboratory investigations are required. Both investigations provide us damage extent and severity, aggregate involvement and mechanism involved etc.

a. Field Observation

Visual symptoms on concrete structures affected by ASR and ACR are generally similar. Both affect the concrete of the dam in the form of expansion, deformation, extrusion of joint seals, closing of joints, differential movement and gate blockage etc. Map cracking on the wall with radial fashion having broad brownish zone with permanent dampness, surface discoloration, gel exudation on the surface of the concrete particularly at joints and through cracks are the indication of ASR (Fig. 2.46). Generally cracking due to AAR is a slow to appear, sometimes taking years to appear. However, in the presence of certain reactive aggregates, high alkali level in the cement and also due to the effect of surface shrinkage, cracking occurs more quickly.

b. Laboratory investigation

Few laboratory tests are discussed below in Table 2.5.

Table – 2.5

Sl. No.	Laboratory test	Nature of test
1	Petrographic analysis	Determination of existence of ASR under the microscope (Fig. 2.47).
2	Mortar-Bar Method	Determination of reactivity of an aggregate.
3	Accelerated Mortar-bar expansion test (AMBT)	Determination of potential ASR of combinations of SCM and aggregate.
4	Quick Chemical test	Determination of potential ASR of an aggregate
5	Soundness test of aggregates	Measurement of aggregate resistance to disintegration
6	Ultrasonic pulse velocity (UPV)	Assessment of cracking and voids, its severity and general soundness of the concrete using frequency range of 20–150 kHz.
7	Carbonation Test	Determination of effect of carbonation within concrete using phenolphthalein indicator solution
8	Evaluation of reactive aggregates	Evaluation based on particles size and shape, estimation of deleterious materials and organic impurities, specific gravity, density, voids, absorption and bulking, mechanical properties, soundness, measuring mortar making properties of fine aggregates, alkali aggregate reactivity – chemical test and petrographic examination according to the BIS Code 2386, 1970.

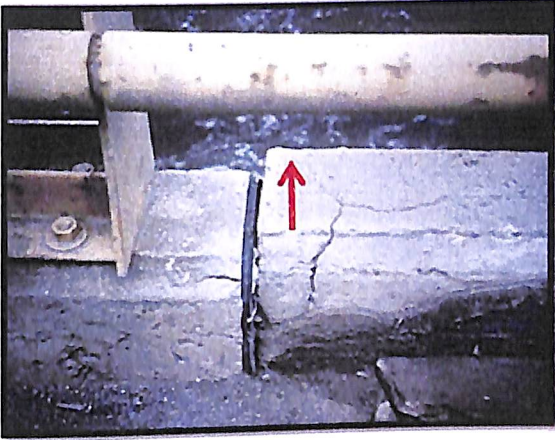
The core samples are collected from the affected or suspected area of the concrete after visual observation (Field inspection) and sent for laboratory test.

2.7.4 Effect of Alkali Aggregate Reaction (AAR) on dam

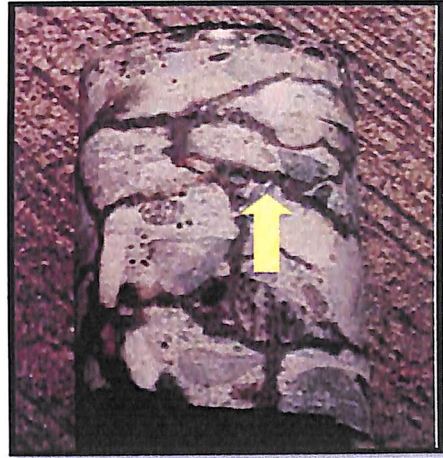
AAR affected concrete structure losses gradually its strength and elastic properties. The dam has a prominent moisture profile or gradient from upstream to downstream and plenty of supply of seepage water as external water which aggravates AAR reaction.

Among concrete dams, (a) Arch dam is affected by increased bending stresses or cracking, concrete block movements, loss of arch action etc. and then collapse. (b) Gravity dam suffers from loss of bond along lift lines, cracking at the heel, increased uplift pressures and next sliding failure etc. and (c) Buttress dam is affected from collapse of slabs or bending of arch barrels, eccentric loading on struts or slabs leading to failure by buckling or buttress failure etc. The failure is generally triggered under earthquake (Dolen, 2003).

Among Spillways and Outlets, (a) Spillway Gate Structure is affected by expansion and cracking of support concrete which leads to loss of anchorage or failure of supports, collapse of



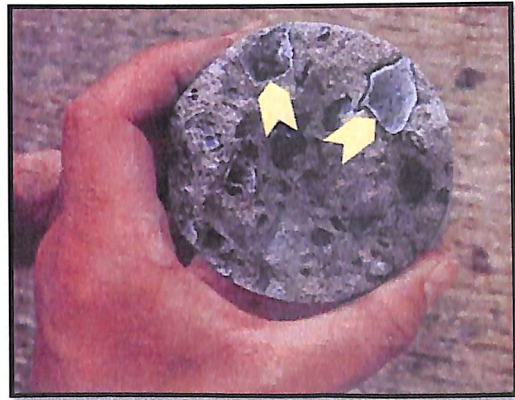
Deformation, relative movement, and displacement



Macro cracks in core sample



Map cracking on concrete wall



Deposits of alkali-silica gel



Surface Pop outs

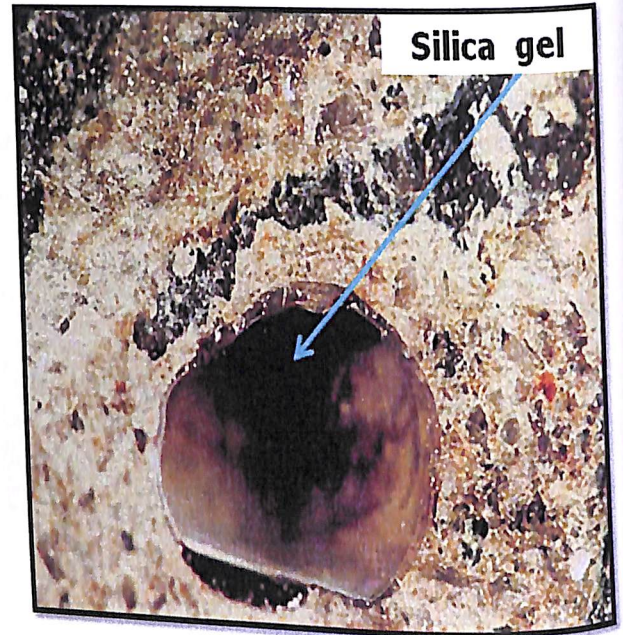


Surface discoloration

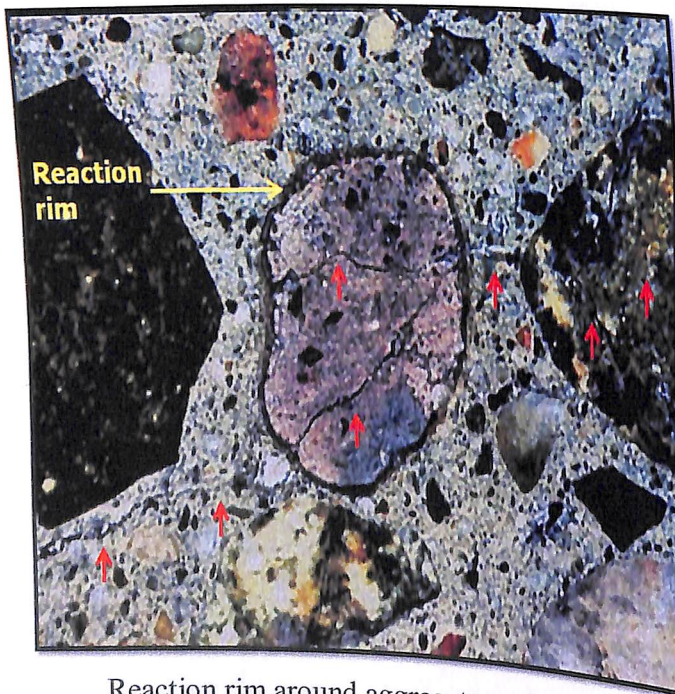
Fig. 2.46 Common visual symptoms of ASR activity



Cracks propagated from one aggregate to other through cement paste
(Arrow indicates presence of crack)



Silica gel (under microscope)



Reaction rim around aggregate with cracks
(Arrow indicates presence of crack)



Strained quartz --- three-way undulatory cracks under microscope

Fig. 2.47 Thin rock samples under microscope (Petrographic analysis)

the gates, failure upon operation (i.e., gate arm buckles and hinge pin fails.) etc. (b) Spillway Chute and Stilling Basin shows buckling or misalignment of slabs or walls which causes large flows under the concrete, plucking of the concrete and leading to erosion of the underlying material. The failure generally occurs during floods. (c) Outlet Conduit (generally in embankment dam) is affected from misalignment or localized collapse of the conduit, expose embankment materials to outlet flow and causes leading to undetected erosion etc (Dolen, 2003).

2.7.5 Mitigation of AAR

Old/ existing concrete structure

Continued or reactivated AAR causes expansion, which in turn reactivates pre-existing cracks. AAR-cracked concrete is unusually permeable. If water penetrates the structure and gets

trapped behind the patching material, the trapped water may cause hydrostatic detachment failure or result in freeze/ thaw deterioration. There are three major duties are to be performed to manage AAR affected structures viz.

- | | |
|-----------------------------------|---|
| 1. Inspection programme | periodic visual inspections, sampling and lab testing (petrography, mechanical and expansion tests) |
| 2. Instrumentation and monitoring | crack development, evolution (mapping), deformations (joint meters), extensometers, plumb lines inverted or standard, concrete stress measurement, reinforcing steel strain measurement |
| 3. Structural evaluation | estimate structure sensitivity to expansion/ need for detailed analysis. |

Damage Control is possible by repairing the existing cracks by grouting or epoxy resin injection (repetitive) and also by reduction of surface freeze- thaw effects by coating, by using structural reinforcement and cosmetic coverings, injection of concrete with lithium compounds (SCM), using suitable aggregates etc (Bleszynski, 1998). The siloxane paint which acts like a one-way valve makes the concrete surface water vapor permeable but liquid water impermeable (Lama, 1978). Structural modification is done by installing anchors or rebar for stability, slot cutting to accommodate expansion and also by partial replacement of

the affected portion. Reaction control (long term) is possible by surface treatment (membranes/coatings) and also by injection of lithium.

The most common measurements undertaken in concrete dams are displacements, tilts, strains, total stresses, pore pressure, flow, temperature, crack width and depth evolution etc. After the expansion ceases when all the alkali components are consumed by the reactions, repairs can be performed to rehabilitate and restore the structure to full operation and serviceability.

New Concrete Structure

For new construction, use of nonreactive aggregates and controlling alkali content within concrete by using SCM is major steps. We can use fly ash, ground iron blast furnace slag, microsilica as SCM to reduce alkali content of the cement below 0.6% or blended cements, low alkali cements etc. In this context, recycled concrete, or crushed waste concrete, is a feasible source of aggregates and an economic reality, because concrete is 100% recyclable, especially where good aggregates are scarce. New concrete made from recycled-concrete aggregate which is not already affected with AAR and tested properly before use generally has good durability.

2.7.6 CW&PRS Experiences

2.7.6.1 Kabbur Tunnel, Karnataka

The 3.315 km long 3.2 m x 3.2 m size tunnel has been excavated as a part of Chikkodi Branch Irrigation Canal. The area around the tunnel comprises variants of basalt of Cretaceous to Eocene age overlying recent thin pile of black cotton soil. It was proposed to use the tunnel muck as concrete aggregate. The tunnel muck was tested to assess suitability. The petrographic analysis is done by CW&PRS, Pune (CW&PRS Report No. 2535).

2.7.6.1.1 Laboratory studies

Laboratory tests of the samples indicated that there are two main rock types viz. agglomerate and altered porphyritic basalt. The test results of both the rock types are as follows:

a. Physical tests

The compressive strength of the rock specimens was determined. The dry strength varied from 527 to 2040 kg/cm² while the wet strength varied from 520 to 596 kg/cm². The

drop in strength is high, probably due to loss of intergranular bond. The results of different physical tests carried out on two agglomerate samples and on altered porphyritic basalt are as follows in Table-2.4:

Table – 2.4
Physical tests of agglomerate and altered porphyritic basalt

Sl. No.	Test	Agglomerate		Altered Porphyritic	Bis specifications (2386, 1963)
		Rock of Approach Cut and Shaft-1	Rock of Exit and Shaft-2		
1.	Specific gravity	1.99	2.33	3.05	--
2.	Absorption (%)	6.51	6.83	1.52	--
3.	Aggregate impact value (% loss)	20.51	14.3	--	30% for wearing surface; 45% non-wearing surface
4.	Aggregate crushing value (% loss)	25.1	26.8	---	30% for wearing surface; 45% non-wearing surface
5.	Los Angeles abrasion value (%loss)	44.1	---	18.1	30% for wearing surface; 50% non-wearing surface
		composite rock samples (Agglomerate + Porphyritic Basalt)			
6.	Sodium sulphate	56.0	32.0	10.12	12% of concrete-surface subject to frost action 5 cycles (% loss)

b. Chemical test

Representative samples of agglomerate and altered porphyritic basalt from tunnel muck were selected for the test. The results of the chemical test carried out at Karnataka Engineering research Station, shows that the reduction in alkalinity and the concentration of silica on two trials for agglomerate and altered porphyritic basalt ranges from 195 to 230 millimoles/ltr, 89 to 138 millimoles/ltr and 174 to 177 millimoles/ltr, 233 to 238 millimoles/ltr respectively. These are near the zone of deleterious aggregates as per the graph (BIS 2386 (Part VIII)).

c. Petrographic analysis

The petrographic analysis of 10 agglomerate and 16 altered porphyritic basalt rock samples from different locations was carried out (CW&PRS Report No. 2535). The petrographic analysis of agglomerate and altered porphyritic basalt rock samples shows that the fragments of basalt and volcanic material are embedded in pyroclastic glassy matrix (Fig. 2.48). The plagioclase laths are seen decomposed at higher magnification which may be due to the reaction between volcanic glass and sodium in plagioclase. Porphyritic basalt with

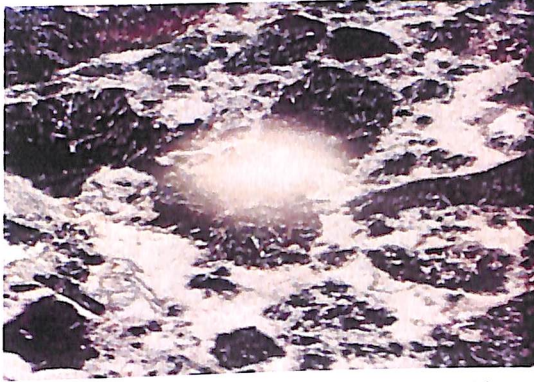
volcanic glass, the plagioclase as a predominant mineral, altered minerals and iron oxides are shown in Fig. 3.48. The porphyritic basalt with predominant plagioclase (cluster and laths) has corroded boundaries and plagioclase shows reaction rim. The sparsely spaced amygdules in porphyritic basalt are filled up with zeolites.

From all test it was concluded that tunnel muck (agglomerate and porphyritic basalt) is potentially reactive because they contain deleteriously reactive minerals (6 - 8%) and volcanic glass, as well. So it was better to avoid them.

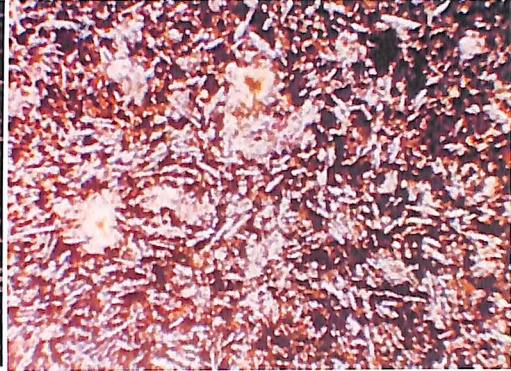
2.7.6.2 Markandeya Dam, Karnataka

Markandeya reservoir project envisages the construction of 47 m high and 475 m long concrete gravity dam with 96.8 m long central spillway on river Markandeya in Belgaum District, Karnataka, India. The rock formation at dam site comprises horizontally bedded stratified quartzite formation of Kaladagi basin precambrian sedimentary rocks. In order to design foundation for the dam, extensive field and laboratory investigations were carried out for assessment of rock mass characteristics. Determination of engineering properties, aggregate properties and petrographic examination (CW&PRS Report No. 3643) of foundation rock mass were carried out.

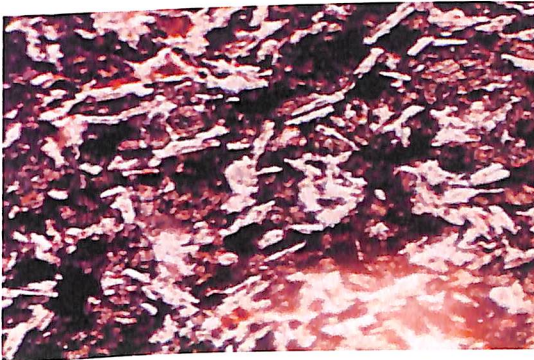
The rock samples tested in the laboratory were found to have density (γ) and unconfined compressive strength (σ_c) as 2710 kg/m³ and 139.0 MPa respectively. Though the quartzite aggregates of the dam site passed through physical tests, the results of the Aggregate Crushing



Agglomerate – angular fragments embedded in pyroclastic glassy groundmass



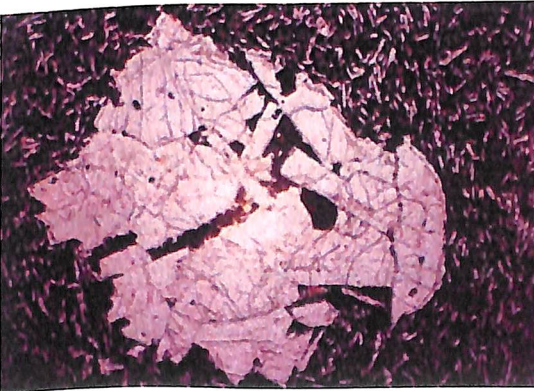
Agglomerate – decomposed plagioclase laths with volcanic glass in lithofragment



Porphyritic basalt – decomposed plagioclase, glass, altered minerals and iron oxides



Porphyritic basalt – corroded boundaries of plagioclase and altered minerals with volcanic glass



Porphyritic basalt – plagioclase cluster and laths having corroded border and reaction rim



Porphyritic basalt – amygdules filled with zeolites

Fig. 2.48 Petrographic analysis of agglomerate and porphyritic basalt

test and Los Angeles test were found to fall very close to the limit given in BIS 383, 1970. Based on chemical tests, though the aggregates were found to come under innocuous category, Petrographic examination (BIS 2386, 1970) of the same aggregates revealed the presence of reactive strain quartz.

The quartzites of Markandeya dam site come under the category of hard variety as per Dere's Engineering Classification, Petrographic examination, envisages the use of in-situ rock as aggregates in the concrete mix with low alkali cement.

2.7.6.2.1 Engineering Properties of foundation rock

The average values of density, hardness and unconfined compressive strength are 2710 kg/m³, 47 and 139 MPa respectively. Based on these parameters and Dere's Classification on intact rock strength, the rock type can be classified as medium to high strength category.

Physical Tests of aggregates

Quartzite samples from Markandeya dam site were collected from crushed aggregates for conducting physical tests viz. Density, % Absorption, Aggregate Impact Value, Aggregate Crushing Value, Los Angeles Abrasion Value and Soundness of Aggregates as per BIS – 2386 Part I to VI.

Aggregate Properties:

The quartzite aggregates indicated density around 2680 kg/m³ and water absorption of 0.3%. These values are satisfactory. Aggregate Impact value of 14% is also satisfactory compared to limited as per BIS specifications. Amongst the two samples tested, first sample showed aggregate crushing value and Los Angeles value at marginal level i.e. 29.5% and 30.12% respectively compared to 30% for both tests in the limits given for wearing surface in BIS code. Soundness test is also satisfactory except for one sample at 10 cycles which exceeds the limit of 12% for wearing surface. In general, quartzites at Markandeya dam site passes through physical tests if sorted properly.

2.7.6.2.2 Petrographic analysis:

Rock core samples for petrographic analysis (CW&PRS Report No. 3643) were collected from exploratory bore holes of foundation strata.



A: General bedding and interlocked boundaries of grains showing Mosaic Texture



C: Fan shaped re-crystallisation due to micro- tectonic effect



Fig. 2.49 Microphotos : quartzite mortar microscope

The rock type is Arenite as the presence of quartz is about 98% and matrix is less than 10%. The overall sorting of the sediment is as moderately well sorted with average medium grain size. In sample graded bedding and interlocked boundaries of grains is showing mosaic texture, (Fig. 2.49A). Monocrystalline quartz is mostly coarse to medium grain in size and ranges upto 95% of the total quartz assemblage and about 20% of these are showing undulose extinction. Microcrack, solid Zircon inclusion and undulose extinction in quartz grain due to result of strain, found in quartz grains from both igneous and metamorphic sources (Fig. 2.49B). Presence of polycrystalline quartz grains about 2 to 4 % in samples are less stable than the single monocrystalline crystal and are destroyed by sedimentary process and by chemical reactions. Due to tectonic effect, fan shaped recrystallisation and micro folding indicating dragging effect of quartz is conspicuously seen in Fig. 2.49C. Slides of quartzite show coarse grained quartz crystals compared to other slides. One of the slide shows graded bed and another one showing dumbbell shaped mica in Fig. 2.49D.

2.8 STABILITY ANALYSIS OF GRAVITY DAMS BY FEM

The philosophy of concrete dam design is founded on rational and consistent criteria which provide for safe, economical, functional, durable and easily maintained structures (Engineering Monograph No. 19, USBR). Stability of gravity dam is assessed based on stress analysis performed to determine the magnitude and distribution of stresses through the structure under static and dynamic load combinations. In case of gravity dams dynamic loading is mainly due to earthquake. Stability analysis of gravity dams considering earthquake response is a more complex problem compared to other simple and common structures. Due to ageing effect and old technologies employed in the construction of dams during that period, many dams have started showing distresses in the form of cracks, large deformations, seepages, bulging of faces of dams and galleries, loss of mortar in joints of masonry dams and dislodging of concrete from faces etc which needs reevaluation based on current standards. On the other hand many ideal dam sites are over and future dams are needed to be constructed in high seismic zones which means earthquake force needs to be assessed in detail. While carrying out stability analysis of existing and while designing future dams the response of gravity dams to earthquake forces fully accommodated. It is necessary to consider simultaneous action of both horizontal and vertical components of ground motion as the response of gravity dams to earthquake forces depends significantly on the vertical component of ground acceleration (Chakrabarti and Chopra, 1973) also. Further, the hydrodynamic forces may have important contribution in enhancing the stresses in the dam and need to be assessed accurately and be included in the formulation (Chopra, 1968). Thus, the complete system to be analyzed for carrying out stability analysis of gravity dam consists of a concrete gravity dam supported on the horizontal surface of the underlying flexible foundation rock extending to infinite depth and impounding a reservoir of water extending to infinity on the upstream side (I D Gupta, 2007). In that case, a simple conventional analysis is not sufficient to check the stability analysis of dam. Due to rapid developments in fast computing techniques because of high-speed computers, the finite element method finds a lot of application in gravity dam analysis. In the lecture note, an attempt is made to explain the stability requirements of gravity dams as per IS code, methods of analysis and subsequent application of FEM with some case studies carried out in CWPRS.

2.8.2 Stability Analysis

Loads

The following are the forces acting on the concrete dam (EM 1110-2-2200) :

- (1) Dead load
- (2) Headwater and tailwater pressures.
- (3) Uplift.
- (4) Temperature.
- (5) Earth and silt pressures.
- (6) Ice pressure.
- (7) Earthquake forces.
- (8) Wind pressure.
- (9) Subatmospheric pressure.
- (10) Wave pressure.
- (11) Reaction of foundation.

Stability analysis is a process to know the effect of combination of these forces against its stability.

Stability Requirements

A concrete/masonry, gravity dam gets its stability due to its own weight. The basic stability requirements as per IS 6512-1984 code for a gravity dam for all conditions of loading are :

- i. That it be safe against overturning at any horizontal plane within the structure, at the base or at a plane below the base.
- ii. That it be safe against sliding on any horizontal or near horizontal plane within the structure at the base or any rock seam in the foundation.
- iii. That the allowable unit stresses in the concrete or in the foundation material shall not be exceeded.

Before gravity dam overturns bodily, other types of failures may occur, such as cracking of toe material due to tension, increase in uplift, crushing of toe material and sliding. A gravity dam is, therefore, considered safe against overturning if the criteria of no tension on upstream face, the resistance against sliding as well as the quality and strength of concrete of dam and its foundation is satisfied assuming the dam and foundation as a continuous body. The overturning of the dam section takes place when the resultant force at any section cuts the base of the dam downstream of the toe. In that case the resultant moment at the toe becomes clockwise (or -ve). On the other hand, if the resultant cuts the base within the body of the dam, there will be no overturning. For stability requirements, the dam must be safe against overturning. The factor of safety against overturning is defined as the ratio of the righting moment (+ ve MR) to the overturning moments (- ve MR) about the toe

$$\square \text{ Factor of safety against overturning} = \frac{\sum M}{\sum V} \quad (1)$$

The factor of safety against overturning should not be less than 1.5.

The stability of a dam against sliding is evaluated by comparing the minimum total available resistance along the critical path of sliding (that is, along that plane or combination of planes which mobilizes the least resistance to sliding) to the total magnitude of the forces tending to induce sliding. Sliding resistances is a function of the cohesion inherent in the materials and at their contact and the angle of internal friction of the material at the surface of sliding.

The factor of safety against sliding is calculated based on IS:6512-1984 using equation (6.1) as follows:

$$F = \frac{\left[\frac{(W - U) \times \tan \phi}{F_\phi} \right] + \frac{(C \times A)}{F_c}}{P} \quad (2)$$

Where,

F= factor of safety against sliding should be greater than 1.0

W= total mass of dam (Kg),

U= total uplift force (Kg)

P= total horizontal force (Kg),
(Kg/cm²)

C= cohesion of the material at the plane considered

A= Area under consideration for cohesion

F_φ & F_c= Partial factor of safety in respect of friction and cohesion

tan φ = coefficient of internal friction of the material

The factor of safety against sliding shall not be less than 1,

Allowable unit stresses in the concrete or in the foundation material

The strength of concrete/masonry shall exceed the stresses anticipated in the structure by a safe margin. The maximum compressive stresses occur at the heel or toe and on planes normal to the faces of the dam. The strength of concrete and masonry varies with age, the kind of cement and other ingredients and their proportions in the work can be determined only by experiment. The strength of dam material should meet the various requirements

mentioned in the IS code for all loading combinations. In case of permissible tension during earthquake relevant international codes, latest literature based on experiments and expertise in the field may be considered. The acceptance criterion for compressive stresses is that they should be less than the compressive strength of the concrete by a factor of 1.5 for new designs (USACE) and 1.1 for existing dams (FERC)].

Loading Combinations

IS 6512:1984 stipulates seven loading combinations; the details of the same are given below.

- Load Combination A (Construction Condition) - Dam completed but no water in reservoir and no tailwater.
- Load Combination B (Normal Operating Condition) - Full reservoir elevation, normal dry weather tailwater, normal uplift; ice and silt (if applicable).
- Load Combination C (Flood Discharge Condition) - Reservoir at maximum flood pool elevation, all gates open, tailwater at flood elevation, normal uplift, and silt (If applicable).
- Load Combination D - Combination A, with earthquake.
- Load Combination E - Combination B, with earthquake but no ice.
- Load Combination F - Combination C, but with extreme uplift (drains inoperative).
- LoadCombination G - Combination E, but with extreme uplift (drains inoperative).

However, depending on the scope and details of the various project components, site conditions and construction programme different loading combinations are selected with suitable modifications:

Methods of Analysis

Three different methods or approaches for analysis are:

In the conventional analysis, dam section is considered as a vertical cantilever beam. The analysis is carried out by beam analysis by taking moment due to various loads at toe to calculate the point of action of resultant force and eccentricity. The seismic/ earthquake loads are applied in the form of equivalent static loads. The factor of safety against sliding and overturning is calculated based on standard equations. The dam is considered safe if resultant of all forces lies in the middle third portion of the dam base. The Normal stresses are calculated at heel and toe of dam section by following equation:

$$\text{Normal Stresses at HEEL} \quad \sigma_{heel} = \frac{V}{b} \left[1 - \frac{6e}{b} \right] \quad (3a)$$

$$\text{Normal Stresses at TOE} \quad \sigma_{toe} = \frac{V}{b} \left[1 + \frac{6e}{b} \right] \quad (3b)$$

Where V = Total vertical Force

B = Base width of dam

e = eccentricity of the resultant force from the centre of the base

Several methods such as beam analysis and finite element analysis are available for carrying out the dynamic analysis. However, such methods are sophisticated and requires dedicated softwares and highly skilled man power. Very often, in spite of sufficient invesigations, site condition at dam site change, which is revealed during excavation; in such cases, dam section changes. Dam sections are revised many times for other reasons also. Apart from this stability analysis of existing dams is required to be carried out for one or the other reasons. In all such cases going for detailed dynamic analysis may not be possible due to time, money or other reasons. To overcome these difficulties many simplified methods are evolved to carryout seismic analysis of dams which require less time and includes all possible effects of earthquake responses with certain assumptions. These methods are tested and checked by doing the analysis with both the dynamic analysis as well as simplified methods. These simplified methods (IS: 1893-1984; USBR, 1977; US Army Corps of Engineers, 1995; etc) also find useful application for preliminary stage design of dams.

2.8.3 FEM Methods

The concept of the Finite Element Method consists of subdividing a given structure or domain in to a discrete number of small region of finite dimensions. Each small region is characterized as finite element, whose analysis is easily carried out by standard principals. The original structure or domain is then reassembled from such elements to study the total behavior of the structure. The strain field inside an element should nearly be constant. The system is called as an assembly of discrete finite elements. The F.E.M. formulation consists of following main steps:

- i) Discretization of the structure
- ii) Selection of displacement stiffness using variational principal
- iii) Derivation of element stiffness using variational principal
- iv) Assembly of the algebraic equations for the overall discretized structure

- v) Solution of the unknown displacement
- vi) Computation of the element strains and stresses from nodal displacements.

To carry out the analysis as per above procedure many general purpose Finite Element Softwares have been viz. ANSYS, LUSAS, SOLVIA, ADINA, NASTRAN, IDEA, SAP, ABAQUS, HYPERWORKS and SOLIDWORKS developed and are in use all over the world.

2.8.4 CWPRS EXPERIENCES

2.8.4.1 2D-Stability of the Spillway Section, Karanja Dam, Karnataka.

The Karanja Dam (Fig. 2.50), constructed in 1987 across river Manjra in Godavari river basin near Byalhalli in Bhalki Taluka, Bidar in Karnataka state, is a composite dam consisting of a non overflow earthen portion and 6 spans of overflow ogee type concrete spillway. As per the directives from Dam Safety Review Panel, to recheck the stability of overflow section, the Water Resources Department, Government of Karnataka requested CWPRS, Pune to take up same. Accordingly, two dimensional finite element stress analysis has been carried out for static and earthquake loads as per the procedure specified under IS 6512-1984, IS 1893-1984 and IS 1893 (Part 1)- 2002. Loading combinations

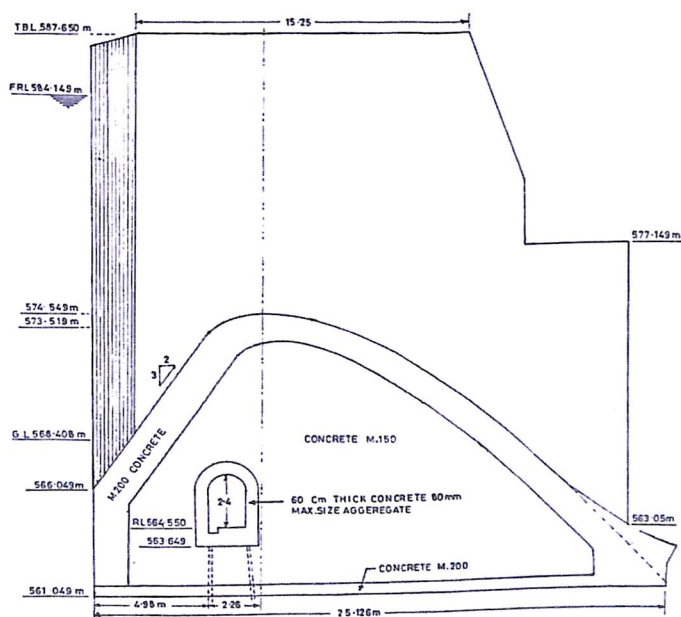


Fig. 2.50 Karanja Dam Section

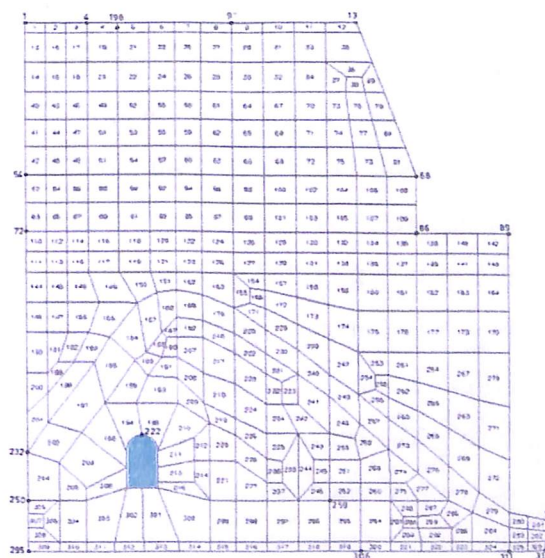


Fig. 2.51 Karanja Dam FEM mesh

A to G as outlined in IS 6512-1984 have been investigated for stability considerations of the deepest overflow section of the dam (height=20.7 m). The dam-foundation system incorporating the deepest overflow section of the dam (height=20.7 m) has been idealised by a two dimensional finite element model having 456 elements(Fig. 2.51) with input parameters as listed in table 2.7.

Table – 2.7 Input Parameters for Karanja dam Analysis

S.N.	Property	Value
1.	Young's Modulus of Elasticity 'E'	
	i) Foundation rock	$2.50 \times 10^5 \text{ Kg/cm}^2$
	ii) Dam material (Concrete)	$2.20 \times 10^5 \text{ Kg/cm}^2$
2.	Poisson's Ratio 'v'	
	i) Foundation rock	0.15
	ii) Dam material (Concrete)	0.20
3.	Mass Density ' γ ' of Dam material	$2.4 \times 10^{-3} \text{ Kg/cm}^3$
4.	Mass Density of water ' γ_w '	$1.0 \times 10^{-3} \text{ Kg/cm}^3$
4.	Horizontal Seismic coefficient α_h	0.108
5.	Vertical Seismic coefficient α_v	0.054

Principal stresses, shear stresses, horizontal and vertical stresses along various planes have been evaluated. For static load combinations A, B, C & F tensile as well as compressive stresses developed are found to be within permissible limits of concrete strength. For earthquake loading combinations D, E and G, ($\alpha_h=0.108$, $\alpha_v=0.054$) small area near heel of the dam has been found to be under high tensile stresses. The maximum tensile stress of the order of 11.976 kg/cm^2 (Fig. 2.52a) is occurring for the load case G. For this case, the high tension zone covers only small area (less than 0.5%) of the dam section. Maximum compressive stress of the order of 43.628 kg/cm^2 (Fig. 2.52b) is seen to be developed in the area in the heel region for load combination D. It is observed that tension and compression developed under static as well as earthquake loads remain within allowable limits of concrete strength.

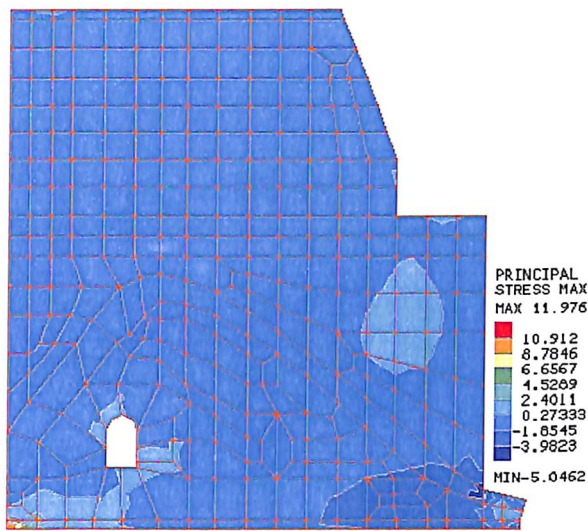


Fig. 2.52a: Max.Principal Stresses for Load Combination 'G'

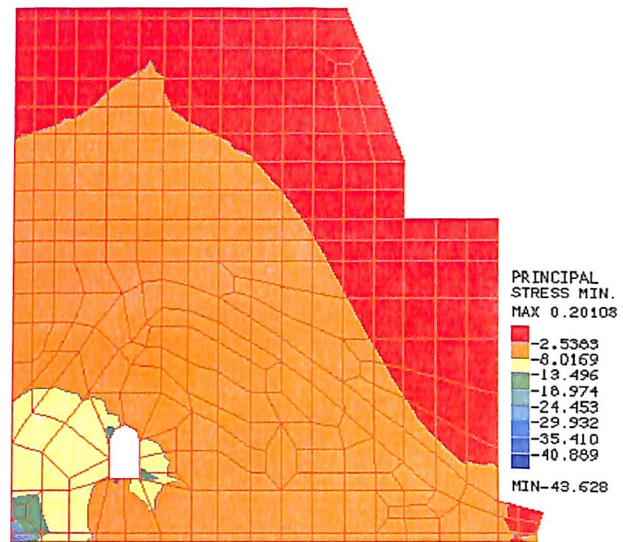


Fig. 2.52b: Min.Principal Stresses for Load Combination 'D'

Factor of safety against shear / sliding has been evaluated using the formulation as outlined in IS 6512-1984 for all load combinations considering cohesion $C = 5 \text{ kg/cm}^2$ and angle of internal friction ($\phi = 45^\circ$). The average factor of safety at the dam-foundation interface has been found to be more than unity and is well within the limits as recommended in IS 6512-1984 (Table 2.8). Displacements at selected locations in the dam-foundation domain have been evaluated (Table 2.9). Maximum horizontal displacement of the order of 1.882 mm is observed at the top upstream point under loading combination G and maximum vertical downward displacement of 1.595 mm is observed at the same point under loading combination D.

Table – 2.8 Max. Stress and Deflections

Parameter	Stresses		Deflections	
Load Combination	$\sigma_{\max}$, Major (Kg/cm ²)	$\sigma_{\min}$, Minor (Kg/cm ²)	δ (Horizontal) (mm)	δ (Vertical) (mm)
A	1.236	- 35.630	- 0.588	- 1.463
B	1.848	- 25.344	0.984	- 1.092
C	5.424	- 27.941	1.449	- 1.140
D	1.568	- 43.628	- 1.202	- 1.595
E	10.505	- 32.975	1.860	- 1.277
F	5.944	- 27.644	1.462	- 1.113
G	11.976	- 32.544	1.882	- 1.236

Table – 2.9 Factor of safety

Sr. No.	Load Combination	F _φ	F _c	Minimum Allowable F	Calculated Avg. F
		As per IS Code 6512:1984			
1	A	1.5	3.6	1.0	9.28
2	B	1.5	3.6	1.0	4.71
3	C	1.5	3.6	1.0	4.18
4	D	1.2	2.4	1.0	15.46
5	E	1.2	2.4	1.0	3.72
6	F	1.0	1.2	1.0	7.91
7	G	1.0	1.2	1.0	5.42

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CHAPTER III

REHABILITATION MATERIALS AND METHODS FOR GRAVITY DAMS

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3.0 INTRODUCTION

A basic concept of dam rehabilitation is to remove weak zone and repair it with proper material and methodology. This will reintroduce a protective and durable environment for dam and will prevent further action of deterioration.

After knowing the location and extent of distress and causes of distress by employing assessment methods, quantification of rehabilitation work is essential. Similarly selection of proper repair material is also prime important. Proper diagnosis of cause of damages, selection of suitable material adopting proper methodology and ascertaining efficacy of the treatment are the key points for successful restoration or strengthening of the damaged gravity dams.

REPAIR MATERIAL

Almost every repair/restoration/strengthening job has unique conditions and special requirements. Selection of a repair material for a particular job is based on the knowledge of physical and chemical properties, the nature of environment, availability and cost and ease of application. Generally, the essential requirements of concrete repair materials are effective adhesion with parent material, resistance to adverse effects and agents of deterioration, rapid development of strength and high degree of impermeability. However, it is always advisable to check the performance of the proposed repair material by conducting pre-application tests in laboratory. For their purpose, many times tests may have to be devised keeping the site application in view. Based on the type of application and composition, the repair materials are classified as;

3.1 Grout materials

Various types of grouts to suit specific application are available. They can be cement sand grouts, fiber reinforced grouts, expanding grouts, epoxy grouts etc. They provide flowable consistency which can be pumped into cracks or remote areas. Grout should remain in

position without cracking and should have negligible shrinkage. Cementitious grouts or cement sand grouts sometimes contain graded aggregates of less than 5 mm size and admixtures to enhance certain properties. Expanding admixtures like aluminum powder are sometimes added to compensate for shrinkage. Fiber reinforced grouts because of their higher tensile strength; abrasion resistance and toughness are becoming popular as a repair material. The percentage of fibers (glass, steel, polypropylene etc) is usually 1 to 3% by volume. Epoxy grouts because of their low viscosity, negligible shrinkage and excellent bond with almost all construction materials are widely in use.

3.1.2 Epoxy Compounds

Epoxyes are synthetic thermosetting resins obtained by poly condensation of epichlorohydrin and bisphenol. A combination of resin and hardener is referred to as epoxy system. Once the resin and hardener are mixed, polymerization reaction starts. Epoxy systems of low viscosity are useful for grouting of cracks whereas fillers like cement, silica flour are added to get epoxy putty. Graded sand such as quartz sand is used as filler for making epoxy mortar and graded aggregates are used for making epoxy concrete. Filled and unfilled epoxy systems are usually used in the restoration of the distressed hydraulic structures.

Epoxy resins have proved to be useful in restoring the damaged hydraulic structures because of properties like rapid curing at ambient temperature from liquid state to hard solids of high strength, strong adhesion with normal structural material like stone, concrete, steel, Negligible shrinkage, High impermeability, High abrasion resistance, High penetrability, Resistance to most of the acids, alkalis and other chemicals etc. Filled and unfilled epoxy systems are used in civil engineering structures for the applications such as Grouting of cracks in concrete/masonry structures to restore their strength, Sealing/plugging/ of leaks, Repairing and coating the damaged surfaces of spillways/sloping aprons/stilling basins due to abrasion/erosion

3.1.3 Polymer based materials

The trend of using polymer based materials has rapidly progressed in recent times. The incorporation of polymers in cement and concrete improves various properties such as strength, adhesion, resilience, water tightness, chemical resistance, durability etc. The types of polymers generally used in modifying mortars and concretes are either thermoplastic or thermosetting resins which can be classified into liquid resins, latexes and water soluble polymers and copolymers. Selection of a particular type of polymer depends upon the intended use. The way in which nature of polymer system modifies a particular property

depends on the type of monomer or resins that constitute the polymer and also on other ingredients added into the system such as surfactants, stabilizing colloids, antifoaming agents etc. The latex admixtures used are styrene butadiene rubber (SBA), polyvinyl acetate (PVA), acrylics or epoxy emulsions. Usually the addition of latex and epoxy to cement mortar makes it more workable and thus facilitates preparation of mortar of low w/c ratio. However, curing is essential to prevent cracking.

3.1.4 Bonding agents

These are natural, compounded or synthetic materials often used in repair application such as bonding old to fresh concrete, sprayed concrete/mortar to attain reliable bond. Latex emulsion and epoxies are the two main types of bonding agents in use. However, bonding action of epoxy is considered ideal for repair works of large magnitude because of its excellent bonding characteristics with substrate concrete.

3.1.5 Free flow micro concrete

Free flow micro concrete is gaining popularity as a repair material because of its multivariate advantages. It is a versatile material for repairs of damaged reinforced concrete especially in areas of low accessibility and where compaction of concrete by vibration is difficult. Micro concrete usually consists of Portland cement, graded aggregates of less than 10 mm size, fillers and additives to impart strength and non shrink characteristics. It is usually supplied in ready to use form, addition of water is only required at site. Usually its 28 days compressive strength is more than 50 MPa.

3.1.6 Modified concrete/mortar

It involves either modifying the constituent composition (superplasticizers, polymers etc.) or method of concrete. Superplasticizers are used to achieve flowability at low w/c ratio. They are particularly useful in repairing inaccessible areas or highly reinforced areas where vibration is extremely difficult. Superplasticizers help to produce a dense and impermeable concrete with reduced shrinkage and cracking potential than normal concrete giving longer durability to structure. Polymer concrete/mortar because of their high early strength and durability characteristics are being used increasingly in rehabilitation jobs. There are three types viz. Polymer concrete (PC), Polymer modified concrete (PMC) and Polymer impregnated concrete (PIC). PC is formed by polymerizing a monomer binder and aggregates. In PMC monomer/ polymer is added to freshly mixed concrete and cured. In PIC, monomer is impregnated into concrete and subsequently polymerized. Generally polymer concrete has high bond strength and chemical resistance. Being cementitious in nature, they

ensure homogeneity in repaired structure. Polymer lattices like SBR latex modified concrete are used in variety of repair application. These materials are also alkaline and therefore restore the alkalinity of distressed concrete. Epoxy mortars are mainly used where high early strength is required.

In certain situations, repairs may have to be done in adverse conditions where special materials and techniques are necessary e.g. Prepacked Aggregate Concrete (PAC) technique is found to be useful for underwater concreting, repairs of piers and spillways etc. In such cases graded coarse aggregates are placed in forms and then filled with pumpable grout. PAC has got very good bond with existing concrete.

3.1.7 Patching materials

These are either polymer modified or plain cement concrete/mortar or pure polymer mortars. They are usually formulated to cope up with demands of site conditions coupled with quick repair schedules. Usually use of admixtures and superplasticizers are made to reduce shrinkage and water content. Epoxy concrete/mortar has also been widely used as a patching material because of high strength and good adhesion with substrate concrete.

3.1.8 Resurfacing materials

Resurfacing materials are basically used to improve wear resistance, chemical resistance and appearance. They produce a finished wearing surface that contains more aggregates and lower paste. Usually cement concrete overlays belonging to superplasticized low slump dense concrete, latex modified concrete, fiber reinforced concrete/mortar, silica fume concrete etc. are used as resurfacing materials for repair and rehabilitation purpose.

3.1.9 Protective coatings

The protective coatings are available for protection of reinforcing members as well as exposed concrete.

Polymer/cementitious coatings are developed for reinforcement protection. The polymer coatings rely on development of interpenetrating polymer network on the surface which is free from micro defects. Cementitious polymeric systems on hardening form network. These protective coatings develop good bond with concrete. The additional layers on reinforcement also control the attack of deleterious ions on steel.

The coatings about 100 to 300 microns thick used over concrete structure are basically used as a skin permeability reduction measure. Acrylic systems are usually used since they have good chemical resistance, color retention characteristics and inter coat adhesion. Polyurethane coating is also preferred in various applications because of their UV light

resistance, but they require prepared surface for application. Pore lining or blocking materials like silanes/siloxanes are used in conjunction with acrylic systems. Epoxy coatings serve as a protective material in extremely aggressive environments where high chemical resistance is required.

3.2 REMEDIAL MEASURES

In case of distressed concrete dam structure, following measures are usually adopted in combination to make them serviceable like grouting of cracks, sealing of joints, repairs with polymer base materials/ epoxy compounds. Following are the various remedial measures adopted for repairs and strengthening of the concrete/masonry dams.

Grouting/Injection -

The process of filling the gap with grout such as cement, epoxy etc. under pressure. Grouting is an effective method. Usually cement grouts with or without addition of admixtures/ fillers are being used in most of the cases.

Polymer impregnation-

Impregnation of monomer into the pores of hardened concrete and then getting it polymerized by irradiation or thermal catalytic process, results in development of High Strength concrete.

Pointing:

The raking and pointing of the masonry on the damaged upstream face.

Gunitting/Shotcreting-

Usually pointing of the upstream face of masonry or damaged portion of masonry is a cumbersome and time taking process therefore concrete conveyed through a hose and pneumatically projected at a high velocity on to a upstream surface of masonry.

Cable Anchoring:

Addition of vertical force by providing anchors to the upstream face of the dam to reduces overturning moment and imparts greater frictional resistance against sliding.

Concrete membrane:

The upstream face is covered using a concrete membrane. Sometimes concrete membrane is provided up to minimum draw down level (MDDL) and gunitting above MDDL.

Use of Geo-membrane:

Upstream face of the dam is covered with geo-membrane by proper clamping to stop the entry of water through the joints/pointing of upstream face.

Steel Jacketing: Encasing the upstream face or the structure using steel structural membrane like plates.

Pre-stressing –

Considered as an emergency measure of strengthening of dams. The treatment is to be followed by some alternative permanent measures.

Earth Backing –

This is a simple way to strengthen the dam by providing earth backing on downstream face of dam. However possibility of separation between earth and masonry particularly near the top can not be ignored in such type of backing. In case of low water level, earth backing induces tension at the toe.

Concrete / Masonry Backing

A viable alternative of permanent remedial measures is by providing buttresses or full masonry or concrete backing. The performances of dams backed using buttresses or full concrete backing has been reported to be satisfactory.

3.3 POST REPAIR EVALUATION

The performance and competence of the finally executed remedial and rehabilitation measures could be evaluated by conducting following studies.

1. Composite cores including the interface between the old and new material provide direct evidence regarding the effectiveness of the treatment.
2. Permeability and tracer studies through suitably located holes can further verify the extent of impermeability achieved
3. Nondestructive tests (NDT) can be used to survey the general quality and uniformity of the repair work
4. Periodic inspection, surveys and monitoring of installed instrumentation responses will reflect the effectiveness of restoration work.

3.4 LABORATORY STUDIES

Following tests have been conducted on the repair/coating/primer systems mentioned above. The tests have been conducted as per relevant standards (BIS/ACI/ASTM) or the methodology developed by CWPRS as per past experience. The tests procedures are explained in short as below;

3.4.1 Tests on Mortar

The studies include testing for determination of workability, density, compressive strength, tensile strength, abrasion resistance and modulus of elasticity etc.

3.4.1.1 Flowability / Workability

Test for flowability/workability has been carried out as mentioned in IS 2250. In this test, mould was placed at the centre of flow table, afterward a layer of mortar in about 25 mm in thickness was filled in it and the layer tamped 20 times for uniform filling. Afterward the remaining portion of the mould filled with mortar and tamped again. After cutting off the excess mortar on the top of the mould and cleaning the edge of the mould and flow table, mould was lifted from the flow table and immediately dropped the table through a height of 12.5 mm, 25 times in 15 seconds.

The flow is the resulting increase in average base diameter of the mortar mass, measured on four diameters expressed as a percentage of the original base diameter. The percentage increase was expressed in terms of flowability.

3.4.1.2 Density & Compressive Strength

For qualitative assessment of mortar, compressive strength has been determined by casting mortar cube specimens of size 50 mm as per the proportions and manner recommended. The mass and volume of each cured specimen is recorded for determining density

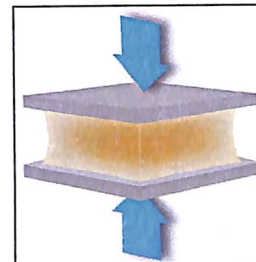


Fig. 1- Compressive Strength

then cube specimen is subjected to loading in a uniaxial compression testing machine (Fig. 1). The compressive strength of the specimen is then calculated by dividing the maximum applied load at failure of the specimen during the test by the original cross-sectional area of the cube specimen.

3.4.1.3 Briquette Tensile Strength: (IS 4456, 9162)

For qualitative assessment of mortar, direct tensile strength test has been determined by casting dumb bell shape briquette specimens using repair mortar.

After curing period, the briquette specimens have

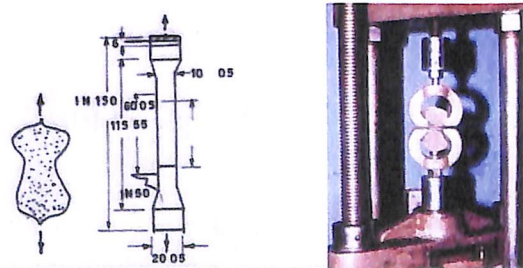


Fig. 2- Briquette, Dumbbell Specimens and test in progress

been tested in a Universal testing machine. The tensile strength of the briquette specimen is then calculated by dividing the maximum load applied at failure by the cross-sectional area of the briquette specimen. The test is shown in figure 2.

3.4.1.4 Determination of Young's Modulus of Elasticity

The young's modulus of elasticity mortar material was determined by subjecting cured cylindrical specimen of size 50 mm dia. x 100 mm height to axial compression. Load is applied and simultaneously loads and strain is recorded to determine modulus of elasticity. The stress and the corresponding strain were measured at regular load intervals. The variation of stress and strain is plotted to determine modulus of elasticity within elastic limit. The test set up is shown in figure 3.



Fig. 3– Set up for modulus of elasticity

3.4.1.5 Abrasion Resistance

The test indicates the relative degree of resistance to abrasion of the repaired/screed surface using repair mortar with respect to hardened concrete surface. This test is performed using Dorry's abrasion testing machine. The initial weight of the cured specimen is taken accurately and the specimen is then subjected to abrasion using abrasive charge in the form of standard sand in Dorry's abrasion testing machine (Fig. 4). Abrasion resistance was then determined by comparing loss in weight of the sample per unit area after fix revolutions with the loss in weight of identical concrete specimens tested under similar test conditions.



Fig. 4 – Dorry's Abrasion machine

3.4.2 Tests on Bonding agents/coating agent

Following tests are conducted on the bonding agent to be used as primer for the treatment to the prepared concrete surface.

3.4.2.1 Mix Viscosity & Pot Life

The viscosity of system indicates its effectiveness in covering surface suitably. The bonding agent consists of two component system. These two components were mixed gently for about five minutes in the



Fig. 5 - Mix viscosity test

proportion as recommended and obtain a uniform mix. The viscosity was then determined using Brookfield Viscometer or Viscosity cups (Fig. 5).

Variations in viscosity of the mix with time were observed. The viscosity of the mix increases with time and ultimately it hardens.

3.4.2.2 Bond strength

For assessing the suitability of adherence of the repair material to substrate /hardened concrete with primer/bonding system has been evaluated using in-house developed methodology. A cube specimen of size 70.6 mm with repair material is cast. At the same time, for bond strength evaluation, one of the faces of the hardened concrete cube of same size i.e. 70.6 mm has been cleaned and roughened for primer application of bonding agent. A primer coat/bonding agent have been applied on one of the cleaned and roughened bonding surface of the concrete cube and on one of the faces of the freshly cast repair material cube. The two cubes were then pressed firmly against their primed surfaces. The assembly was then held tightly in a clamp for effective bonding.

After a curing period, the end faces of bonding surface of assembly have been capped in steel caps. The specimen thus so obtained was tested under tension in a universal testing machine (UTM).

The load at failure of specimen was

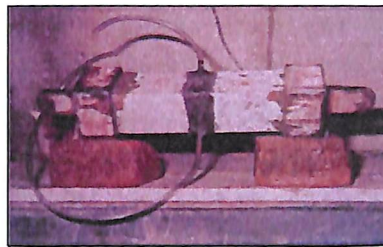


Fig. 6 – Test Specimen and test set up

recorded. The failure pattern of the specimen and the zone of failure were also noted carefully (Fig. 6). The load at failure has been divided by contact area to obtain bond strength. The stress at failure was noted, which is the bond strength of primer system to bond repair material with concrete. The pattern of failure of specimen i.e. at bond, in repair material or in concrete was noted carefully.

3.4.2.3 Pullout/adhesion strength of Primer system with concrete.

For assessing the suitability of adherence of the primer system to hardened concrete has been evaluated using adhesion/pull out test. For adhesion /pull out test, a coat of primer is applied on prepared hardened concrete surface. After a curing period, dolly is glued on the

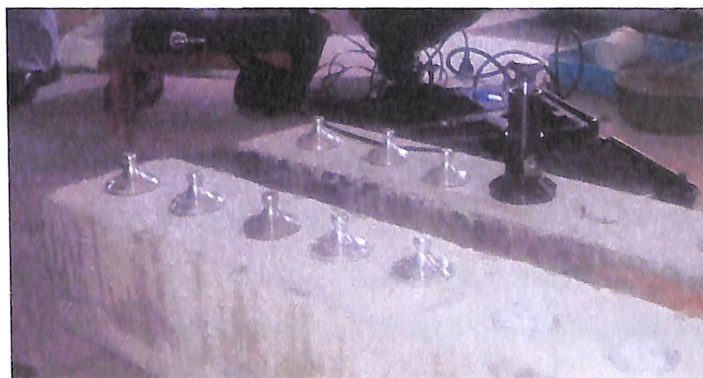


Fig.7: Pullout/adhesion strength of Primer system

coating surfaces of the concrete. After sufficient curing of glued material, coating along the external face/side of the dolly's were cut to get the exact dimension of the contact area. The specimen thus so obtained is then tested with the help of Pull out test equipment (Fig. 7). The failure pattern of the specimen and the zone of failure are also noted carefully. The load at failure has been divided by dolly's contact area to obtain adhesion/pull out strength. The stress at which dolly's fail in bonding with substrate concrete is noted which gives the adhesion strength of primer system with concrete.

3.4.2.4 Direct Shear test

The repair material when applied on the surface may be subjected to different forces due to movement of concrete due to different behavior of top repair material and substrate hardened concrete. It is therefore essential to determine direct shear test. For this test, a methodology developed at CWPRS for determining puncture/direct shear test of repair material along with primer system. The test procedure adopted is described as follows.

A rectangular concrete specimen of size 30 cm x 30 cm x 10 cm was cast using a suitable concrete mix (M 20 grade). During casting only a groove of size 10 cm x 10 cm x 10 cm was provided at the centre by means of a wooden template. The specimen was then water cured for 28 days. After application of primer from the top 5 cm on four side walls of groove (leaving bottom space of 10 cm x 10 cm x 5 cm empty) primed groove and top 5 cm over it then filling by repair material, thus making repair material Prism specimen of size 10 cm x 10 cm x 10 cm as shown in figure 8. After curing the repair material and

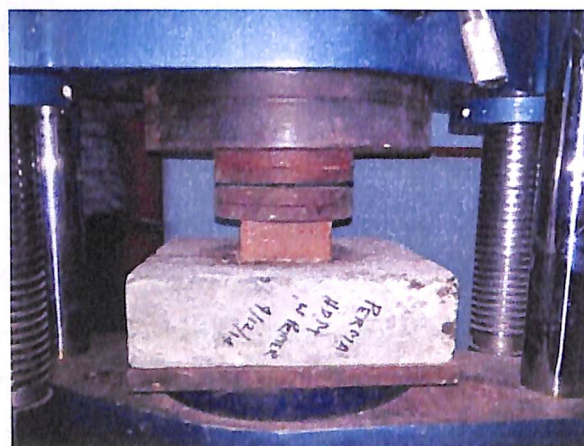


Fig.8: Direct shear test setup

primer system, this composite cast specimen subjected to compression mode in compression testing machine. The load was gradually increased till the central repair material prism shear with reference to the adjoining concrete. The load at which the repair material gets shear or crack/ breaks the concrete is noted. The direct shear strength of the system is then calculated by dividing the maximum applied load at which repair material shear or crack the adjoining concrete during the test by the total contact area of the prism specimen with concrete (i.e. 4x10 cm x 5 cm) as "direct shear strength" of the repair material with primer system.

3.4.2.5 Bond Strength in shear – Slant Cone test (ASTM C 882-99)

This test method covers the determination of the bond strength of primer system for use with hardened concrete with repair material under combined effect of compression and shear. It covers bonding hardened concrete with repair material by primer system. Two equal sections, of which one is pre-cast and pre cured ordinary concrete specimen and other one is freshly cast with repair material bonded at an angle of 30° , applying bonding agent /primer at bonding face of pre-cast specimen of Portland cement concrete to form 75 X 150 mm composite cylinder. The cured bonded specimens tested under compression mode in UTM machine (Fig. 9).



Fig. 9: Slant cone Shear Test

3.4.2.6 Flexural Strength of Composite Beam specimen (IS : 516 – 1959)

For this test, beam specimens of M-20 concrete of size 15 cm x 15 cm x 70 cm and 12.25 cm x 15 cm x 70 cm were cast and water cured. After curing period of 28 days, beam specimen of size 12.25 cm x 15cm x 70 cm dried and prepared beam surface area of 15 cm x 70 cm primed with Primer system and then placed in the beam mould size 15 cm x 15 cm x 70 cm and filled with repair material to form composite beam of size 15 cm x 15 cm x 70 cm. Composite beam was allowed to cure and then placed in the universal testing machine for flexural test such that bedding/screed mortar lies on tensile zone. Similarly concrete beams were also tested for comparison/ improvement. The load has been applied until the specimen fails, and the maximum load applied to the specimen during the test was recorded (Fig. 10). The appearance of the fractured faces of concrete and any unusual features in the type of failure is also noted. The flexural strength (modulus of rupture) of the specimen has been then calculated. This test method covers the determination of the bond strength of Primer system

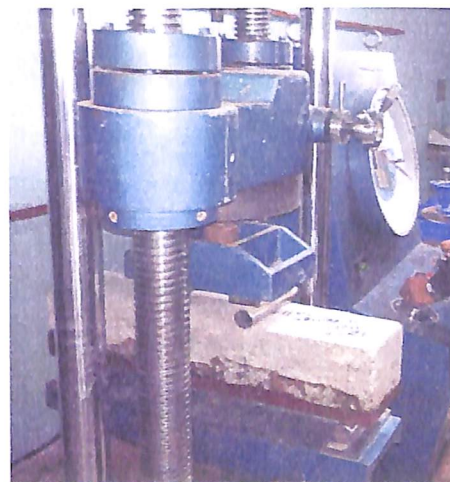


Fig.10: Flexural Strength of Composite Beam specimen test

for use with hardened concrete with repair material under combined effect of compression and tension.

3.4.2.7 Split tensile Strength of composite cylindrical specimen

For this test, half split cylindrical specimens of M-20 concrete with the help of cylindrical mould of size 15 cm diameter x 30 cm height were cast and water cured. After curing period of 28 days, plane surface of half split cylindrical specimen prepared for primer application by cleaning with wire brush followed by water jet cleaning and allowed to dry. After application of primer system on plane prepared surface the specimen placed in cylindrical mould of size 15 cm diameter x 30 cm height and remaining portion was filled with repair material to form composite cylinder of size 15 cm diameter x 30 cm height. The cured composite specimen then placed in the two platen of the compression testing machine with packing strip

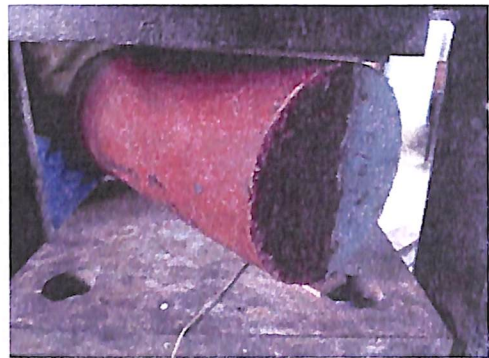


Fig.11: Split tensile test specimens for loading

carefully positioning along the top and bottom of the plane of loading of the specimen. It was ensured that the upper platen is parallel with the lower platen (Fig. 11). Similarly concrete cylinders were also tested for comparison/ improvement. The load was then applied without shock and increased continuously at a nominal rate within the range 14 to 21 kg/sq.cm./min. The maximum load applied was then recorded. The appearance of composite cylindrical specimen and any unusual features in the type of failure is also noted. This test method covers the determination of the split tensile strength of primer system for use with hardened concrete with repair material.

3.4.2.8 Strength Improvement

For observing improvement in the compressive strength of distressed concrete, cylindrical specimens of size 15 cm diameter x 30 cm height were cast of with M-20 concrete. After 24 hours the concrete specimen removed from the mould, and chipped off its circular curved surface up to 15-25 mm. After water curing for 28 days, curved surface of specimen prepared for primer application by cleaning with wire brush followed by water jet cleaning and allowed to dry. After application of primer system on curve surface, the specimen placed in cylindrical mould of size 15 cm diameter x 30 cm height at center and annular portion was

filled with repair material to form composite cylinder of size 15 cm diameter x 30 cm height. After curing composite cylindrical specimen then placed in the universal testing machine for compression loading. Similarly concrete cylinders were also tested for comparison/improvement. The appearance of composite cylindrical specimen and any unusual features in the type of failure was also noted. This test method covers the determination of the improvement of compressive strength for distressed concrete when repaired with Primer system and repair material.

3.5 CWPRS EXPERIENCES

3.5.1 Rehabilitation of Koyna dam, Maharashtra

103 m high and 807 m long rubble concrete Koyna Dam was constructed across river Koyna in Satara District of Maharashtra. Due to earthquake on 11/12/1967, some of the blocks of non overflow section showed cracks on both u/s and d/s faces and also in drainage gallery. For restoring the monolithic action at the cracked section, temporary measures like grouting of cracks by epoxy/polyester resin, sealing of the cracks at upstream face, strengthening of seven high monoliths by pre-stressed cables, drilling of drainage holes to relieve the locked up water were taken. Pre-application tests were carried out in CWPRS on number of epoxy grout systems for grouting cracks which are dry/moist/wet condition and suitable epoxy grout systems suggested for crack repair.

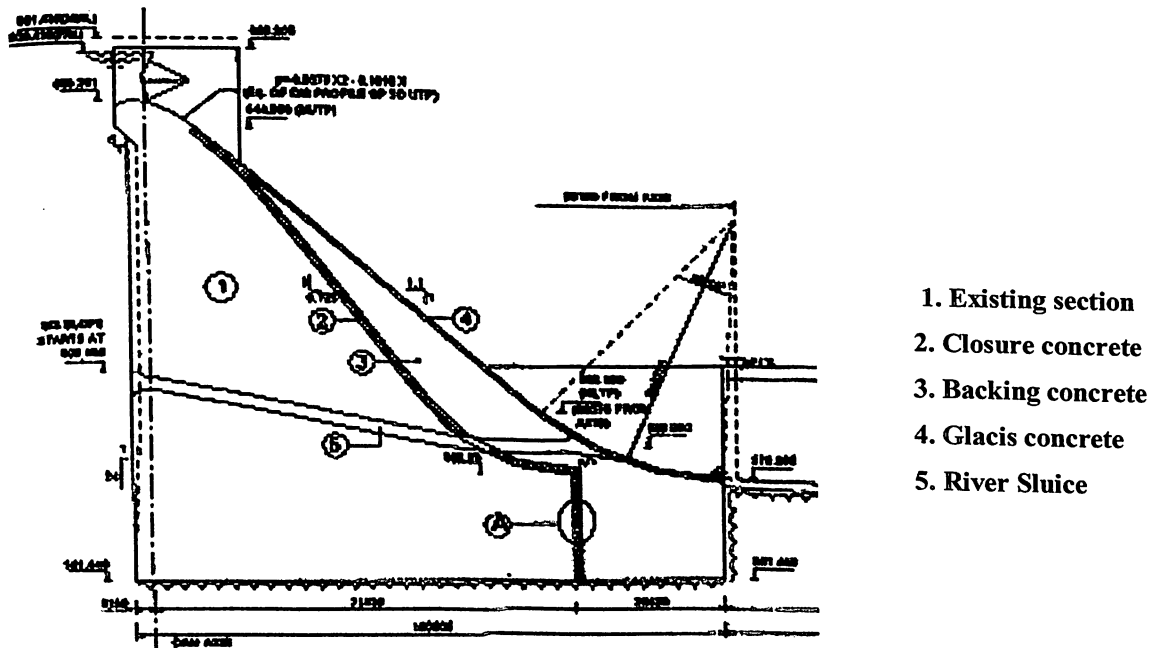


Fig. 12 Koyna dam spillway – Scheme of strengthening

As a permanent strengthening measure full concrete backing upto a certain elevation and buttress backing above that level was adopted for non over flow section. However, Killari earthquake of 6.3 magnitude on Richter's scale, in 1993 indicated the necessity of strengthening of overflow section from the foundation level to upper tangent point of ogee.

To overcome the interface problem for concrete backing of over flow section it was decided to provide the concrete backing in two stages i.e. first to cast strengthening concrete leaving a gap of 1.22 m from downstream face of old dam, allow it to cool and shrink and then place closure concrete in the gap to join old and strengthening concrete (Fig. 12) and planned to complete the work in two seasons. Both strengthening and closure concrete were placed in the lifts of height 1.5 m at an interval of 72 hrs.

To determine the strength and thermal properties of both strengthening and closure concrete, to suggest suitable placement temperature and cooling arrangement, studies were carried out in CWPRS. (CWPRS TR No. 4204, 2005).

3.5.2 Underwater treatment of cracks in Hirakud dam, Orissa

59 m high Hirakud dam (Fig. 13) is built in the year 1957-58 across river Mahanadi in Orissa. It is a composite structure comprising of a earth dam, a concrete dam and a masonry dam. The total length of dam is 4.8 km flanked by earthen dykes of 21 km on left and right sides. The installed hydropower generating capacity of the project is 307.5 MW.

The main dam has two concrete spillways having 32 blocks. Left spillway is of 402 m length consisting of 20 blocks and the right spillway is of 256 m length consisting of 12 blocks. In addition there are 64 low level sluices and 34 radial crest gates.

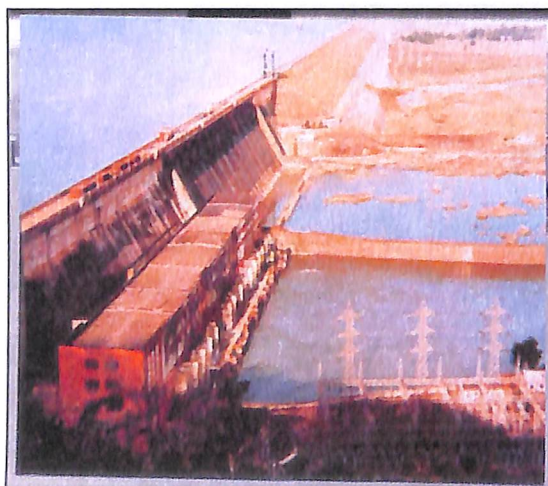


Fig. 13 – Hirakud Dam

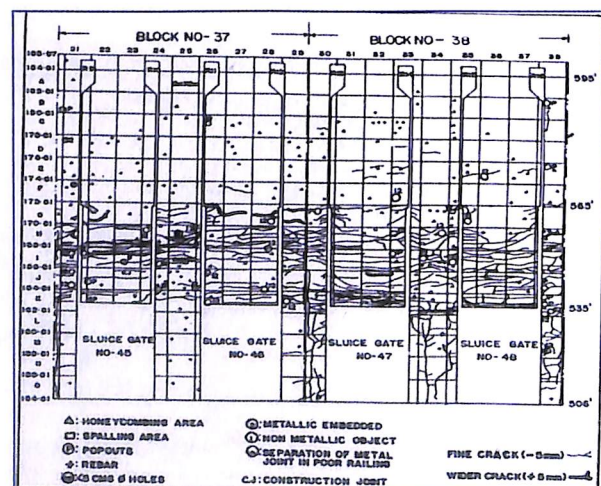


Fig 14- Cracking Pattern

During the course of time, horizontal cracks of various sizes were observed on the upstream face, sluice barrels, operation gallery and around gate shafts and vertical cracks near the right end blocks of spillways. The cumulative length of these cracks was found to be more than 22 km (Fig. 14). These cracks were classified into 2 groups – fine cracks which were less than 5 mm in width and wide cracks between 5 to 12 mm in width. 95% of the cracks were found to be fine. The investigations revealed that these cracks were developed due to thermal stresses caused by rapid concrete placement thus generating high heat inside concrete mass and subsequent seepage through these cracks causing adequate ingress of moisture influencing alkali silica reaction.

For certain reasons depleting the water level of the reservoir was infeasible and uneconomical. Reservoir level had always to be operated between RL 630 ft to RL 595 ft and never below RL 595 ft. Therefore underwater repairs of cracks was resorted to for cracks between RL 595 ft. and RL 506 ft. The cracks above RL 595 ft upto RL 610 ft were repaired under dry condition.

It was proposed to grout the cracks with epoxy system suitable for underwater treatment. A number of epoxy grout system and sealing system were obtained from foreign market and also from indigenous market. Necessary tests were conducted for grout and sealing systems by casting the specimen underwater. The injection system having low viscosity of the order of 150 cps and bond strength of more than 24 kg/sq.cm and 16 kg/sq.cm in direct tension and shear mode respectively was adopted for use. The sealing system selected also indicated pressure bearing capacity of more than 4 kg/sq.cm.

After grouting of the cracks, cores of 65/40 mm dia. and length 5 times dia. (Fig. 15) were taken from treated crack surface for visual inspection, observation of penetration of epoxy and to check the efficacy of grouting. The cores so obtained were tested under compression, indicated failure in concrete and away from treated crack plane (CWPRS TR No. 2875, 1991).



Fig. 15 – Cores after grouting

3.5.3 Rehabilitation of slotted bucket, Jawaharsagar dam, Rajasthan.

Jawaharsagar dam, a concrete gravity dam of 36 m height and 393m in length built across river Chambal, was constructed in 1971. The dam is provided with Ogee shaped spillway with 12 radial gates. Slotted roller bucket of radius 13.24 m and consisting of 83 teeth of 1.9 m x 3.3 m size are provided at the toe of dam with sloping apron for energy dissipation.

Out of 1763 sq.m. of the apron surface area, 1155 sq.m. area was damaged due to flow induced cavitation, erosion and also impact action caused by unstable river bed material from the vicinity of the bucket. Cavities ranging from few cms to about 50 cm deep were observed on the sloping surface of apron exposing reinforcement at few places. The damages were also seen on the curved surface and end face of many teeth. The edges and corners of some of the teeth were broken indicating impact action.

Suitability tests were carried out in laboratory on various epoxy concrete and epoxy mortar mixes using different epoxy systems from different manufacturers for repair of damaged teeth and apron surface.



Fig. 16 – Areas repaired in 1994

Treatment Procedure

- The damaged surface of apron was made dry and the loose material was removed by chipping and using compressed air.
- A primer coat of plane epoxy was applied on cleaned surface of the cavity
- The deep cavities were filled with epoxy concrete in layers of 15 cm leaving top 15 mm finishing with epoxy mortar.
- The top 15 mm was finished with 15 mm thick epoxy mortar of 1:6 proportion.
- The shallow cavities less than 5 cm deep were filled with epoxy mortar with prime coat
- The damaged teeth were also repaired in similar manner (Fig. 16). (CWPRS TR No. 3177,1994).

3.5.4 Rehabilitation of Manikdoh dam, Maharashtra

Manikdoh dam, a composite dam constructed across river Kukadi in un-coursed rubble masonry is located near village Manikdoh of Pune District, Maharashtra during the years 1976-1983. The dam is 53 metres in height and 927 metres in length comprising an overflow section of 92 metres in length, inspection and foundation gallery, each of size 2.3 m x 1.5 m. The upstream face of the dam was gunited soon after its construction.

Leakages were observed in the body dam leading to loss of water from reservoir, hence to identify suitable repair materials and to develop the repair methodology for arresting seepage. Suitability tests were carried out in laboratory on various epoxy/polymer/ cement concrete / mortar mixes from different manufacturers for repair of cavities and pointing work (CWPRS TR No. 4498, 2007).

From the nature of the damages it was decided to adopt following methodology for repair and strengthening of dam:

- Sounding of the entire u/s gunited portion to identify weak portions.
- Removal of rusted steel elements / loose masonry stones found the u/s face.
- Filling of cavities using repair concrete or mortar.
- Raking of masonry joints- pointing by repair mortar.
- Grouting dam body using cementitious grout.
- Guniting of u/s surface after applying a bond coat.
- Removal of leached material deposits inside the galleries.

3.5.5 Rehabilitation of Anjunem Dam, Goa

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 has 11 blocks including a spillway comprising four bays each of 7.62 m width and 5 piers each of 3.0 m width.

The seepage quantity recorded was of the order of 3608 liter/minute including about 525 liter/minute from below the foundation gallery (Fig. 17). Sweating was also observed on the d/s surface of the dam. As a first step of studies, the dam was inspected for identifying the damages and suitability tests were carried out in laboratory on various epoxy/polymer/ cement concrete / mortar mixes from different manufacturers for repair of cavities and pointing work.



Fig. 17: Seepage in gallery & pumped out seepage

After careful examination of the damages and their nature, it was proposed to undertake the rehabilitation of upstream surface by filling of cavities and pointing of the joints at first and to observe the seepage conditions followed by dam body/foundation grouting.

On the basis of laboratory test results and in view of the exposure of repaired surface to ultraviolet radiations (sun rays), the Polymer based cementitious compound was selected for use. An area of about 640 m² is repaired out of total upstream face area of 5400 m². These repairs resulted in reducing the seepage by about 25-30% which is quite significant. If the repairs of entire damaged area including those under water are undertaken, the seepage would reduce further. (CWPRS TR No. 5019, 2012).

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CHAPTER 4

SAFETY AND REHABILITATION OF EARTH / ROCKFILL DAMS

B Muralidhar (Scientist C)

4.1 INTRODUCTION

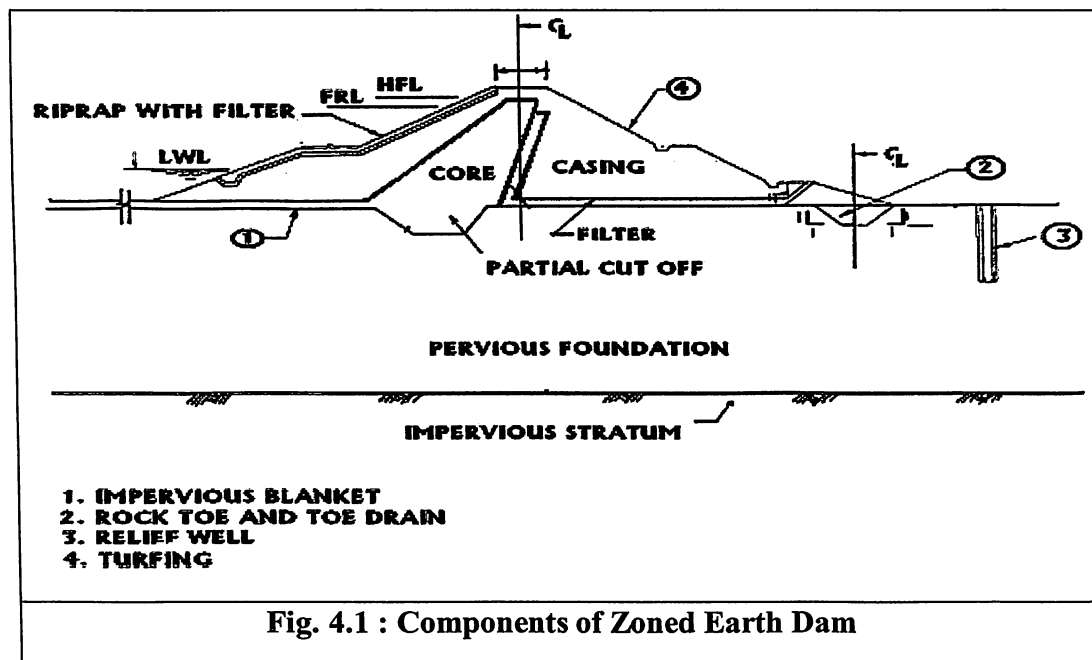
Dams are designed to withstand all the possible destabilizing forces with a certain factor of safety. It has been an indicator of a factor of ignorance or lack of knowledge of various response processes of materials used in construction, the stresses caused, the strains experienced and finally the failure mechanism. Design of large dams has to be extra safe so that there is a minimum probability of their failure. Sudden release of storage can cause disproportionate flooding and losses to the human habitats in the downstream. Failures of dam by overtopping and seepage/ piping constitutes 25 – 30 % each, Slope slides and leakages cause 13-15 % each, offers such as damage to slope paving, unknown etc constitutes 5 – 7 %. It is a fact that the number of earth dams is more than the masonry and gravity dams. The earth dams being constructed with locally available soils, their performance on seepage aspect is relatively poor. Many earth dams require to be rehabilitated so as to prevent the loss of precious water. The overall safety and stability of dams require various aspects to be addressed. The chapter covers types of soil required for embankment construction, slope stability analysis, dynamic analysis of earth dams and liquefaction potential of foundation against site specific earthquake, remedial measures in case of likely failure and instrumentation required for monitoring the behavior of dam for safe operation.

4.2 SUITABILITY AND TYPE OF SOIL FOR CONSTRUCTION

The embankment dams are constructed with locally available soil. These dams are suitable for any type of foundation; as such rock in the foundation is not important; Height of dam can be raised easily in relatively safe earthquake areas. The embankment dams are classified as i) Homogenous embankment ii) Zoned embankment iii) Rockfill dams with central clay core, and iv) Rockfill dams with upstream Face membranes.

i) **Homogenous embankments:** This dam section consists almost entirely of one type of material. This type is adopted due to compulsions of material availability within a reasonable distance at site. Usually this type of section is made of low permeability material and requires flatter slopes than a zoned section.

ii) **Zoned embankment:** This uses two or more types of materials, depending on their availability, utility and costs. There is an impervious zone called the Core inside the section (Fig 4.1). The outer zones on both sides, called casing are of pervious materials. These zones are separated by filters. If the casing material is not pervious enough, it may still be necessary to provide internal drainage. Zoned dams have greater stability during rapid drawdown. They are suitable for large heights.



iii) **Rock fill dams with central core:** These dams have rock fill zones on both sides, with an impervious zone in the middle, and transition zones and/or filters in-between. Rock fills are fragmented rock, either natural boulder deposits. Good quality rock fill provides free drainage and high shear strength.

iv) **Rockfill dams with upstream face membrane:** These types have concrete on upstream slope of the rockfill dam function as water barrier.

4.2.1 Types of Soil

Suitability of soils for construction of earth dams as per Indian Standard is shown in Table 1.

Table -I Suitability of soils for construction of earth dam:

Relative suitability	Homogenous Dykes	Zoned earth dam		Impervious Blanket
		Impervious	Previous	
Very suitable	Gravelly Clay (GC)	GC	Well graded Sand or Gravel (SW,GW)	GC
Suitable	Intermediate & Low Plasticity Clay (CL,CL)	CL,CI	Silty Gravel (GM)	CL,CI
Fairly suitable	Poorly graded sand (SP), Silty sand (SM), High plasticity Clay (CH)	OM,OC, SM, Clayey sand (SC) ,CH	SP,OP	CH,SM, SC,OC

Silty soil such as MI,MH, ML and all soil containing organic contents are not suitable. (IS: 8826-1978)

4.2.1.1 Component of embankment:

The different components of the embankment dams are i) Core ii) Casing iii) Cut-off, iv) Internal Drainage system and foundations v) Slope protection vi) Surface drainage vii) Impervious blanket viii) Relief well etc.

i) Core

The core provides impermeable barrier within the body of the dam. Impervious soils are generally suitable for the core. However, soils having high compressibility, high liquid limit and having organic content are avoided. They are prone to swelling and formation of cracks. The core may be located either centrally or inclined upstream. The locations will depend mainly on the availability of materials, topography of site, foundation conditions, diversion consideration, etc.

The main advantage of a central core is that it provides higher pressure at the contact between the core and foundation reducing the possibility of leakage and piping. On the other

hand, inclined core reduce the pore pressure in the downstream part of the dam and thereby increases its safety. The section with an inclined core allows the use of restively large volume of random material on the downstream side. The practical considerations that govern the thickness of the core are i) Availability of suitable impervious material ii) Resistance to piping iii) Permissible seepage through dam iv) Availability of other materials for casing, filter etc v) Minimum width that will permit proper construction. The top level of the core generally should be fixed at 0.5 m above design MWL.

Suitability of soils for construction of core in earthquake zones depends upon particle size gradation, plasticity of the clay etc. The soil which is a well graded, coarse mixtures of sand, gravel and fines having $D_{85} > 60\text{mm}$, $D_{50} > 8\text{mm}$ and cohesionless fines (0.075 mm) $< 20\%$, are **most suitable** as core. The soil is considered **good suitability** for core if a well graded mixture of sand, gravel and clayey fines with $D_{85} > 25\text{ mm}$ and having plastic index $\text{fines} > 12$. The soil is graded **fairly suitable** as core when fairly well graded , gravelly, medium to coarse sand with cohesion less fines having $D_{85} > 19\text{mm}$, D_{50} between 0.5mm and 3.0mm , fines $< 25\%$ and $\text{PI} > 25$. Clay of low plasticity ($\text{PI} = 5$ to 8 , $\text{LL} > 25$) or Silts of medium to high plasticity ($\text{PI} > 10$) with little coarse fraction, are **poorly suitable** as core. Finally, the soil that are **not to be used** as core are the Fine, uniform, cohesion less silty sand having $D_{85} < 0.3\text{mm}$. The expensive, dispersive soil shall not be used in the construction of the embankment dam.

ii) Casing

The function of casing is to impart structural stability of the dam and protect the core. The relatively pervious materials which are not subjected to cracking on direct exposure to the atmosphere are suitable for casing. Upstream slope protection measures are to provide Hand place riprap, Dumped riprap, Cement concrete facing etc. Downstream slope protection measures are providing Turfing, Stone pitching, Network for open paved drains, Geonet etc

iii) Cut-Off

It is used to reduce loss of stored water through foundations and abutments and thereby prevent subsurface erosion by piping. Cut off may be in the form of sheet piling,

cement bound curtain, diaphragm wall of bentonite, concrete of other impervious materials. The type of cut-off is decided based on the detailed geological investigation. A cutoff which completely penetrates and seals the pervious foundation strata is called **positive cutoff**. It should be keyed at least to a depth of 400 mm into continuous impervious sub stratum. The alignment of the cut-off should be fixed in such a way that it's central line should be within the base of the impervious core. Minimum width of COT is 4 m for 10 to 30 % of Head.

Where it is not possible to provide positive cut-off, partial cut-off with or without upstream impervious blanket may be provided. The partial cut off is specially suited for the horizontally stratified foundation with relatively more pervious layer near top. The depth of the partial cut-off in deep pervious alluvium will be governed by i) Permeability of substrata, ii) Relative economic of depth of excavation governed usually by cost of dewatering iii) various length of upstream impervious blanket.

iv) Internal Drainage System

Internal drainage system comprises of i) inclined or vertical filter, ii) a horizontal filter, iii) a rock toe and iv) toe drain. The design of filter takes into account the particle size distribution and the shape of the sand/gravel. Moreover, the stability of the base soil adjacent to a given filter depends on its resistance to drag forces. Inclined or vertical filter is to be provided as specially to protect the core material from migration. Filters also help in controlling the leak through crack in the core. A transition filter zone between the core and the downstream shell would be necessary in case of rock fill dams. It is required to provide adequate toe protection. (IS:9429-1980)

v) Slope Protection

The upstream slope protection is ensured by providing riprap. The riprap can be placed on the slope either by hand or simply dumped. The thickness of riprap is not less than 300 mm. The downstream slope protection is ensured by providing riprap or turfing. If the annual rainfall is less than 200 cm, it is usual practice to protect the downstream slope from rain cuts by providing suitable turfing on the entire slope. In case if annual rainfall is more than 200 cm, a riprap of 300 mm thick is provided. Details of downstream slope protection

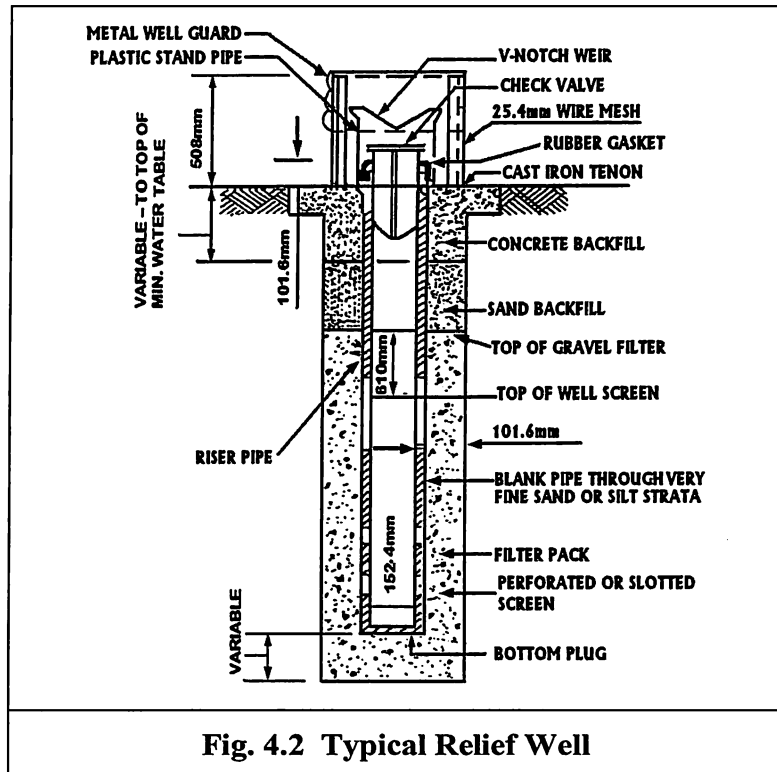
such as prevention from erosion by Rain-wash, Prevention from erosion by tail water etc. are given in IS:8237-1985.

vi) Impervious Blanket

The horizontal upstream impervious blanket is provided to increase the path of seepage, when full cut-off is not practicable on pervious foundations. It may be provided either with or without partial cut-off. It shall be connected to core of the dam. Its permeability should be far less than the foundation soil. The use of soil with high plasticity for blanket will lead to formation of cracks. In order to prevent cracking due to exposure to atmosphere, spreading of a layer of random material of 300 mm thick, over the blanket is recommended. The details of design of blanket and suitability of soils for its construction are given in *IS: 8414* and *IS 1498* respectively. The general design guideline is to have a blanket of minimum thickness of 1.0 m and a minimum length which is 5 times the maximum water head.

vii) Relief Wells

It is used to reduce the pore pressure developed in the foundation. It consists of small drainage well (45 to 90 cm in dia) sunk near the downstream toe of an earth dam. It has a slotted pipe (about 10 to 15 cm in dia) placed in the centre which is surrounded by graded sand filter media. This arrangement permits the ingress of seepage water into the well, allowing it to rise to the outfall (relief) level, where the pressure gets relieved. They are installed to ensure safety in cases where seepage control is depended only on partial cut-off or upstream blanket. Its use is imperative if top strata is impervious followed by pervious strata. A system of relief wells suitably spaced are installed, to reduce the intensity of the under seepage pressure. The seepage water is safely and conveniently led to a natural drainage channel. (Fig. 4.2) (*IS:5050: 1992*)



4.3 SAFETY REQUIREMENTS IN DESIGN OF EMBANKMENT DAM

The basic requirements for design of embankment dam are to ensure Safety against overtopping, Slope stability, Safety against internal erosion, Phreatic line within downstream face, Safety against wave action. The details of these safety aspects are as follows:

4.3.1 Overtopping

Sufficient spillway and outlet capacity should be provided to prevent overtopping of earth embankment during and after construction. The freeboard should be sufficient to prevent overtopping by waves and should take into account the settlement of embankment and foundation. In case of unyielding foundation, the amount of settlement for the embankment should be restricted to 1.0 percent of the height of dam. In case of compressible foundation, the settlement should be computed based on laboratory test results and same should be provided by increasing the height of dam correspondingly. Longitudinal camber should be provided on the top of dam along the dam axis to provide for settlement. The camber varies from zero height at the abutments to maximum at the central section in the valley where maximum settlement is anticipated. Guidelines for free board requirements in

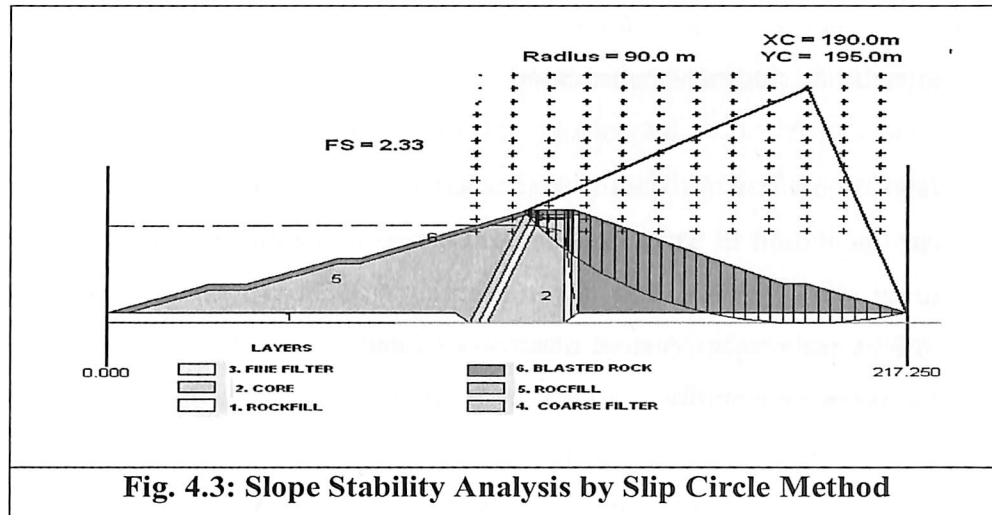
Embankment dams are explained in IS:10635-1993. There should be no risk of over topping of the dam section. The most important aspect of this criteria is estimation of the design flood and provision of adequate spillway capacity to pass that flood with required net freeboard to protect the dam crest against wave splash.

4.3.2 Slope Stability

The purpose of slope stability analysis is to provide a quantitative measure of the stability. The slopes of the embankment should be stable under all loading conditions. It is expressed as the Factor of Safety (FS) against failure. It is defined as the ratio of Restoring forces to the Disturbing forces, for an assumed potential failure surface. FS for several failure surfaces are computed and the minimum value of FS is considered for stability. The value of FS less than 1.0 indicates failure of the slope. There are different analytical methods available to carry out stability of earth / rockfill dams such as Swedish slip circle, Bishop's Simplified method, Spencer method, Janbu's rigorous method, Morgenstern-Price, etc. These methods are called limit equilibrium methods and do not give the deformation of the failed slope. Now days, Numerical methods software are available which give FS as well as the likely deformation of the failed slope. Embankment slopes are to be designed in accordance with the provisions contained in IS:7894-1975. Slope stability analysis by slip circle method is shown in Fig. 4.3

The critical issues in the analysis are i) potential failure mechanism, ii) geometry of sliding, iii) pore pressure and iv) shear strength of soils. The dams have to be assessed for following conditions:

- i. Downstream slope for steady state seepage
- ii. Upstream slope for rapid drawdown
- iii. Construction condition for both side slopes.
- iv. Earthquake loading



4.3.3 Internal Erosion

The seepage through the embankment and foundation should be such as to control piping, erosion and sloughing and excessive loss of water. Seepage control measures are required to control seepage through dam and seepage through foundation. Design for control of seepage through dam shall be made in accordance with provisions contained in 'Indian Standard drainage systems of earth and rock fill dams.' Design for control of seepage through foundation may be made in accordance with provisions contained in IS: 8414-1977.

The phreatic/ seepage line should be well within the downstream face of the dam section. If the dam section is homogeneous and no drainage arrangements are made, any seepage is going to emerge on the downstream face. This results in "sloughing" or softening of the downstream face and may lead to local toe failure, which may progressively develop upwards. This can be safeguarded against by providing a free draining zone on the downstream face or by intercepting the seepage inside the dam section by internal drainage.

4.4 INSTRUMENTATION OF EMBANKMENT DAMS

All embankment dams should have an adequate level of instrumentation to enable design engineers to monitor and evaluate the safe performance of the structures during the construction period and under all operating conditions. This includes all appurtenant structures and facilities whose failure or malfunction would cause or contribute to loss of life, severe property damage, or loss of function or interruption of authorized mission.

Instrumentation is not a substitute for an inadequate design. It is a tool to monitor and verify the performance of the design as constructed.

In view of concerns for dam safety, it has become increasingly important to provide sufficient instrumentation in earth and rock-fill dams for monitoring the performance of the structure during construction, and for all anticipated stress conditions throughout the operational life of the project. Visual observations and the interpretation of instrumentation data from the embankment, foundations, abutments, and appurtenant features provide the primary means for engineers to evaluate dam safety. In recent years, technology of devices for measuring seepage, stresses, and movements in dams has improved significantly with respect to accuracy, reliability, and economics. These technologies should be used to the extent necessary to acquire sufficient information within the required timeframe to assure the thorough understanding of dam performance.

4.4.1 Plan and Records

The planning, design, and layout of an instrumentation program are integral parts of the project design. Instrument data are an extremely valuable asset that supplies an insight into the actual behavior of the structure relative to design intent for all operating conditions, establishes performance that is uniquely characteristic to the dam, and provides a basis for predicting future behavior. As structures age and new design criteria are developed, the historical data provide most of the information necessary to evaluate the safety of the structure with respect to current standards and criteria. Older structures may require additional instrumentation to gain a satisfactory level of confidence in assessing safe performance. Instrument data can be of benefit only if the instruments consistently function reliably and the data values are compared to the documented design limits and historical behavior. Automation of dam safety instrumentation is a proven, reliable approach to obtaining instrument data and other related condition information.

4.4.1.1 System Design

The design and construction of new projects as well as the rehabilitation, dam safety modifications, and normal maintenance of older projects present opportunities to prepare for

the future engineering analyses of structural performance. As a minimum, the parameters that are critical to satisfactory performance will dictate the selection of instrument types. Generally, the types of measurements are (1) Horizontal and vertical movement, (2) Alignment and tilt, (3) Stresses and strains in soil and rock fill, (4) Pore pressure, (5) Uplift pressure, (6) Phreatic surfaces, (7) Seepage clarity and quantity.

In all circumstances, background information that may affect the validity of the data or the analysis of the performance (such as hydrologic or weather conditions) is documented and baseline instrument data for each type of measurement is obtained for future comparison.

4.4.1.2 Installation and Maintenance

Instrumentation for a project should be included in the design phase, during construction, and throughout the operational life of the project as conditions warrant. After a project has been operational for several years, appropriate maintenance, repair, and replacement of instrumentation must be accomplished during the normal operation to assure continued data acquisition and analyses of critical performance parameters. Specialized expertise are required to install and maintain automated instrumentation.

4.4.1.3 Data Collection, Interpretation, and Evaluation

The frequency with which instrumentation data are obtained must be tailored to the monitoring purpose, period of construction, investigation, or other interest, and project operating conditions. In all cases, sufficient calibration must be performed and background data must be obtained to ensure that a valid and reliable database is developed, maintained, and available to facilitate subsequent comparisons. After a baseline of performance is established, the subsequent reading of instruments during construction and operating conditions should be based on an anticipated rate of loading or changes in reservoir levels. The evaluation of the data should follow immediately. As a minimum, all data should be plotted as instrument response with respect to time, as well as reservoir level or other range of loading.

4.4.1.4 Documentation

Information relative to instrumentation systems is an invaluable resource that is necessary to evaluate instrument and system performance, as well as influence the assessment of dam performance and should be preserved and readily accessible.

4.4.2 Types of Instrumentation

The type, number, and location of required instrumentation depend on the layout of the project and the construction techniques employed. Devices may consist of the following: i) piezometers (open tube, Casagrande type, electrical, vibrating wire, or occasionally closed systems) located in the foundation abutment and/or embankment, ii) surface monuments, iii) settlement plates within the embankment, iv) inclinometers, movement indicators (at conduit joints, outlet works, and intake tower), v) internal vertical and horizontal movement and strain indicators, vi) earth pressure cells, and vii) accelerographs (in areas of seismic activity).

4.4.2.1 Piezometers

The safety of a dam is affected by hydrostatic pressures that develop in the embankment, foundation, and abutments. Periodic piezometer observations furnish data on pore water pressures within the embankment, foundation, and abutments, which indicate the characteristics of seepage conditions, effectiveness of seepage cutoff, and the performance of the drainage system. At each cross section that piezometers are placed, some should extend into the foundation and abutments and be located at intervals between the upstream toe and the downstream toe, as well as being placed at selected depths in the embankment. Two of the more important items in piezometer installation are the provision of a proper seal above the screen tip and the water tightness of the joints and connections of the riser pipe or leads.

4.4.2.2 Surface Monuments

Permanent surface monuments to measure both vertical and horizontal alignment should be placed in the crest of the dam and on the upstream and downstream slopes. Monuments should be embedded in the embankment by means of a brass or steel rod encased

in concrete to a depth regionally appropriate to avoid frost action. Guidance on spacing is as follows: 50-ft intervals for crest lengths up to 500 ft, 100-ft intervals for crest lengths to 1,000 ft, and 200- to 400-ft intervals for longer embankments. These monuments should be installed as early as possible during construction and readings obtained on a regular basis.

4.4.2.3 Inclinerometers

Inclinerometers should be installed in one or more cross sections of high dams, dams on weak deformable foundations, and dams composed at least in part of relatively wet, fine-grained soils. Inclinerometers should be installed particularly where dams are located in deep and narrow valleys where embankment movements are both parallel and perpendicular to the dam axis. Inclinerometers should span the suspected zone of concern. It is essential that these devices be installed and observed during construction as well as during the operational life of the project.

4.4.2.4. Movement Indicators

Various types of instrumentation may be installed to measure horizontal spreading of the embankment (particularly when the foundation is compressible), movements adjacent to buried structures, foundation settlement, and internal strains. Strain measurements are particularly significant adjacent to abutments and below the crest to detect cracking of the core. Where there is a possibility of axial extension, as near steep abutments, surface monuments should be placed on the crest at 10.0 m intervals to permit measurement of deformations along the axis.

4.4.2.5 Pressure Cells

The need for reliable pressure cells for measuring earth pressures in embankments has long been recognized, and much research has been done toward their development. Although many pressure cells now installed in earth dams have not proved to be entirely satisfactory, newer types are proving to be satisfactory and increased usage is recommended. Some types of pressure cells installed at the interface of concrete structures and earth fill have performed very well.

4.4.2.6 Accelerographs

It is desirable to install strong motion, self-triggering recording accelerographs to record the response of the dam to the earthquake motion. Digital accelerographs are recommended. The digital units record and provide fundamental event information on a near real-time basis and should be incorporated into dam safety monitoring programs.

4.4.2.7 Weirs for Seepage Measurements

The seepage flow through and under a dam produces both surface and subsurface flow downstream from the dam. The portion of the total seepage that emerges from the ground, or is discharged from drains in the dam, its foundation, or abutments, is the only part that can be measured directly. An estimate of the quantity of subsurface flow from flow net studies may be based on assumed values of permeability. The portion of the seepage that appears at the ground surface may be collected by ditches or pipe drains and measured by means of weirs or other devices.

4.4.3 Automated Data Acquisition Systems

It is possible to install and operate automated instrumentation systems that provide cost-effective real-time data collection from earth and rockfill dams. Installation of these computer-based automated data acquisition systems (ADAS) provides for more accurate and timely acquisition, reduction, processing, and presentation of instrumentation data for review and evaluation by geotechnical engineers. Consideration should be given to providing an ADAS for all new dam projects, dam safety modifications to existing dams, and monitoring system rehabilitation that are necessary to assure appropriate data acquisition.

4.5 DYNAMIC ANALYSIS OF EARTH DAM

Deformation of earth dam due to an site specific earthquake is carried out by the method proposed Prof.B.Seed and F Makdisi, which is based on the concept of Prof. Newmark. The method assumes that failure occurs on a well defined slip surface and that the material behaves elastically at stress levels below failure but develops a perfectly plastic

behavior above yield. Seed and Makdisi carried out finite element dynamic analysis on selected earthen dams and they prepared empirical relations in the form of design chart. The earthquake induced deformation can be determined by using these charts. The steps involved in the method are given below:

- I. Conduct slope stability analysis with different of horizontal seismic coefficient to determine Yield acceleration (K_y).
- II. Conduct Dynamic Response Analysis of the dam for a given Maximum credible earthquake (MCE) by finite element method, to determine Maximum Crest acceleration ($\ddot{U}_{max}$). Using this, evaluate acceleration of the Sliding mass (K_{max}) from first empirical relation.
- III. Using values of K_{max} and K_y , evaluate the crest displacement (U) for different magnitudes of Earthquakes, from second empirical relation.

4.5.1 Yield Acceleration (K_y)

The yield acceleration, K_y , is defined as that average acceleration producing a horizontal inertia force on a potential sliding mass and causes it to experience permanent displacement. It is determined from slope stability analysis with different values of horizontal acceleration. The value of horizontal acceleration for which FoS reduces to 1.0, is called Yield acceleration. It is the threshold limit between stability and instability.

4.5.2 Maximum Crest Acceleration ($\ddot{U}$)

Earth / Rockfill dam section is idealized by Finite Element mesh as shown in Fig 4.4. Mesh contains 863 nodes and 840 elements. The dynamic properties of the soil such as Shear modulus (G) and Damping ratio (D), which are shear strain (γ) dependent, are determined from **Resonant column test** in laboratory. These non-linear properties are shown in Fig 4.5. Site specific earthquake in the form of acceleration-time history is applied to the base of the dam. Fig. 4.6 gives the Acceleration – time history of Maximum Credible Earthquake (MCE) record. Dynamic response analysis of the dam is carried out using computer software such as QUAD-4, FLAC-2D etc. The analysis solves the dynamic equation of motion:

$$\mathbf{M}\ddot{\mathbf{x}} + \mathbf{D}\dot{\mathbf{x}} + \mathbf{K}\mathbf{x} = \mathbf{F}(t) \quad (1)$$

Where,

M , D and

K

$F(t)$

$\ddot{X}$, $\dot{X}$, X

Mass, Damping and Stiffness matrix of assembly of the dam

Site specific earthquake

Nodal acceleration, nodal velocity and nodal displacement

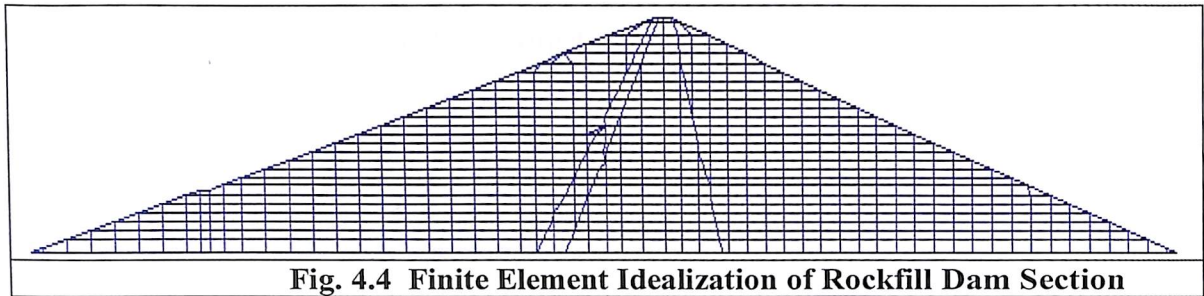


Fig. 4.4 Finite Element Idealization of Rockfill Dam Section

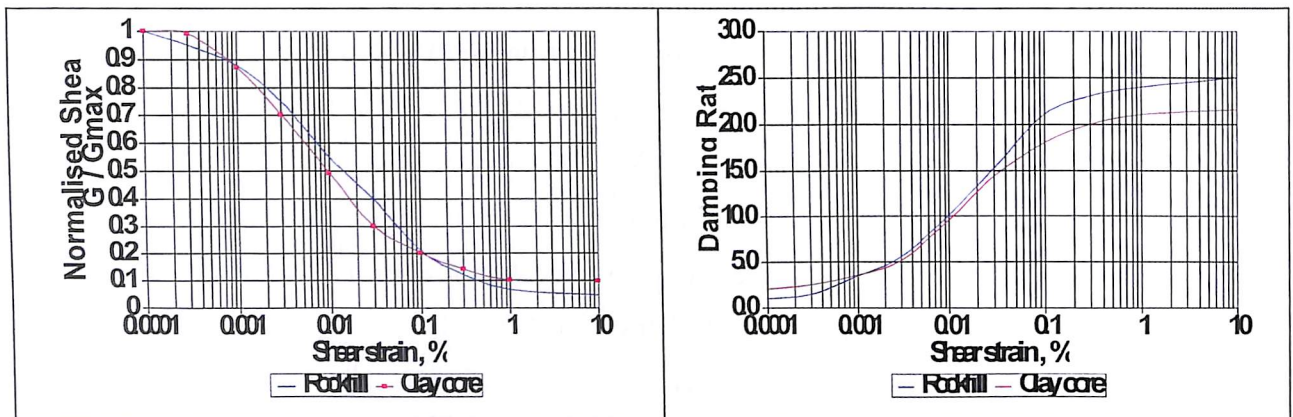


Fig. 4.5a : Shear Modulus Vs Shear Strain

Fig. 4.5b : Damping Ratio Vs Shear Strain

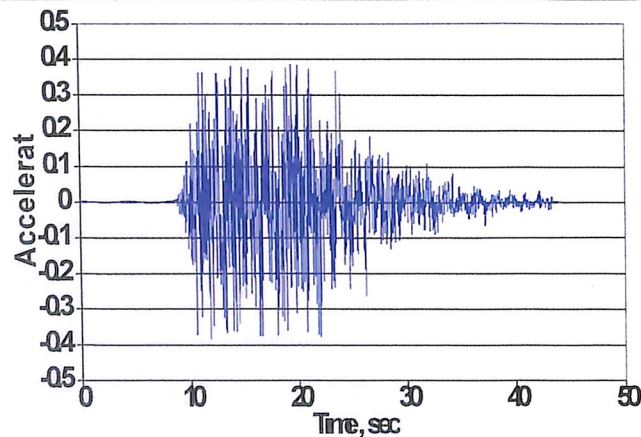


Fig. 4.6 : Horizontal Acceleration-Time History of Maximum Credible Earthquake

The dynamic response analysis gives maximum horizontal and vertical acceleration that are likely to be induced at each nodal point. The Fundamental period (T_0) of the dam was found to be 1.33 sec. The first design curve of Prof. Seed is shown in Fig.4.7.

4.5.3 Crest Displacement (U)

Prof. Seed's second design curve is shown in Fig 4.8. This curve gives variation of the ratio (K_y / K_{max}) with 'Normalized permanent displacement' ($U / K_{max} \cdot g \cdot T_0$) for different magnitudes of earthquakes ($6 \frac{1}{4}$, $7 \frac{1}{2}$ and $8 \frac{1}{4}$), where 'U' is the actual displacement. It is mentioned that permanent displacement of the sliding mass occur only if the ratio (K_y / K_{max}) is less than 1.0. For a given ratio of (K_y / K_{max}), normalized displacement can be found out for a given magnitude of earthquake. Actual crest displacement (U) of the sliding mass that is likely to undergo is determined from the value normalized displacement.

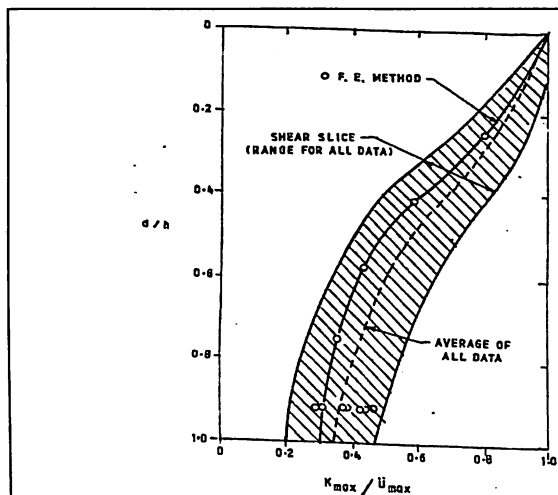


Fig 4.7 : Seed's Empirical Curve-1

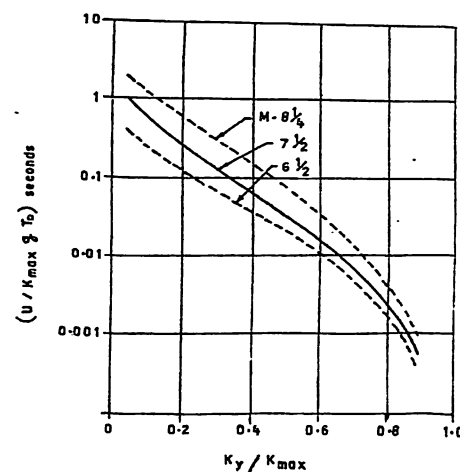


Fig 4.8 : Seed's Empirical Curve-2

4.6 FOUNDATION LIQUEFACTION

Liquefaction is a phenomenon in which a saturated silty-sand foundation stratum behaves like a liquid when subjected to an earthquake. In liquefaction, the superstructure sinks in to the ground because of reduction in the effective stress of the foundation. Many structures had been damaged due to liquefaction during Bhuj earthquake. The pore water

pressure increase in foundation soil during the earthquake causes reduction of effective stress. The liquefaction resistance of an element of soil depends on how close the initial state of the soil is to the state corresponding to “failure” and on the nature of the loading required to move it from the initial state to the failure state. Characterization of liquefaction resistance developed along two lines: methods based on the results of laboratory tests, and methods based on in situ tests, such as Standard Penetration tests, Cone Penetration test, Shear wave velocity etc.

Steps for determining the Zone of liquefaction in the field from laboratory tests are as follows:

- i. Design earthquake is established
- ii. Shear stress time history induced at various depths in the foundation are determined from the dynamic response analysis for the given design basis earthquake.
- iii. The shear stress time histories are converted to number of equivalent cycles (N_{eq}) ($B M Das$)
- iv. Using laboratory tests, determine the magnitude of the cyclic shear stress required to cause initial liquefaction in the field in N cycles at various depths. These can be plotted with depth.

The zone in which the cyclic shear stress levels required to cause initial liquefaction are equal or less than the equivalent cyclic shear stresses induced by an earthquake is the zone of possible liquefaction.

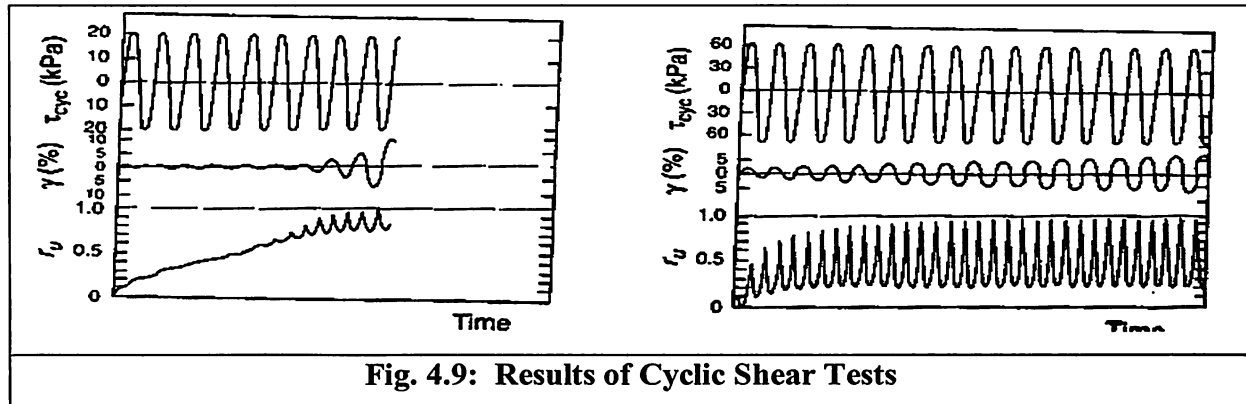
Liquefaction is determined from i) Laboratory tests and ii) In situ/ Field tests.

4.6.1 Laboratory Tests

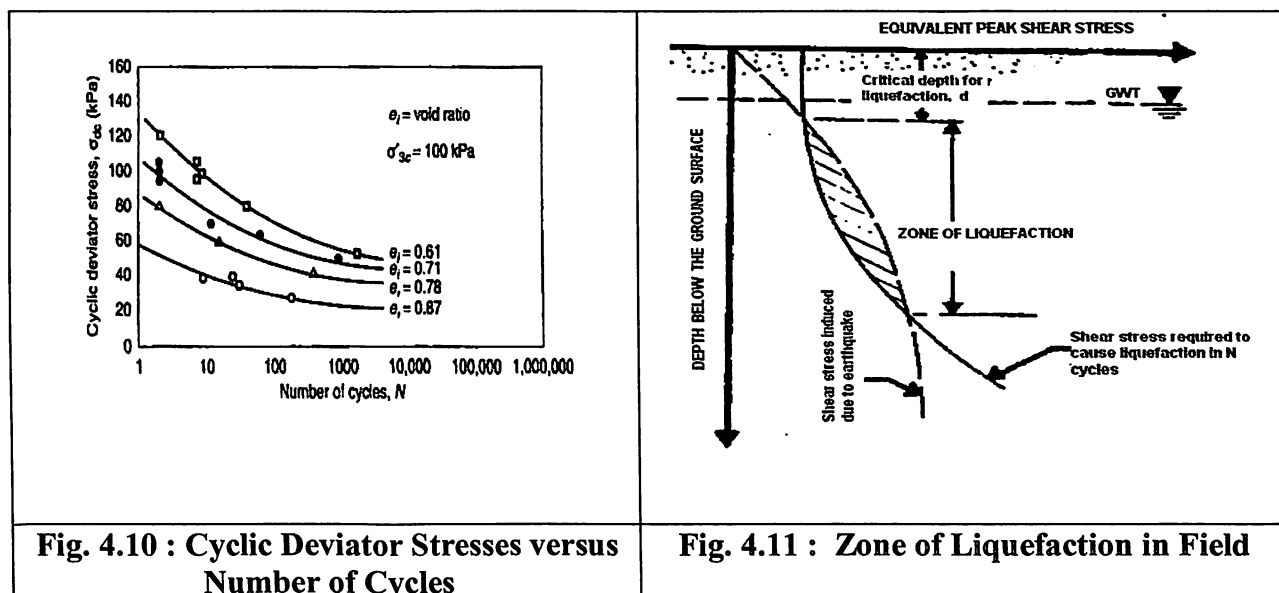
Cyclic shear stress will be imposed on the soil element due to ground shaking. This situation is simulated in laboratory tests. The most commonly used laboratory test procedures are Cyclic triaxial shear test and Simple shear test. (*S L Kramer 2005*)

Cyclic triaxial tests or simple shear tests are conducted to determine liquefaction failure in sand. The liquefaction failure is defined as the point at which the pore pressure

reaches the confining pressure or cyclic strain amplitude (5 %) is reached. The number of loading cycles required to produce liquefaction failure (N_L) decreases with increasing shear stress amplitude and with decreasing density. Fig. 4.9 shows Test record of cyclic shear tests and cyclic deviator stress versus Number of loading cycles under confining pressure.



- A. Loose sand with 47% relative density, liquefies at 10th loading cycle
- B. Dense sand with 75% relative density with high cyclic shear stress not liquefied.



Cyclic Stress Ratio (CSR) is the cyclic strength of soil normalized by the initial effective overburden pressure, for different void ratio. (Fig. 4.10)

4.6.2 Field Tests

Liquefaction of foundation is determined from Simplified Procedure proposed by Seed and Idriss (1971). As the seismic loading is excited at the base of the soil column, the

shear wave propagates to the ground surface and shear stress is generated in the soil column. The shear stress time history during earthquake is random in nature. An average of shear stress (τ_{ave}) is normalized with the initial effective overburden pressure (σ_0'), is called Cyclic Stress Ratio (CSR) and is given as:

$$CSR = \frac{\tau_{ave}}{\sigma_0'} = 0.65 * \frac{\sigma_0'}{\sigma_0'} * \frac{a_{max}}{g} * r_d \quad (2)$$

- τ_{av} = maximum shear stress for rigid body
- σ_0 = total overburden pressure
- a_{max} = peak horizontal acceleration on the ground surface
- g = acceleration of gravity
- r_d = the stress reduction coefficient (1.0 at the top and 0.9 at 15.0m below)

The CSR is the seismic demand of a soil layer and determined for each depth.

Next, the capacity of the soil to resist liquefaction, called Cyclic Resistance Ratio (CRR) is determined. It is computed from Penetration resistance value (N_m = blow counts per 300mm of penetration), of Standard Penetration Test (SPT). Procedures are also available based on Cone Penetration Tests (CPT), shear wave velocity measurement, and Becker Penetration Test (Youd, et al., 2001).

The SPT blow counts, N_m , are corrected. The correction factors are applied for Overburden (C_N), hammer energy (C_E), bore hole diameter (C_B), rod length (C_R) and sampler size (C_S). The corrected SPT blow count $(N_1)_{60}$ is determined as follows:

$$(N_1)_{60} = N_m C_N C_E C_B C_R C_S \quad (3)$$

The value of SPT blow counts for soil with fines content can be adjusted to the equivalent clean sand value of $(N_1)_{60CS}$. This can be done by applying constants, α and β , that are functions of fines content. Hence, the effect of fines content (FC) on the value of CRR is included as

$$(N_1)_{60CS} = \alpha + \beta (N_1)_{60} \quad (4)$$

Where α and β can be determined for Fines Content (percent of soil having size < 0.075 mm) in the soil.

Prof. Seed's base curve for CRR versus SPT is shown in Fig 4.12. This curve is for earthquake with magnitude of 7.5. For other earthquake magnitudes, a magnitude-scaling factor (MSF) should be applied. (L T Youd 1997)

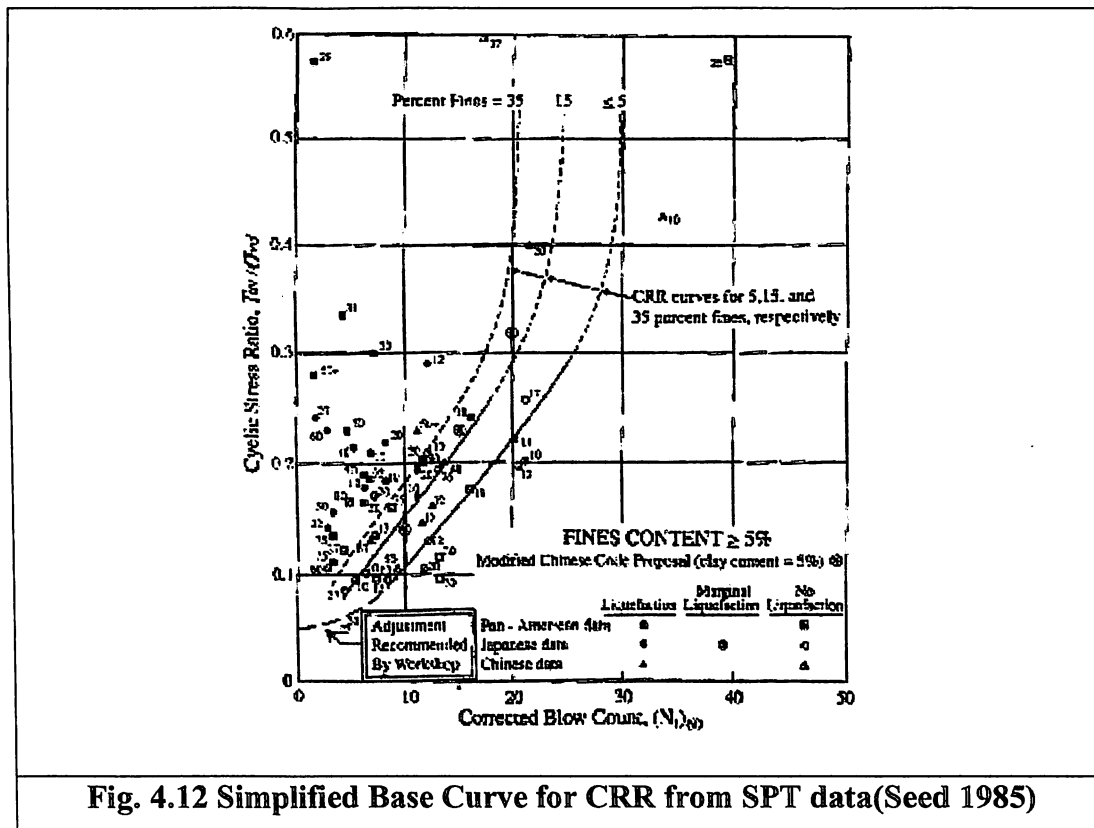
The Simplified Procedure applies for level to gently slope sites and for depths less than 15 meters. Therefore, the value of CRR should be corrected for greater depths that are for high overburden stresses. The K_σ is equal to unity for effective overburden pressure less than 1 tsf and then decreases with increasing effective overburden pressure.

$$CRR = CRR_{M=7.5} * MSF * K_\sigma \quad (5)$$

The Factor of safety against liquefaction can be written as :

$$FS_L = \frac{CRR}{CSR} \quad (6)$$

The value of $FS_L < 1.0$ indicates occurrence of liquefaction and $FS_L \geq 1.0$ indicates occurrence of 'No Liquefaction'. (Seed and Idriss, 1971)



4.7 REHABILITATION MEASURES

In spite of construction of dam as per the codal practices, the actual behavior of the dam differs during its operation due to uncertain / unexpected loads such as earthquake. It is true that using existing engineering technology, engineers today are capable of designing and constructing new dams that will behave acceptably during the design earthquake. If, for example, there are loose alluvial sands in the foundation, you simply remove them. On the other hand, engineers are faced with existing dams, founded on alluvial material that is potentially liquefiable. It is the seismic retrofit of these existing dams that are the key concern facing geotechnical earthquake engineers. In order to rehabilitate a deficient embankment dam to prevent potential seismic instability, one must either change the engineering properties of the dam and/or foundation; modify the geometry of the existing dam, or both. If predicted permanent deformations are estimated to be small and tolerable, then the dam is safe. On the other hand, if the deformations are intolerable and the dam is not to be taken out of service, then seismic remediation are required. There are a wide variety of treatments for rehabilitating dams are in use. (*Seismic Rehabilitation of Earth Dams, Journal of Geotechnical Engineering, Vol122, Jan 1996*)

4.7.1 Methods of Remediation

The present methods available for engineered remediation of seismically deficient earth dams are explained in the following paragraphs. (*Robin Fell 1992*)

4.7.1.1 Berms and Buttresses

Upstream and downstream berms and buttresses are used to increase the effective overburden pressure on the problem material and thus increase its liquefaction resistance. This increase in overburden also causes a small amount of consolidation and thus increases the density. Berms and buttresses are also used to increase the length of the failure surface, provide a counterweight to limit movement, and maintain a remnant section. The effectiveness of a berm is generally limited to a zone that is about as deep as the berm is thick. A berm or buttress can not reduce the factor of safety during day-to-day operation, and its presence is obviously verifiable. If coarse-grained soil or rock is available, berms and

buttresses can, with some difficulty, be constructed on the upstream shell without lowering the pool.

4.7.1.2 Excavate and Replace

This method assures that the problem material is removed and replaced with a non-liquefiable material. Excavation and replacement offers the advantage of providing relative assurance that what was designed has actually been constructed in the field. It is often expensive and operationally difficult. Dewatering is almost always required and in many cases the reservoir must be lowered significantly. This approach is most useful when the problem material is near the ground surface. In addition to excavating liquefaction prone material, the excavation-and-replace method can also be used after an earthquake to remediate shallow cracking.

4.7.1.3 In-situ Densification

When excavation and replacement are ruled out for some reason, in-situ densification can sometimes be used to decrease the potential for liquefaction by decreasing the void ratio of the problem material. The method includes vibro-techniques, dynamic compaction, compaction grouting, and displacement techniques. In-situ densification is most effective when the material to be improved is close to the ground surface and has limited fines content. To date this approach has not been used under an existing dam except in cases where most of the embankment over the foundation zone to be densified has been temporarily removed. Foundation densification will not be uniform and could adversely change the dam-foundation interface. Cracking might occur which could increase the risk of piping. Verification of the amount of improvement and of the spatial variability of the improvement is required.

4.7.1.4 In-situ Strengthening

While somewhat similar to in-situ densification, in-situ strengthening forms a composite material that is strong enough to ensure stability. Soil nailing, stone columns, and methods of deep soil mixing are examples. Some of these methods may also cause consolidation and increase the strength of the soil around the feature, but this increase in

strength is generally ignored in stability analyses. In-situ strengthening is generally most effective when the potentially liquefiable material is confined to a relatively thin layer but it can be implemented for thick deposits in the case of deep soil mixing. The method is to be applied with caution, if it is used under an existing structure. Conventional grouting of the foundation through the embankment has not been used for the purpose of strengthening for two reasons. One is the possibility of hydraulic fracture in the embankment. The other is that it is not easy to determine to what extent the grout has penetrated the zones needing improvement.

4.7.1.5 Increase Freeboard

An increase in freeboard may be used when the seismic analysis indicates that the dam is marginally stable and/or only small earthquake-induced deformations are probable. Obviously, this approach decreases the probability of overtopping associated with settlement or slumping of the crest.

4.7.1.6 Drainage

This approach provides for relief of seismically induced pore water pressure. Techniques include strip drains, stone columns, and gravel trenches. Gravel trenches can be used to intercept migrating elevated pore pressure plumes. Analysis of drains is problematic, because accurate and reliable in situ permeability are extremely difficult to obtain. Care must be used in placement of stone columns and gravel trenches so that stone crushing is minimized and permeability remains high. Stone column spacings should be sufficiently small to dissipate pore pressures to a low level during the earthquake, and to prevent the occurrence of high hydraulic gradients that could carry large amounts of fines into the gravel drains. Potentially, stone columns could be flushed periodically to maintain their performance. It is inevitable that extra drains added for remediation will reduce the length of flow lines and thus will increase the seepage gradients under static pool conditions. Hence, even if the drains are designed as filters with respect to the adjacent material, the static safety of the dam will be somewhat reduced and more water will filter through the dam.

4.7.2 Validation of Rehabilitated Dam

Good judgment requires that absolutely nothing be done to increase the likelihood of failure during normal static operating conditions such as increasing the likelihood of failure by piping, or hydraulic fracturing. One must also verify that any intended densification and/or drainage are achieved. Verification of densification is usually achieved by the use of a test section where before-and-after SPT, CPT, and/or shear wave velocity measurements are used to assess the effects of the densification technique attempted. If the technique is found successful in the test section, the same before-and-after index tests are used to verify densification of the improved zone.

4.8 CWPRS EXPERIENCES

CWPRS has been associated in solving cost-effective solutions to the safety, stability and rehabilitation aspects of embankment dams for many projects. One case study pertaining to Dudhawa dam is described below.

4.8.1 Seepage and Stability Analysis of Dudhawa Earthen Dam, M.P.

Dudhawa dam comprises a 2.9 km long and 24.7 m high earthen dam across Mahanadi river. The dam had shown signs of distress by way of leakages, sand boils since its first impoundment in 1962. This resulted in under utilization of its full capacity. Temporary remedial measures, such as relief wells and toe loading, were being carried out by the authorities as per the recommendation of the Dam safety Panel. Finite element seepage analysis were carried out for two sections, one at Chainage 40-42 km and another at 83-84 km, based on the prototype piezometric data. (CWPRS Technical Report No. 3160, 1994)

The classification of soil from the different zones of the dam is given in the Table II.

Table II: Zones of the Earthen Dam

Sr. No	Zone	Classification as per BIS	Permeability cm/ sec
1	Casing soil	SC	7.06×10^{-6}
2	Core soil	CH	1.0×10^{-8}
3	Foundation soil : Chainage 40 to 42 Km	SP	2.88×10^{-5}
	: Chainage 83 to 84 Km	CH	1.0×10^{-8}

Two dimensional seepage analysis was carried out for the two sections using Finite Element method (FEM) using computer software SOLVIA-TEMP. Fig. 4.13 shows the finite element idealization on the dam section. Analysis used 8 noded isoparametric elements. The soil permeability values as determined in the laboratory were used in the analysis is shown in Table II.

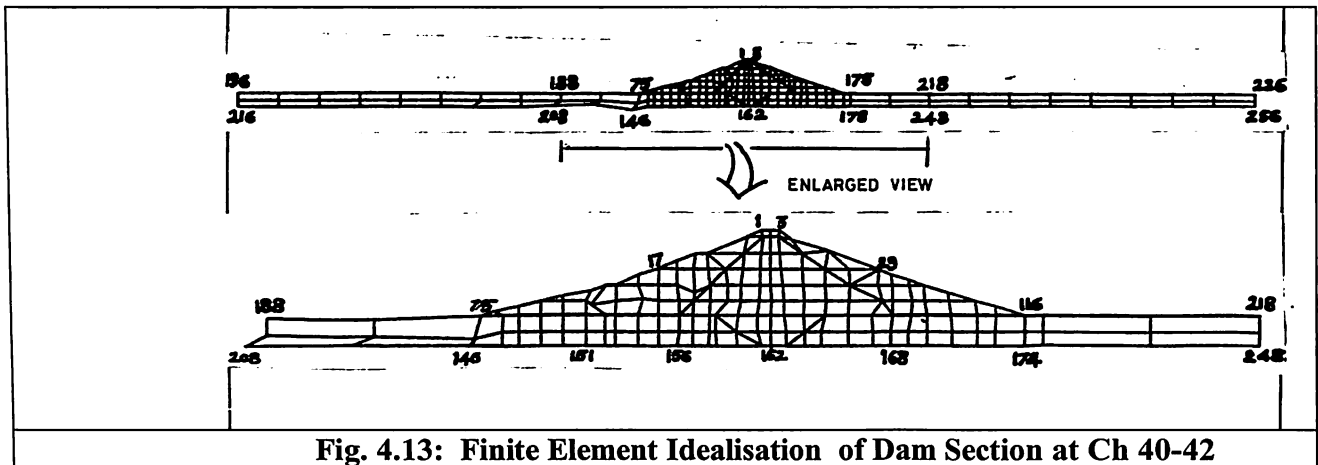
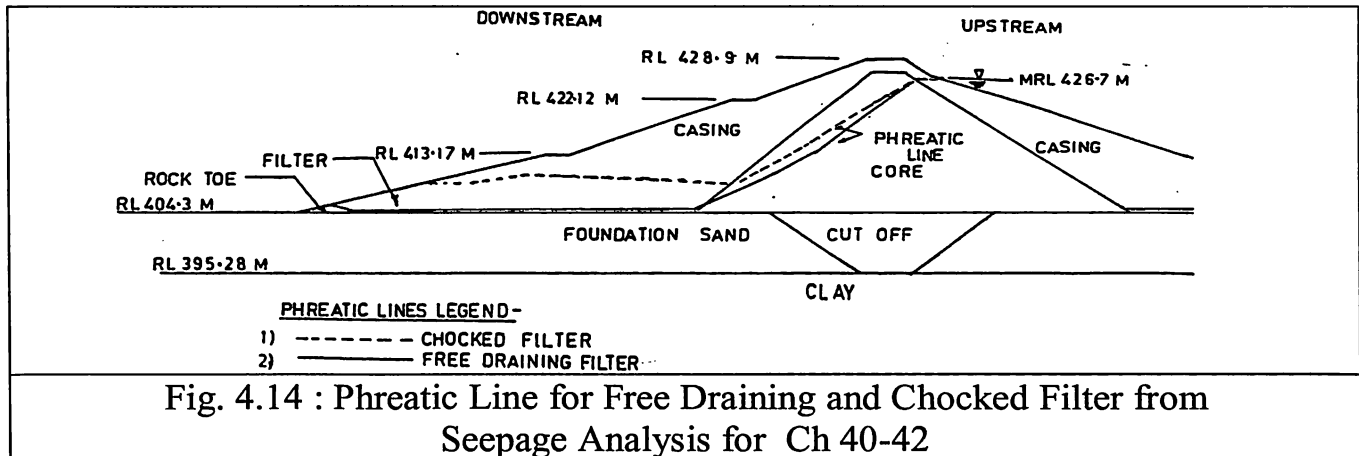


Fig. 4.13: Finite Element Idealisation of Dam Section at Ch 40-42

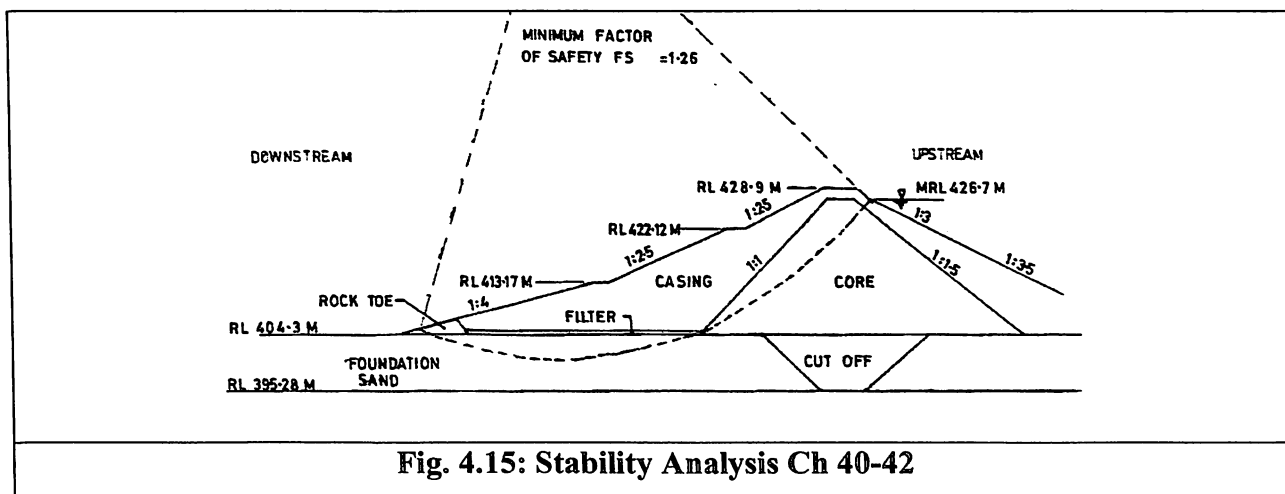
Boundary conditions for the analysis were maximum reservoir water level RL 423.76 m), tail water level (or piezometric head at toe). The analysis yielded values of the total head the nodal points and the water pressure values at the gauss points of the elements. Initially, analysis were carried out with considering the freely draining filter zone. The results of the analysis, however, indicated that the pressure heads on the down stream slope did not match with the actual observed piezometric data. Analysis was repeated with different lower values of the permeability of the filter/rock toe (i.e. not freely draining filter) such that the total head values of the analysis matched with the actual observed piezometric data. The contours of the pressure lines were drawn. The phreatic line, which is the zero pressure line, was identified from the pressure lines. It was observed that the phreatic line emerged close to the down stream slope of the earthen dam, which was confirmed from the field observations. These pressure lines data were used as input subsequently in the slope stability analysis (STABR).

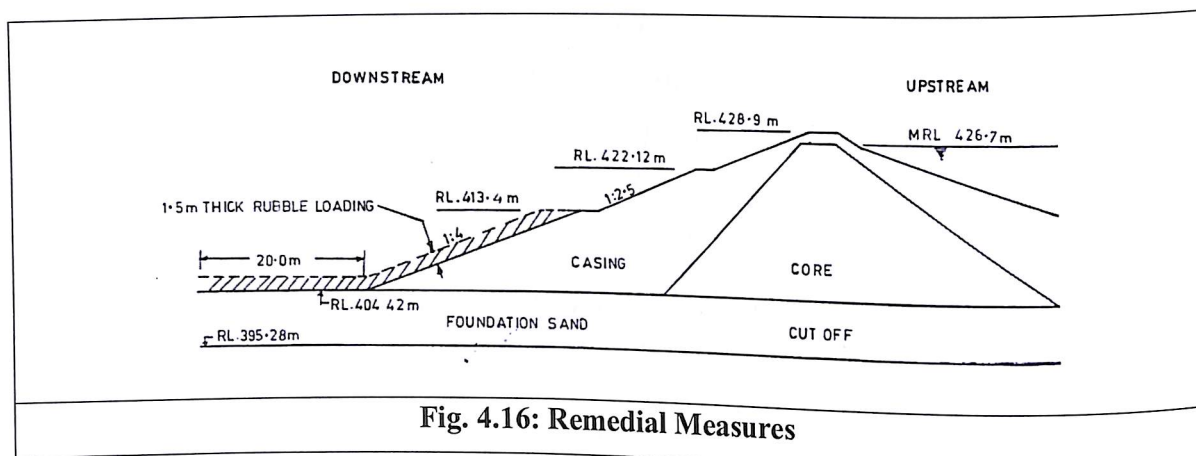
Slope stability analysis using Bishop's modified method was used to check the Factor of safety (FoS) of the slope of the dam. The program has a provision for incorporating the equipressure lines in the analysis. The equipressure lines as evaluated from the seepage analysis was used in this analysis. The FoS of 1.26 and 1.12 were evaluated for the sections at Ch 40-42 and Ch 83-83 respectively. Fig. 4.15 shows critical slip circle for the section.

These values of FoS are well below the FoS of 1.5 recommended by BIS. The high pore pressure in the downstream portion of the dam is due to the fact that the filter and rock toe do not function as required. This has resulted in the lower FoS in the stability analysis.



Remedial measures in the form of rubble fill loading on the downstream slope from RL 404.42m to RL 413.4 m for section between chainage 40-42 and from RL 411.50 m to RL 417.5m for section between Ch 83-84. The thickness of 1.5m rubble fill was computed from the exit hydraulic gradient from the seepage analysis. The large berm will counter balance the actuating forces and increase stabilizing forces and provide adequate overburden to improve stabilizing effective normal stresses. Fig. 4.16 shows the proposed remedial measures. A properly constructed drainage system must be provided below the loading berm to let all the seepage flow safely to the downstream.





4.9 CONCLUDING REMARKS

A safe and stable earth / rock fill dam is constructed by following the guidelines in the Standard codes in respect of selection of soils. The soil properties are determined from field tests as well as laboratory tests for deciding type and suitability of soil for construction. The design criteria is adopted from stability analysis and evaluation of earthquake and liquefaction resistant design from dynamic analysis. Probable uncertainties are a priori visualized and implemented to arrive at an optimum and safe section. In addition, instrumentation with regular monitoring and analyzing the data, will improve the safety of the dams.

However, dams constructed in the past, with knowledge of yester years, needs to be assessed for their stability and safety. The existing dams also need to be checked for stability by using the available numerical techniques for different loading conditions. The properties of the soil can be determined from detailed soil investigation of dam and foundation. The analysis of a dam which indicates excessive settlement or liquefaction due to earthquake, the remedial measures can be incorporated and dams are rehabilitated. The case study of Dudhawa dam and the case history of Bhuj earth dam have revealed that safety; stability and strengthening of embankments can be achieved by selecting type of soil, determining its properties, performing seepage, static and dynamic analysis, also by monitoring instrumentation data.

4.10 REFERENCES

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CHAPTER – 5

CONCLUSIONS

R.K.Kamble (Scientist E)

Large number of dams has been built all over India for development of water resources for irrigation, water supply, power generation and other benefits since independence to cater the need of large population. A huge amount of financial investment is made in planning, designing, construction, operation and maintenance of dams for storage of water to meet the needs of water supply, irrigation and hydropower for socio economic development and thus occupy a pivotal role in the development activities of the human race. Dams, however, are not unmixed blessings. They do pose a major hazard in the unlikely event of a failure. There have been about 200 notable reservoir failures in 20th century in the world so far. It is estimated that more than 8000 people lost their lives in these disasters. An unsafe dam also constitutes a hazard to human life and property in the downstream reaches. Safety of dams is very important for safeguarding the national investments and the benefits derived. As such proper analysis, diagnosis of dam distresses and their corresponding causes are essential to ensure dam safety and to adopt appropriate remedial measures to rehabilitate the dam. Realizing the importance of dam safety, many countries in the world have initiated action to review the safety of dams in their countries and United States of America can be considered a pioneer in this field. The review conducted recently by US Army Corps of Engineers revealed that out of 8819 review inspection completed, 2925 dams were evaluated as unsafe. Of the various causes, inadequate spillway capacity was the primary deficiency found in 81% of the unsafe dams. Keeping in view the importance of dam safety in our country, a Dam Safety Organization was established in May 1979 in Central Water Commission to assist the State Governments in various activities in dam safety.

Geological surprises, dam failures, occurrence of high magnitude earthquakes in geologically strong regions, advent of latest technology in the field of dam engineering and other regions enabled the dam engineers to make new safety guideline in the design of dams. All the present day dams have been designed based on the new concepts and designs. However, most of the dams in India were constructed based on the earlier design concepts. Now need arises to recheck and rehabilitate dams following of their compliance with current standards and also due to distresses resulting from ageing, foundation failure, seepage, heavy

floods, earthquakes, etc. In addition, for augmentation of storage capacity, it is necessary to increase height of the dam which necessitates rehabilitation of the structure. To reduce the risk of failures, regular health inspections are necessary to identify the defects by applying advanced and integrated methods. Investigations for dam safety and rehabilitation 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. Control measures should be adopted to mitigate problems so as to avoid future consequences. As such, timely adoption of monitoring, detection and analysis measures using conventional and non-conventional techniques and appropriate repair methodologies for rehabilitation of dams should be undertaken. This will ensure safe functioning of dams throughout its design life.

CWPRS a centenary organization and pioneer in the field water and water related structures over the years which is developed an unmatched expertise while rendering solutions to the field problems across the country and limited global involvement. The studies undertaken in all aspects of water, hydraulics, hydraulic structures and other aspects of water since 100 years were of diversified nature and result oriented. Suggestions made by the CWPRS proven their fruitfulness during successful maintenance of the hydraulic structures. To help the practicing and scientific fraternity CWPRS is publishing large number of technical outputs in the form of Memorandum, papers, notes, reports, etc.

In India, many dams are ageing and have various structural deficiencies and shortcomings in operation and monitoring facilities. To reduce the risk of failures, regular health inspections are necessary to identify the defects and whenever severe deficiencies are observed, comprehensive remedial measures are required to be undertaken. Once the occurrence of distresses and its causes are detected, decision is to be made to adopt suitable remedial measures for ensuring safety of the dam. The decision for remedial measures depends on several factors viz. type of dam, extent and severity of the distress, safety hazard posed to the structure, financial constraints, etc. Remedial measures generally adopted for concrete and masonry dams are buttressing, foundation grouting, body grouting, pointing, cable anchoring, concrete / mortar jacketing, geomembrane lining, steel jacketing, guniting, etc. Grouting aims at filling of 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 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.

The present technical memorandum is a compendium of expertise gained while finding solution to the field problems which will give an insight to the engineering and science fraternity to look into the field problems and relate them with their problems and sometime may find solution also. A limited theatrical approach is also provided so that the reader should not go in search of theory separately. Owing to their vast experience in conducting multidisciplinary studies comprising of field studies, laboratory investigations, analysis and advising on the remedial measures; authors have, over the decades, developed an expertise in providing cost effective and viable solutions for dam safety and rehabilitation. It is written with the aim to provide comprehensive information on various practical and theoretical aspects of Dam safety and rehabilitation. The four chapters covered the causes of distress, methods of investigations and finally effective rehabilitation techniques. It can be stated that timely adoption of appropriate 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 dam throughout its design life. It is perceived that this technical memorandum will be a great tool for practicing engineers, consultants in the field of irrigation projects to derive at optimum solution for problems related to Dam safety and rehabilitation.



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