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जल शक्ति मंत्रालय
जल संसाधन, नदी विकास और गंगा संरक्षण विभाग

Government of India
Ministry of Jal Shakti
Department of Water Resources,
River Development & Ganga Rejuvenation

Technical Memorandum on

REHABILITATION OF EARTHEN DAMS - A STUDY BASED APPROACH

by

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Dr. Tanusree Samanta, Scientist 'C'
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अनुसंधान के माध्यम से सेवा
केन्द्रीय जल और विद्युत अनुसंधान शाला, पुणे
CENTRAL WATER AND POWER RESEARCH STATION, PUNE
Estd. 1916
Service through Research

Dr. Prabhat Chandra
Director

केन्द्रीय जल और विद्युत अनुसंधान शाला, खडकवासला
CENTRAL WATER AND POWER RESEARCH STATION
June 2025

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JUNE 2025

FOREWORD

For more than 100 years, CWPRS has pioneered in providing services through research in water and power sector. During its journey, the institution has made incredible contributions in nation building and boasts of innumerable achievements. It has accomplished the milestone of being center of excellence in hydraulic engineering and research. The gamut of CWPRS services includes field studies, desk studies, physical modelling and mathematical model studies.

One of the service areas of CWPRS constitutes conducting specialized studies for assessing safety of dams and recommending rehabilitation measures. India today ranks third globally in number of large dams, following only the China and the USA. During the post-independence era, there was a major thrust in building large-scale dam projects in India, often referred to as the "Temples of modern India". These dams were a key component of the vision for modernizing India and boosting its economy through increased power generation, irrigation and flood control. Even today, after 77 years of independence, there is a dire need to build dams and storage reservoirs in India to tap the estimated 690 BCM of surface water, out of which only 257.81 BCM is being stored presently. As India embraces the Water Vision @2047, an initiative focused on ensuring water security to all Indian citizens by 2047; the importance of dams is even more accentuated.

The journey towards achieving this milestone remains blatantly challenging though. From finding suitable construction sites to overcoming socio-economic and environmental hurdles, new dam construction is an arduous task. On the other hand, existing dam infrastructure is aging, thereby posing challenges such as safety risk, sedimentation, environmental impacts and social issues. Many dams are old, built before climate change was a significant factor, and are now struggling to cope with extreme weather patterns. While the country today cannot afford to decommission old dams, there are increasing concerns about the safety and functionality of aging dams. The Government of India is hence implementing the Dam Rehabilitation and Improvement Project (DRIP) which aims to increase the safety of selected dams and to strengthen dam safety management in India.

Over the years, CWPRS has made significant contributions across India and in neighbouring countries like Afghanistan, Bhutan and Nepal by conducting advanced studies for safety assessment and rehabilitation of numerous dam projects of national and international importance. Assessing the need for rehabilitation and recommending suitable remedial measures *is a skilled task and requires a comprehensive study based approach. The present Technical Memorandum on "Rehabilitation of earthen dams – A study based approach" attempts to cover in great detail these aspects pertaining to earthen dams. The publication is expected to be a useful resource document for dam authorities and research staff at CWPRS for conducting future studies.*

PUNE
June 2025

Dr. Prabhat Chandra
Director

PREFACE

Embankment dam is the oldest and most common type of dam construction worldwide, as well as in India. Out of total 6628 specified existing dams in India, about 80% are embankment dams. An embankment dam is a barrier created to store water, primarily constructed from natural soil, rock and sometimes waste materials from mining. India has constructed significant embankment dams since independence viz. the Tehri (tallest dam in India, 260.5 m high), Pong, Salal, Hemavathy, Tungabhadra, Kulamavu, Bansagar, Tawa, Vaitarna, Gosi Khurd, Jayakwadi, Totladoh, Balimela, Kapur, Hirakud, Upper Indravati, Ranjit Sagar, Sri Komaravally Mallannasagar, Rajghat, Ram Ganga, Kangsabati Kumari, etc.

Journey of CWPRS in the field of geotechnical studies for embankment dams dates back to more than 70 years. Starting from limited investigatory works such as field and laboratory testing, borrow area survey, foundation investigations, QA/ QC testing of soil samples during initial days to conducting specialized studies for assessing static and seismic safety and seepage aspects of earthen, rockfill, tailings dams and ash dykes; the journey has been enriching with knowledge and experience. With the advent of time, study methodologies also advanced from conventional to numerical modeling using softwares.

Considerable experience has been attained in conducting the studies and recommending rehabilitation measures for stability and seepage mitigation in embankment dams. More than 100 studies have been conducted so far for various government agencies, PSUs and private firms. The major clientele includes dam authorities, power corporations, river management authorities, municipal corporations, irrigation departments and various other offices of state and central government.

In view of the above, a need was felt to document various aspects of geotechnical studies for embankment dams in the form of a Technical Memorandum. The memorandum is divided into ten chapters viz. (1) Introduction; (2) Site inspection and distresses, (3) Geotechnical investigations, (4) Seepage analysis, (5) Structural stability analysis, (6) Dynamic stability, (7) Rehabilitation measures, (8) Instrumentation and Monitoring and (9) Case studies and (10) Overall guidelines and recommendations.

Overall introduction of embankment dams is given in **Chapter 1**, covering broadly their advantages, limitations and the need for safety assessment. The stage wise study approach adopted by CWPRS for safety assessment of embankment dams is also presented in this chapter. **Chapter 2** describes in detail various types of failures in embankment dams. The significance of detailed site inspection visit and inferences drawn from site observations about condition of the dam and probable causes of distresses is also elaborated upon. **Chapter 3** describes various field and laboratory investigations required for geotechnical safety assessment of embankment dams. Seepage is an inevitable phenomenon in earthen dams due to the inherent porous nature of soil. Seepage also has direct implication on stability of the dam. The study methodology for conducting seepage analysis is presented in **Chapter 4**.

Chapter 5 talks about structural stability assessment of earthen dams and the need for implementation of rehabilitation measures. Earthquakes are a major cause of damage and failure in earthen dams. Various methods for assessing stability during seismic conditions are discussed in **Chapter 6.** Depending upon the results of seepage and static/ seismic stability studies, the need to implement rehabilitation measures is assessed. Various rehabilitation measures and their suitability depending upon site specific characteristics are discussed in **Chapter 7**. Monitoring and maintaining earthen dams involve a combination of visual inspections, instrumental measurements and maintenance practices to ensure structural integrity and safety. These aspects are elaborated in **Chapter 8**. Finally, few significant case studies for which elaborative studies were conducted by CWPRS are presented in **Chapter 9**. Overall recommendations and guidelines for earthen dams are given in **Chapter 10**.

Altogether, an attempt is made to cover relevant aspects of geotechnical safety and stability assessment of earthen dams. A basic understanding of theoretical concepts in soil mechanics may be necessary for comprehending the subject. Wherever possible the concepts are elaborated, however an attempt is made to keep the content generic in nature rather than covering technical aspects in great detail. Also, the focus of the memorandum is on geotechnical studies for earthen portion of a dam; aspects related to concrete/masonry overflow and non-overflow portions, spillway gates, conduits etc. are not covered. The memorandum is expected to be a useful reference document for dam authorities and research staff at CWPRS for future studies.

ACKNOWLEDGEMENT

First and the foremost the authors express a deep sense of gratitude to Dr. Prabhat Chandra, Director, CWPRS for his constant encouragement and support during preparation of this Technical Memorandum. His motivation helped in timely completion of the document in limited time period.

Special thanks are due to Shri Rizwan Ali, Scientist ‘E’ for his constructive suggestions and technical inputs which were instrumental in achieving the desired outcome. The authors are thankful to him for his valuable guidance.

Acknowledgement is extended to various government agencies, PSUs, private firms, dam authorities, power corporations, municipal corporations, irrigation departments for referring earthen dam projects to CWPRS and for extending cooperation in every possible way which enabled conducting studies. The authors are also immensely thankful to all retired and present research staff of Geotechnical Engineering Division who directly or indirectly contributed in the studies. The technical memorandum would not have been a reality without their valued contribution.

Last but not the least; authors are grateful to officers and staff of LIBIS division for extending help in providing technical references during the course of studies as well as during preparation of this technical memorandum and printing of the document.

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CHAPTER - 1

INTRODUCTION

J. S. Edlabadkar, Scientist ‘D’

1. GENERAL

Dams are monumental engineering structures that fulfil the essential need of water for various purposes such as domestic, agricultural, municipal and industrial use; fisheries and tourism development; hydro power generation and flood mitigation. Built from diverse materials such as concrete, masonry, earth fill and rockfill; dams are amongest the most complex structures built by mankind. With 6628 structures listed in the National Register of Specified Dams (2025), India ranks third globally in the number of large dams, following China and the USA. The gross storage capacity of specified dams in India accounts for 330.022 BCM; with 26 dams of height more than 100 m, 702 between height 30 m to 100 m, 3440 between height 15 m to 30 m and 2460 between height 10 m to 15 m. Maximum number of these dams i.e. 6407 are owned by State Government authorities while the remaining few are owned by Central Government, State/ Central PSUs and Private agencies. Maharashtra ranks topmost in the number of specified dams (2696), followed by Madhya Pradesh (1370) and Gujarat (525).

1.1. NEED FOR DAM SAFETY ASSESSMENT AND REHABILITATION

Dam failures in the history have demonstrated catastrophic impacts. ‘Failure’, in layman’s terms, is defined as total/ partial collapse or breach of a dam. Collapse or breach results in sudden release of water causing disproportionate flooding and losses to human habitats on downstream side. However, in engineering context; ‘failure’ of a structure is also termed as a damage that results in its compromised performance and functionality which is below a minimum anticipated level. There are 36 reported dam failures in India. The first dam failure was in Madhya Pradesh in 1917 when the Tigra dam failed due to overtopping. The worst dam disaster till date was that of Machu dam, Gujarat in 1979, in which about 2000 lives were lost. Other notable instances of dam failures in India include Kaddam (1957), Panshet/ Khadakwasla (1961), Chikkhole (1962), Nanak Sagar (1967), Kodaganar (1977), Pratappura (2005) and Tiware (2019). Lessons from these failures, time and again emphasize, albeit in a harsh way, the importance of appropriate design, construction and maintenance, as well as the need for robust emergency plans and effective communication.

Age wise distribution of dams in India (Fig. 1.1), hints to the fact that India’s dams are ageing; with over 2754 being more than 25 years old, 1390 between 50 to 100 years old and around 291 are even over a century old. Although, data indicates that most failures have occurred in dams aged 5 – 10 years; ageing does pose a potential risk to dam safety. Hence, ensuring safety of dams is of vital importance to save human habitats, protect national investments and to secure their wide-ranging benefits. In view of the ageing effect, safeguarding these structures by suitable rehabilitation measures becomes even more critical.

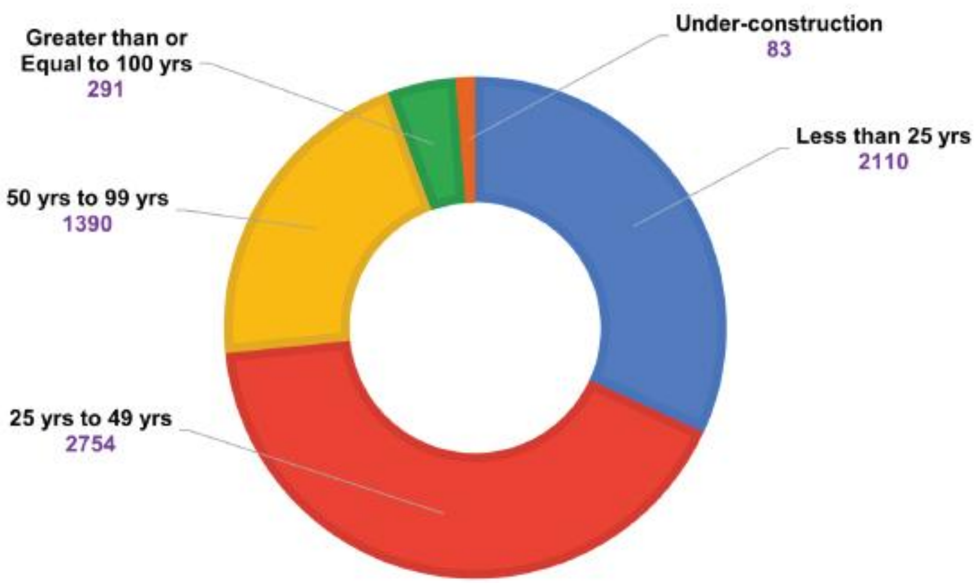


Figure 1.1 Age wise distribution of specified dams in India

(Source: National Register of Specified Dams, 2025)

Need for rehabilitation of a dam arises due to many reasons. Most of the dams were constructed during the period when technology, design principles and construction techniques were in a very primitive stage in India. In olden days only some information on the geology and riverbed rock properties was alone available. Even the basic hydrographs were not available for design or projection of flood. Also, the very concept of integrating various fields such as hydrology, geology, geomorphology, hydrogeology, structural mechanics, geotechnical engineering and hydraulics did not exist at the time of construction of some of the major existing dams. With the advent of emerging science and technology, design methodologies and construction techniques have advanced, based on which various dam authorities have developed state-of-art guidelines. Safety check of existing dams to assess their compliance with current standards, may sometimes necessitate their rehabilitation. Rehabilitation is also required due to damages such as foundation failure, seepage, heavy floods, earthquakes, etc. In addition, need for rehabilitation also arises when height of the dam is to be increased for augmentation of storage capacity or to account for revision in hydrological and climate change conditions.

The necessity of safeguarding dams has been further underscored by the Dam Safety Act of 2021, which is a significant step by the Government of India to address dam safety issues. The Act mandates a comprehensive framework to enhance safety of all specified dams in India. The Government of India is also implementing the Dam Rehabilitation and Improvement Project (DRIP) with financial assistance from the World Bank and Asian Infrastructure Investment Bank. The project aims to increase the safety of selected dams in participating states and to strengthen dam safety management in India.

CWPRS has made significant contributions across India and in neighbouring countries like Afghanistan, Bhutan and Nepal by conducting advanced studies for safety assessment and rehabilitation of numerous dam projects of national and international importance. The recent significant contributions include those for Salma dam, Afghanistan (Afghan India Friendship Dam); Kholongchhu dam, Punatsangchhu dam, Bhutan; Lower Seti Project, Nepal; Polavaram dam, Andhra Pradesh; Sardar Sarovar dam, Gujarat; Hirakud dam, Odisha; Bhakra dam, Himachal Pradesh; Indira Sagar dam, Madhya Pradesh; Temghar dam, Maharashtra; etc.

To avert dam failures, safety assessment of its existing condition is crucial which requires a specialized, multi-disciplinary approach involving scientific and technical principles of diverse fields including hydrology, hydrometeorology, structural, geotechnical, rock mechanics, seismology and various destructive and non-destructive investigatory techniques such as laboratory and field testing, geophysical and borehole logging methods. Using an array of investigatory, analytical, numerical and physical modeling techniques; CWPRS conducts robust safety assessments and delivers impactful recommendations that support safety and ongoing functionality of these critical infrastructures. This study-based approach is also essential to select optimum and most suitable rehabilitation measures. A number of study techniques and rehabilitation measures are available; however, selecting adequate investigatory technique, appropriate study methodology, identifying the causes of abnormality and implementing the most suitable rehabilitation measures is a challenging job, as every problem is unique in its own way due to site-specific characteristics. But effective application of a study based approach not only helps in identifying the source and cause of problem but also to evolve suitable rehabilitation measures for achieving dam safety.

1.2. EMBANKMENT DAMS

Earthen dams are the oldest and most common type of dam construction. Although the practice of construction of earthen dams is old, due to advancement in the field of soil mechanics, limitations in their construction have been overcome to a large extent. With progress in geotechnical engineering and with the use of sophisticated earth moving equipment, earthen/ embankment dams of significant heights can be built in recent times. Incidentally, the tallest dam today in India viz. the Tehri dam (260.5 m high) is a rock fill type embankment dam. Other significant embankment (earth fill and rockfill) dams in India include Pong, Himachal Pradesh; Salal, Jammu & Kashmir; Hemavathy, Tungabhadra, Karnataka; Kulamavu, Kerala; Bansagar, Madhya Pradesh; Tawa, Madhya Pradesh; Vaitarna, Gosi Khurd, Jayakwadi, Totladoh, Maharashtra; Balimela, Kapur, Hirakud, Podagada (Upper Indravati), Odisha; Ranjit Sagar, Punjab; Sri Komaravally Mallannasagar (*India’s largest artificial reservoir of 50 TMC*), Telangana; Rajghat, Uttar Pradesh; Ram Ganga, Tehri, Uttarakhand; Kangsabati Kumari, West Bengal, etc.

Figure 1.2 Tehri dam, Uttarakhand
(Height: 260 m, Length: 575 m)

Figure 1.3 Sri Komaravally Mallannasagar dam, Telangana

(Height: 59.6 m, Length: 22.6 km)

Figure 1.4 Kangsabati Kumari dam, West Bengal

(Height: 41.15 m, Length: 11.27 km)

As per ICOLD data, earthen and rockfill dams constitute about 81% of dams constructed worldwide. In India too, out of total 6628 specified dams, about 80% are embankment dams. Their share in dam failures is also quite significant. Out of 36 reported dam failures in India, 30 are those of earthen dams. The number of existing and failed earthen dams indicates the quantum of significance that should be given to their safety assessment in order to avoid catastrophic failures.

1.2.1. Components of earthen dam

A zoned cross-section of an earthen dam typically consists of following components: Hearting or core, Casing or shell, Drains and filters, Rocktoe, Pitching and Turfing.

a) **Hearting (core):** It is central part of the dam constructed by impervious soil. The main objective of provision of hearting is to prevent seepage through the dam.

b) **Casing (shell):** The core is surrounded by a layer of semi pervious material known as casing. Casing is constructed by rock, sand and/ or gravel. Casing gives stability to the core and also distributes the load over large areas of foundation.

c) **Cut-off-trench:** A cut-off trench constructed of impervious soil, generally similar to that of hearting; is constructed below the dam base to prevent seepage through foundation of the dam.

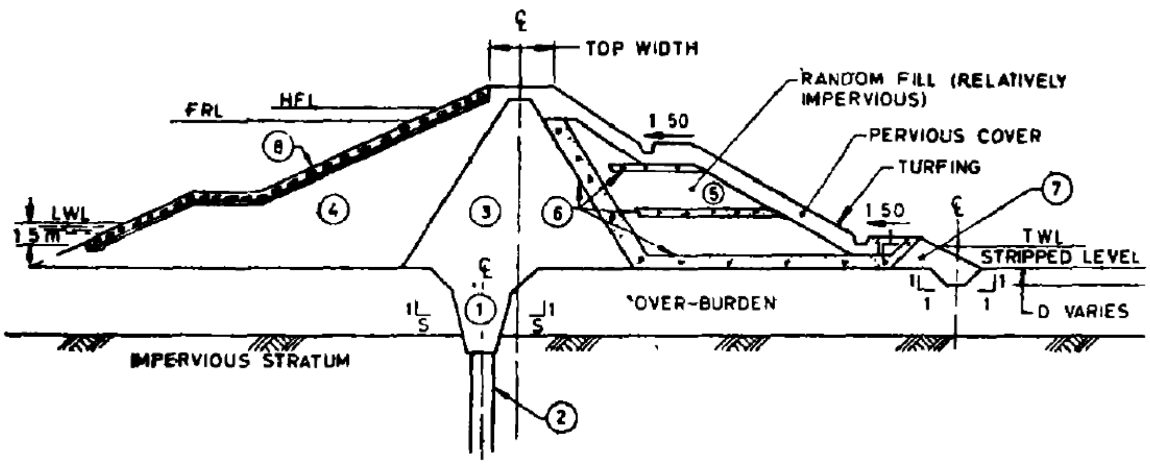
d) **Internal drains and filters:** To safely drain out water from the dam body, inclined and horizontal drains are provided inside the dam. The inclined filter intercepts the phreatic line and directs water towards the horizontal drain. The function of horizontal drain is to collect water from the inclined drain and drain out towards downstream toe of the dam. The drains also act as filters which are generally of intermediate gradation in between hearting and casing zones. Filters prevent movement of particles from core which is of finer gradation to the casing zone which is of coarser composition.

e) **Surface drains:** To safely drain out surface water over the dam slopes, a series of drains viz. longitudinal drains, cross drains and toe drains are constructed on downstream slope of the dam. The surface drains are provided to collect rainwater and prevent infiltration of water into the dam body.

f) **Rock toe:** Rock toe is constructed of small stones or rock pieces. Rock toe facilitates drainage of seepage water. Rock toe is provided to protect lower portion of downstream slope from erosion due to tail water.

g) **Pitching:** Stone pitching is provided on upstream slope of the dam. This layer prevents erosion of soil on slope by wave action, heavy rainfall, etc.

h) Turfing: Turfing is the layer of grass provided on downstream face of the dam. It prevents erosion of downstream slope during rainfall.



1. Positive cut-off
2. Grout curtain
3. Central impervious core
4. Upstream casing
5. Downstream casing
6. Inclined and horizontal filters
7. Rock toe and toe drain
8. Riprap with filter
9. Catch water drain

NOTE — Horizontal filters at intermediate levels are sometimes also placed in the upstream casing zone where casing material is of impervious nature.

Figure 1.5 Components of earthen dam

(Source: IS: 8826 - 1978)

1.2.2. Advantages and limitations of earthen dam

An earthen dam is constructed by placement and compaction of different types of soil/ rockfill material, in which the friction and interaction between particles bind them together into a stable mass, rather than by use of any cementing material. Construction of earthen dam is very popular and widely used in India because they can be built on any type of foundation using locally available soil material and their construction is comparatively economical. Gravity dam requires strong foundation, as it resists the exerted forces and overturning moments caused due to impounded water on upstream side, by its self-weight. In gravity dam, loads exerted on the dam structure are transferred to the foundation by cantilever action; as such a strong foundation is must for gravity dam, which is not the case with earthen dams.

However, soil being a naturally available complex material, earthen dams also have many limitations in terms of complexity in their design, construction, maintenance and susceptibility to seepage. Seepage is an inevitable phenomenon in earthen dams, which occurs due to the inherent porous nature of soil. Although, earthen dams are designed to withstand all possible destabilizing forces with a certain factor of safety, the possibility of their failure cannot be denied. In earthen dams, failure is indicated by presence of leakage, excessive seepage, wetness, slushiness or heave on downstream of dam, slip failures, undue settlement, presence of cracks, non-functioning of drains and relief wells, etc.

1.3. CWPRS CONTRIBUTION IN SAFETY ASSESSMENT AND REHABILITATION

Journey of CWPRS in the field of geotechnical studies for embankment dams dates back to more than 70 years. During initial days, the activities were restricted to conducting limited investigatory works such as field and laboratory testing, borrow area survey, foundation investigations, QA/ QC testing of soil samples, etc. Over the years, the scope diversified into conducting specialized studies for assessing static and seismic safety and seepage aspects of earthen/ rockfill dams, tailings dams, ash dykes and also for other structures such as barrages, hill slopes, embankments, mine slopes, river protection works, breakwaters, navigation channels, shore slopes, etc. These studies are conducted using analytical, empirical, limit equilibrium and numerical modeling techniques. Considerable experience has been attained in conducting the studies and recommending remedial measures for stability and seepage mitigation in embankment dams. More than 100 studies have been conducted so far for various government agencies, PSUs and private firms. The major clientele includes dam authorities, power corporations, river management authorities, municipal corporations, irrigation departments and various other offices of state and central government of India.

Brief areas of expertise in conducting geotechnical studies for earthen dams include:

(a) Static and seismic stability of earthen, rockfill and tailings dams, ash dykes
(b) Seepage potential for earthen dams, tailings/ ash dykes and barrages
(c) Seepage potential through foundation into underground powerhouse caverns
(d) Liquefaction potential of soils
(e) Stability and settlement analysis of dams, hill slopes, etc.
(f) Reservoir rim stability
(g) Ground improvement measures for weak foundations
(h) Guidance for geotechnical field investigations and estimation of engineering soil properties
(i) Confirmatory laboratory testing for determination of physical, hydraulic and engineering properties of soil samples

(j) Application of geosynthetics, recommending suitable geosynthetics and assessing stability of structures with geosynthetics, laboratory testing of geotextiles

1.3.1. Stage wise study approach

To assess geotechnical safety and seepage aspects of an earthen dam; a stage-wise study approach is adopted, comprising of following:

STAGE – I	:	Site inspection visit
STAGE – II	:	Review of available data
STAGE – III	:	Recommendations for additional investigations required if any
STAGE – IV	:	Finalization of design input parameters for studies
STAGE – V	:	Studies to assess seepage characteristics of existing dam
STAGE – VI	:	Studies to assess static and dynamic stability of existing dam
STAGE – VII	:	Studies for recommending suitable and optimized rehabilitation measures

Site visit gives a comprehensive understanding of existing dam conditions along with the extent and probable causes of distresses. During site visit, accessible regions of the dam site are inspected with project authorities. Extent of damage due to seepage, discharge quantity, location and emergence of seepage (through dam body, foundation or through abutment), evidence of piping, sand boils, settlement, subsidence, bulging of slopes, cracking, burrow holes, vegetation, etc. are assessed. Discussions are held with project authorities regarding construction practices followed; historical occurrences of incidents such as heavy floods, earthquakes, timeline of progression of distresses; remedial measures implemented earlier; etc. Further, the available data viz. design drawings, geotechnical investigation reports, etc. is collected from project authorities and reviewed. The adequacy of available data for conducting detailed studies is assessed. If required, additional investigations by drilling boreholes, trial pits including field and laboratory investigations are recommended. Occasionally, survey of the entire dam is recommended to be carried out to assess if there is any change in design and existing cross-sections of the dam. Geotechnical reports are studied to assess type of soil/ rock used for construction of the dam, as well as in foundation. Upon availability of complete data as detailed above, geometry of the dam and input parameters are finalized for conducting studies. Heterogeneity in the strata along entire length of the dam and existing distresses if any are accounted for in finalizing geometry and input parameters.

Seepage and stability of earthen dams are inherently linked. Seepage, which is movement of water through the dam and its foundation, significantly impacts the dam's stability by creating pore water pressure, reducing effective stresses and potentially causing erosion and piping. Understanding and controlling seepage is crucial for ensuring the long-term safety and functionality of earthen dams. In view of the above, geotechnical studies are conducted by numerical modeling methods to assess seepage characteristics for different operating conditions such as steady seepage, sudden drawdown, etc. The results of seepage analysis are integrated in limit equilibrium methods to determine dam stability for static conditions. Dynamic stability assessment is then carried out to check safety of the dam during earthquake conditions. Results of these studies give a fair idea about safety condition of existing dams. This forms the basis for further studies to determine the most suitable and optimized rehabilitation measures.

The stage-wise approach thus proves to be holistic in nature, comprehensively covering all geotechnical aspects including seepage as well as static and dynamic stability of earthen dams. Each of the above stages of study is covered in detail in subsequent chapters of the Technical Memorandum.

CHAPTER - 2

SITE INSPECTION AND DISTRESSES

Neeraj Mansingh Meena, Scientist ‘B’

2. GENERAL

Regular site inspections are fundamental not only to undertake detailed safety assessment studies but also for ensuring long-term safety and operational performance of embankment dams. Their primary purpose is the systematic identification of conditions, either subtle or obvious, those have already or could potentially compromise the dam’s structural integrity and functional capacity. While routine maintenance can address minor upkeep issues, formal inspections are critical for detecting potentially severe deficiencies. Early detection allows for timely investigation by qualified engineering professionals, preventing problems from escalating into major safety concerns or requiring expensive emergency repairs.

Through systematic visual examination, potential weaknesses such as developing cracks, signs of structural movement or settlement, problematic seepage patterns, erosion damage or inadequate slope protection, etc. can be identified. Vigilance during inspections is paramount, as seemingly minor anomalies can sometimes be surface indicators of major underlying issues like internal erosion or foundation instability. Promptly identifying, evaluating and addressing these concerns not only extend the dam's useful service life but also crucially prevents potentially catastrophic failures. Furthermore, inspections verify that essential components like spillways and outlet works are clear of obstructions, structurally sound and fully functional to pass flows as designed. The present chapter brings out significant observations that should be carried out during site inspection visit of an earthen dam and their likely implications.

2.1. TYPES OF DISTRESSESS AND PROBABLE CAUSES

Embankment dams can exhibit various signs of distresses. Understanding these manifestations and their likely underlying causes is essential for effective inspection and evaluation. These distresses are elaborated below.

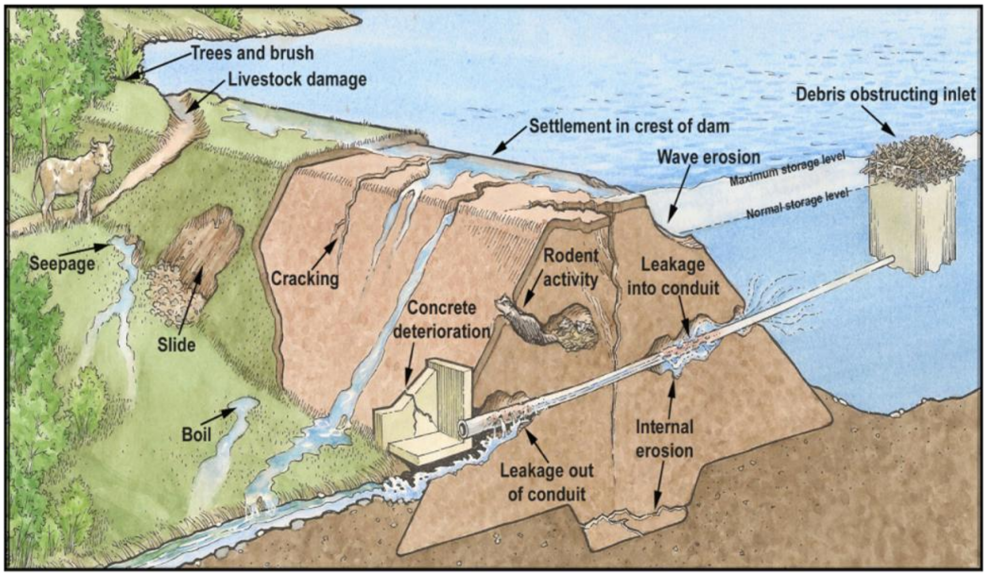


Figure 2.1 Various distresses in embankment dams

(Source: https://www2.gov.bc.ca/gov/content/environment/air-land-water/water/drought-flooding-dikes-dams/dam-safety/11965/12021/module-3-common-issues)

2.1.1. Cracks (longitudinal, transverse, desiccation)

Description: Physical breaks or fissures in soil surface on the crest or slopes. Longitudinal cracks run generally parallel to the dam's axis, potentially indicating slope instability or differential settlement along the length. Transverse cracks run perpendicular to the axis, posing a high risk if they extend below the water line as they can create direct seepage paths. Desiccation cracks are typically shallower, random, or of honeycomb patterns resulting from drying and shrinking of clayey soils.

Probable causes: Differential settlement between different zones of the embankment or between embankment and its foundation (especially over steep or irregular abutments); foundation movement or failure; inadequate compaction during construction; drying and shrinking of high-plasticity clay soils; structural stress concentrations leading to tensile failure.

2.1.2. Slides (slumps, slips, sloughs) and bulging

Description: Downward and outward movement of a mass of soil on upstream or downstream slopes. Slides can be shallow (sloughs involving only surface material) or deep-seated (involving larger mass and deeper failure surface). Bulging refers to an outward swelling or protrusion of the slope face, often visible near the toe, which can

Figure 2.2 Cracking in earthen dam

Figure 2.3 Slides in earthen dam

precede or accompany a slide. Slides are often identified by a distinct scarp (steep surface at top of the slide) and a bulge or accumulation of material at the toe.

Probable causes: Soil shear strength being exceeded by gravitational forces, often due to overly steep slopes; poor compaction during construction; saturation and weakening of embankment soils by seepage or prolonged rainfall; rapid drawdown of reservoir level removing the stabilizing effect of water column on upstream slope; foundation instability or weakening; erosion undercutting toe of the slope.

2.1.3. Settlement, depressions and low areas

Description: Localized areas on the crest or slopes that are noticeably lower than surrounding design grade. Crest settlement is particularly concerning as it reduces freeboard. Depressions on slopes can collect water.

Probable causes: Post-construction consolidation of embankment or foundation soil (some settlement is expected, but excessive or differential settlement is problematic); internal erosion (piping) removing soil from within, causing overlying material to collapse; lateral spreading of the embankment; decomposition of unsuitable buried materials (ex., wood); improper final grading after construction.

Figure 2.4 Indication of upstream berm settlement

2.1.4. Sinkholes

Description: Distinct, often steep-sided, localized depressions or holes forming suddenly on the surface.

Probable causes: Advanced internal erosion (piping) creating significant voids within the embankment or foundation, leading to surface collapse; collapse of undetected animal burrows or decaying root systems; erosion occurring around poorly sealed conduits or structures passing through the embankment.

2.1.5. Seepage (wet areas, flowing water, boils, turbidity)

Description: Water emerging on the downstream slope, beyond the toe, at abutment contacts, or around conduits. Manifestations range from soft, wet ground and lush vegetation to distinct flowing springs or "boils" (where seepage pressure lifts soil, creating a cone-like deposit). Turbid (muddy or cloudy) flow is a critical sign.

Probable causes: Natural permeability of embankment and foundation materials (controlled seepage through drains is expected); flaws or discontinuities in the dam's impervious core or cutoff trench; seepage pathways along cracks, root channels, or animal burrows; permeable layers within the foundation; poor compaction or cracking around conduits; improperly sealed embankment - abutment contacts.

Figure 2.5 Sand boils on downstream side

2.1.6. Erosion (rills, gullies, benching, scarps)

Description: The detachment and transport of soil particles from embankment surface by flowing water (rainfall runoff or wave action). Rills are small, shallow channels; while gullies are larger, deeper channels. Benching and scarps are features typically formed on the upstream slope by wave action eroding material at the waterline.

Probable causes: Inadequate or damaged slope protection (sparse vegetation, missing/ undersized riprap); concentrated surface runoff due to improper grading (especially on the crest or at groins); wave action against unprotected or inadequately protected upstream slopes; damage from livestock trails or vehicular traffic.

2.1.7. Inadequate slope protection

Description: Vegetative cover that is sparse, dead, or absent; riprap that is missing, displaced, weathered, undersized, or lacking a proper filter layer.

Probable causes: Poor initial design or installation; erosion of underlying filter or soil causing riprap settlement/ displacement; wave or ice action dislodging riprap; freeze-thaw weathering of non-durable rock; lack of maintenance (ex. not reseeding grass); erosion preventing vegetation establishment.

2.1.8. Inappropriate vegetation

Description: Presence of deep-rooted vegetation like trees and large shrubs on the embankment or near abutments; excessively tall grass obscuring visual inspection; large bare spots lacking cover.

Probable causes: Lack of vegetation management (mowing, clearing of woody growth); natural seeding by wind or animals; poor soil conditions inhibiting desired grass growth; traffic or animal damage killing vegetation. Tree roots can create seepage paths, and uprooting can breach the embankment.

2.1.9. Animal burrows

Description: Holes, entrances and associated tunnels excavated into the embankment crest or slopes.

Probable causes: Presence of burrowing animal species (ex. groundhogs, muskrats, beavers, crawfish) establishing dens or pathways within the embankment soil, often attracted by water proximity or suitable habitat like overgrown vegetation. Burrows can create seepage paths and weaken the embankment structure.

Figure 2.6 Vegetation on upstream slope

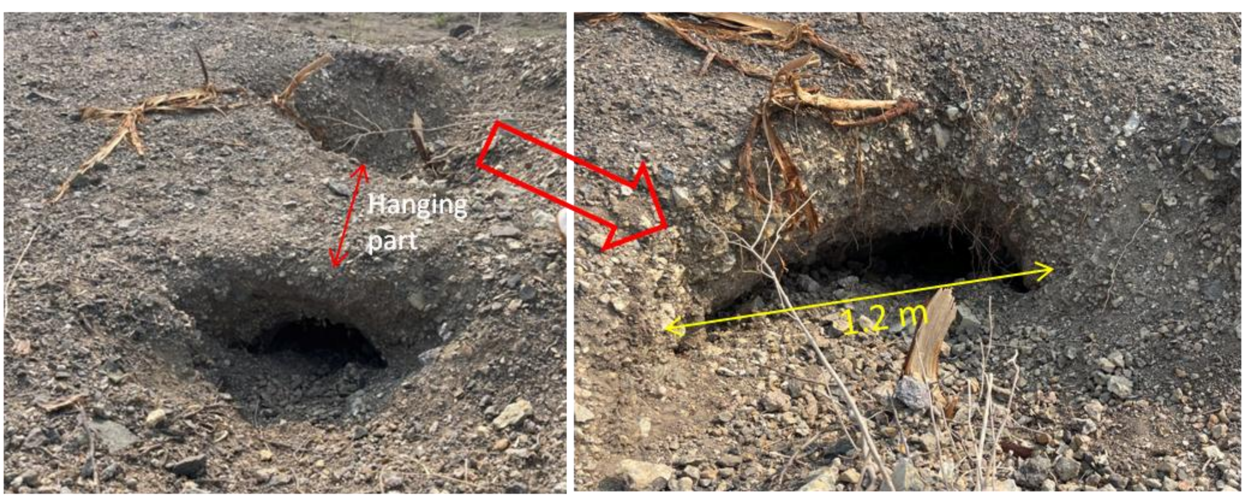


Figure 2.7 Animal burrows in dam body

2.1.10. Debris

Description: Accumulated logs, branches, trash, sediment or other foreign materials on embankment slopes, crest or near appurtenant structures (spillway inlets, outlets).

Probable causes: Inflow carrying debris into the reservoir which then washes onto slopes; wind action; lack of routine cleanup and removal by maintenance personnel. Debris can obscure inspection, damage slope protection and block flow structures.

2.2. CRITICAL OBSERVATIONS REQUIRING IMMEDIATE ATTENTION

While all distress signs warrant attention, certain observations demand immediate concern and action as they can signify rapidly developing, potentially catastrophic conditions:

- **Muddy/ turbid seepage**

  Inference: Active internal erosion (piping). Soil particles are being transported by seepage flow.

  Significance: This process can rapidly enlarge flow paths within the dam or foundation, leading to void formation, potential sinkhole development, structural instability, and ultimately breaching failure. Hence it requires immediate evaluation.

- **Boils**

  Inference: Concentrated exit point for piping/ foundation seepage. Significant upward seepage pressure is lifting and carrying away soil particles.

  Significance: Indicates a potentially unstable seepage condition that could rapidly worsen, undermining the dam's foundation or toe. Often directly linked to piping. Requires immediate evaluation and potential emergency stabilization by ring dykes.

- **Sinkholes (especially sudden or enlarging)**

  Inference: Significant subsurface void formation, likely caused by advanced piping, collapse of large animal burrows, intersecting seepage paths or failure around a conduit.

  Significance: Represents a direct pathway for failure progression, potentially leading to rapid collapse or breach. Requires immediate investigation and likely reservoir lowering.

- **Transverse cracks (especially below water level)**

  Inference: Potential direct seepage path through embankment, creates a shortcut for water flow under pressure

Significance: Highly dangerous condition that can quickly initiate concentrated seepage, leading to piping and rapid breaching failure, requires immediate evaluation and likely reservoir lowering.

- **Deep-seated slides or slides with significant scarps/ bulges**
  Inference: Major slope instability indicates failure along a deep shear surface within the embankment or foundation.

  Significance: Involves large volumes of material, significantly compromising the dam's structural integrity and potentially leading to overtopping if the crest is lowered or involved in the slide, requires immediate geotechnical evaluation and likely reservoir lowering.

- **Sudden unexplained drop in reservoir level**
  Inference: Significant new leak or developing breach, indicates large volume of water escaping the reservoir through an uncontrolled path.

  Significance: Signals a potentially rapid failure progression occurring within the dam, foundation or abutments, requires immediate, urgent investigation of the entire structure and downstream area.

- **Vortex in reservoir (near dam)**
  Inference: Rapid flow into a subsurface opening, water is being drawn quickly into a piping channel, conduit failure, or other breach point below the surface.

  Significance: Often an indicator of an advanced failure mechanism underway, signalling imminent danger of collapse or breaching, requires immediate emergency action and evacuation protocols.

- **Seepage emerging high on downstream slope**
  Inference: High Phreatic (saturation) line, indicates that saturation level within the dam is higher than desirable or expected.

  Significance: Reduces stability of downstream slope, increasing the risk of sloughing or slides, especially during prolonged high reservoir levels or rainfall. Warrants careful monitoring and potential investigation of internal drainage.

- **Sudden increase in seepage flow or drain/ well flow (uncorrelated with reservoir level)**

Inference: Worsening internal seepage condition, may indicate enlargement of existing seepage paths, development of new cracks, or initiation/ progression of piping.

Significance: Signals a potential deterioration of dam's internal integrity that needs prompt investigation to understand the cause and assess the risk.

These critical observations often necessitate immediate notification of dam safety officials, implementation of emergency action plans (including potential downstream evacuation warnings and reservoir lowering) and urgent investigation by qualified dam safety engineers. All above observations are also accounted for in undertaking studies for safety evaluation and deriving optimum rehabilitation measures.

2.3. CHECKLIST OF SITE OBSERVATIONS

A dam inspection checklist ensures systematic evaluation of all significant aspects of a dam without missing any critical parameters. An ideal checklist should cover various aspects of the dam's structures and their functionality, ensuring thorough assessment of potential issues. Key areas to inspect include the dam's embankment, spillways, outlets, gates/ valves, foundation and reservoir conditions. The checklist should also address aspects like seepage, erosion, vegetation and all other signs of distress or damage. Comprehensive dam inspection checklists and manuals have been prepared and published by CWC, DRIP, etc. The one followed by CWPRS for dam inspections while undertaking safety evaluation studies is given below:

2.3.1. Preparation and overview

- ✓ Review previous inspection reports, design drawings and site plans (note location of internal drains, relief wells, piezometers, past problem areas).
- ✓ Plan the inspection route systematically (ex. crest → upstream slope → downstream slope → groins → abutments → spillways → outlet → downstream channel → general areas).
- ✓ Note current and recent weather conditions (precipitation, temperature extremes) and reservoir level.
- ✓ Assemble necessary tools (camera, notebook/ forms, probe, measuring tape, level, sample containers, binoculars, safety gear).
- ✓ Periodically stop during the walk-over; view slopes and crest from various angles and distances to detect subtle irregularities in alignment or surface.

2.3.2. Dam crest

- ✓ Walk entire crest width using overlapping passes (zigzag or parallel).
- ✓ Check for cracks
  - Longitudinal (parallel to axis): Note location, length, width, depth, vertical offset, pattern (single, multiple), Monitor changes.
  - Transverse (perpendicular to axis): Note location, length, width, depth, Critical - investigate thoroughly.
  - Desiccation (drying pattern): Note extent, pattern, depth of major cracks. Distinguish from structural cracks.
- ✓ Check for settlement, depressions, low areas
  - Note location, approximate size, depth relative to design grade.
  - Sight along crest alignment using fixed points or level; check for sags or deviations.
  - Check for ponding water or evidence of poor surface drainage.
- ✓ Check for sinkholes (distinct, often steep-sided): Probe floor for voids, Note location, size, depth, shape. Critical - investigate immediately.
- ✓ Check for ruts (vehicles) or trails (animals): Note location, depth, extent, associated erosion.
- ✓ Check surface condition: Note erosion, bare spots, condition of surfacing material (gravel, pavement).
- ✓ Check for inappropriate vegetation: Note presence/ location of trees, shrubs, excessively tall grass (> 30 cm).

2.3.3. Upstream slope

- ✓ Walk entire slope using overlapping passes (zigzag or parallel), access may depend on reservoir level
- ✓ Inspect slope protection
  - Riprap: Check for displaced/ missing stones, undersized/ weathered rock, settlement/ depressions in riprap layer, erosion of underlying filter/ soil (look for exposed filter fabric or soil boils between rocks), vegetation growth within riprap.
  - Vegetative cover: Check for sparse/ bare areas, signs of erosion (rills, gullies), type and health of grass.
- ✓ Check for wave action damage: Note presence/ extent of benching (erosion forming a flat area) or scarps (steep cuts) at or near the waterline.

- ✓ Check for slides, slumps, bulges: Note location, size, presence of scarps or cracks above the feature, any associated seepage.
- ✓ Check for cracks (longitudinal, transverse, desiccation): Note type, location, dimensions.
- ✓ Check for depressions or sinkholes (more visible when reservoir is low).
- ✓ Check for animal burrows: Note location, size of openings.
- ✓ Check for debris accumulation: Note type and amount, especially near waterline or structures.
- ✓ Check waterline alignment against slope: Note irregularities suggesting settlement or slope movement.
- ✓ Inspect inlet structures (visible portions): Check for blockage, debris accumulation, structural damage.

2.3.4. Downstream slope

- ✓ Give extra scrutiny to this area, especially below normal pool level, as it is the primary indicator of seepage and stability issues.
- ✓ Walk entire slope using overlapping passes (zigzag or parallel).
- ✓ Check for seepage
    - Wet areas, damp spots, soggy ground ("trampoline effect" underfoot).
    - Areas of unusually lush, green, or different vegetation (ex. cattails, reeds, mosses), Note distinct lines of vegetation change.
    - Flowing water (springs, trickles, concentrated flows), Note specific location, estimate flow rate (ex. using bucket/ stopwatch if possible).
    - Check clarity: Observe if flow is clear or muddy/ turbid (cloudy), turbidity indicates piping - critical finding. Collect samples if monitoring change over time or for testing.
    - Sand boils: Note location (usually at/ beyond toe), size, height of cone, quantity of deposited material, clarity of flow from boil.
    - Seepage near toe drain outfalls or relief wells (indicates seepage bypassing or overwhelming drains).
- ✓ Check for slides, slumps, bulges: Note location, size, scarp height/ length, moisture content within slide mass, any associated seepage or cracks.
- ✓ Check for cracks (longitudinal, transverse, desiccation): Note type, location, dimensions.
- ✓ Check for erosion: Note rills, gullies (location, size, depth), sparse/ bare spots susceptible to erosion.

- ✓ Check for depressions or sinkholes: Note location, size, depth, Investigate cause.
- ✓ Check for animal burrows: Note location, size, density.
- ✓ Check for inappropriate vegetation: Note trees, shrubs, excessively tall grass (> 30 cm).
- ✓ Check toe drain outfalls: Locate all outfalls, Check for flow (presence/ absence). Measure flow rate if flowing, Check water clarity (turbidity), Note pipe condition (damage, blockage), Compare flow/ clarity with past records and corresponding reservoir level, Note drains that are unexpectedly dry or flowing excessively.
- ✓ Check relief wells (if present): Locate wells, Check for flow, Measure flow rate, Check water clarity, Compare with past records/ reservoir level, Note wells performing unusually.

2.3.5. Groins (embankment - abutment contacts)

- ✓ Walk entire length of both upstream and downstream groins (where embankment meets natural valley sides).
- ✓ Check for seepage (especially downstream groin): Note location, extent, flow rate, clarity, seepage here is common but needs monitoring.
- ✓ Check for erosion or gullies: Runoff often concentrates here; check for erosion damage.
- ✓ Check condition of any drainage channels/ linings within groins.
- ✓ Check for inappropriate vegetation concentrated in groin area.
- ✓ Check for animal burrows concentrated in groin area.

2.3.6. Abutments

- ✓ Traverse accessible portions of abutments adjacent to the dam.
- ✓ Look for signs of instability or slides in the natural abutment slopes that could affect the dam.
- ✓ Look for seepage emerging from abutment rock/ soil downstream of the embankment: Note location, flow, clarity, any staining (ex. iron), distinguish from embankment seepage if possible.
- ✓ Check for inappropriate vegetation near the embankment contact zone.

2.3.7. General considerations

- ✓ Check for debris: Note significant accumulations anywhere on dam slopes, crest or in reservoir near structures.

- ✓ Check for signs of vandalism or unauthorized access/ activity (ex. vehicle tracks on slopes, graffiti, damage to equipment).
- ✓ Document all findings: Use consistent terminology, record locations (stationing, GPS), dimensions (length, width, depth, offset), flow rates, clarity. Take clear, referenced photographs of all deficiencies and representative conditions. Use sketches to illustrate complex situations.
- ✓ Compare current observations with previous inspection reports and monitoring data (seepage flows, crack measurements, survey points). Note changes or trends.
- ✓ Note and prioritize any conditions requiring immediate attention, further investigation, or consultation with a qualified dam safety professional.

2.4. BASIC DATA COLLECTION DURING SITE VISIT

Before or during site visit, relevant background information and historical data is collected from project authority which is essential for a comprehensive inspection. This data provides context, aids in understanding the dam's design and performance history, and helps focus the inspection effort. Key data includes:

- **Dam cross-sections and longitudinal ('L') section:** These drawings show the dam's geometry, internal zoning (core, filters, shells), foundation levels and original ground profile. They are crucial for understanding the intended design and identifying deviations or areas prone to specific issues (ex. seepage through specific zones).
- **Survey data:** Historical survey data, especially of crest elevation benchmarks and alignment monuments, provides a baseline for detecting settlement, deformation, or movement over time. Comparing current observations to past surveys is vital for assessing changes.
- **Water level records:** Long-term records of reservoir levels, tail water levels, seepage measurements (from weirs, drains) and piezometer readings are critical. Correlating seepage or piezometric levels with reservoir elevations helps identify normal behaviour versus anomalies that might indicate developing problems like clogging or increased leakage.
- **Historical references:** This includes construction records (compaction data, material sources, foundation preparation), design reports, geological and geotechnical investigation reports, previous inspection reports and instrumentation data. These

documents provide insight into design intent, construction methods, known geological conditions, past performance and previously identified issues.

- **Remedial measures adopted earlier:** Details of any past repairs or modifications (ex. slope flattening, seepage control measures, crack repairs, vegetation removal) are important.
- **Details of foundation treatment:** Information on foundation preparation methods, such as excavation depth, grouting programs (locations, pressures, takes), dental concrete or other treatments used to control seepage and improve stability, is vital for understanding potential foundation seepage pathways.
- **Actual cut-off-trench (CoT) details:** Records of constructed depth, width, backfill materials and keying-in details of CoT are essential for evaluating its effectiveness in controlling under seepage.

Comprehensive dam inspection conducted as per above checklist and review of data mentioned above, create a strong ground for conducting further studies for dam safety assessment and rehabilitation measures. Correlation of site observations with project data can infer valuable insights of critical aspects that need to be considered in studies.

CHAPTER - 3

GEOTECHNICAL INVESTIGATIONS

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3. GENERAL

Comprehensive and systematic geotechnical investigation plays a critical role in safety and rehabilitation of earthen dams. Geotechnical safety of earthen dams involves careful assessment of several aspects such as: (i) seepage through dam body and foundation, (ii) slope stability under static and seismic conditions, (iii) differential settlements in dam body and foundation and (iv) liquefaction potential under cyclic loading. Evaluating these risks before new construction or during rehabilitation efforts is vital to prevent catastrophic failures. Geotechnical investigations are therefore indispensable, whether for safe design of new structures or to diagnose causes of distress in existing ones. Assessing safety of existing dams and selecting appropriate rehabilitation measures requires conducting detailed studies through a combination of analytical methods, limit equilibrium analysis, and advanced numerical modeling methods.

A key component of such studies is the accurate determination of soil properties in both embankment and foundation strata. Soil parameters such as grain size distribution, Atterberg limits, water content, dry density, shear strength (cohesion and angle of internal friction), compressibility characteristics, permeability, and dynamic response indicators (SPT N-values, CPT resistance) are fundamental inputs in safety analyses. These parameters can be derived from any of the following methods:

- In-situ tests (ex., SPT, CPT, vane shear, pressure meter),
- Laboratory tests (on disturbed and undisturbed soil samples),
- Geophysical surveys, and
- Empirical correlations or literature when direct measurements are limited.

However, reliance on assumptions or secondary sources like empirical correlations and literature may compromise the reliability of safety assessments. As soil properties are highly site-specific and heterogeneous, direct investigation through boreholes or trial pits at the dam site remains the most dependable approach for rehabilitation planning. This chapter outlines the key geotechnical investigation methodologies required for earthen dams and emphasizes the role of specific parameters in various analytical and modeling frameworks. It also highlights the sensitivity of certain soil parameters to different failure modes, thereby aiding in prioritization of field and lab testing during safety evaluation studies.

3.1. GEOTECHNICAL INVESTIGATIONS AT SITE

At site, geotechnical investigations typically include activities such as drilling boreholes, excavating trial pits, conducting in-situ tests (ex. Standard Penetration Test, Cone Penetration Test), and collecting disturbed and undisturbed soil samples for laboratory analysis. It is essential to consult a qualified geotechnical expert prior to awarding of any drilling or investigation contract, to ensure formulation of a detailed and technically sound investigation plan. This plan should clearly define the number and optimal locations of boreholes and trial pits, scope of in-situ and laboratory tests to be performed, number and depth of soil or rock samples to be extracted from each borehole, dimensions and layout of trial pits, and specific parameters required from the testing program. The data obtained helps dam authorities to assess various geotechnical aspects such as bearing capacity, settlement behavior, slope stability, seepage potential, and liquefaction risk, among other critical factors.

3.1.1. Soil exploration

In general, soil exploration methods can be broadly classified into three categories based on the level of direct access to sub-surface materials:

- Direct methods – These involve physical excavation such as test pits, trial pits, or trenches, allowing direct observation, sampling, and classification of soil strata.
- Semi-Direct methods – These include various types of borings (ex. auger, rotary, percussion), which provide access to deeper strata with limited disturbance and allow sampling for laboratory testing.
- Indirect methods – These consist of soundings or penetration tests (ex., SPT, CPT) and geophysical techniques (ex. seismic refraction, electrical resistivity), offering inferred information about subsurface conditions without physical extraction of materials.

In practice, a well-designed exploratory program may combine two or more of these methods to obtain comprehensive and reliable geotechnical data, tailored to the complexity and requirements of specific project.

a) Direct methods: Trial pits, typically excavated to a minimum size of 0.5 m × 0.5 m × 0.5 m, are used to investigate subsurface conditions either within the foundation soil or along the embankment section. These pits provide direct access for visual inspection and soil sampling. The locations of trial pits are usually selected along alignment of the dam in a staggered manner to ensure spatial coverage.

In each trial pit, in-situ density of soil is measured using standard procedures such as core cutter or sand replacement method. Disturbed soil samples collected from the pits are subjected to laboratory testing to determine a range of geotechnical parameters. These include in-situ moisture content, specific gravity, soil classification based on gradation and consistency limits as per BIS standards, and compaction characteristics such as maximum dry density (MDD) and optimum moisture content (OMC) using the standard Proctor test. Shear strength parameters, namely cohesion and angle of internal friction, are determined either by direct shear or triaxial testing methods. In addition, soil permeability is also determined.

For accurate laboratory assessment, reconstituted soil samples should be compacted to the field-measured in-situ density, thereby simulating actual site conditions. The dimensions of each trial pit are generally decided based on the volume of soil required for these laboratory tests, ensuring sufficient material is available for all intended tests.

b) Semi direct method: Making or drilling bore holes into the ground with a view to obtaining soil or rock samples from specified or known depths is called 'boring'. The common methods of advancing bore holes are:

- **Auger boring:** It involves the use of a soil auger, which may be hand-operated for shallow depths (up to 3–5 m) or power-driven for deeper borings. The auger is rotated and pressed into the soil, collecting the sample within its helical blades. It is mainly suited for soils that remain dry and stable without casing support. Once filled, the auger is withdrawn, and the soil sample is collected for inspection and classification.
- **Auger and shell boring**: If sides of the hole cannot remain unsupported, the soil is prevented from falling in by means of a pipe known as 'shell' or 'casing'. The casing is to be driven first and then the auger; whenever the casing is to be extended, the auger has to be withdrawn, this being an impediment to quick progress of the work. An equipment called 'boring rig' is employed for power-driven augers, which may be used up to 50 m depth (a hand rig may be sufficient for borings up to 25 m in depth). Casings may be used for sand or stiff clays. Soft rock or gravel can be broken by chisel bits attached to drill rods. Sand pumps are used in case of sandy soil.

- **Wash boring:** Wash Boring is widely used below the groundwater table, where auger methods are ineffective. This method is applicable to most soils except those with coarse gravel or boulders. After advancing the hole slightly with an auger, a casing pipe is installed, and a jetting arrangement is employed. Water is pumped under pressure through a hollow drill rod and bit, loosening soil at the base. The resulting soil-water slurry rises to the surface through annular space around the rod and is collected in a settling tank. Although this method does not yield undisturbed samples, it is effective for rapid borehole advancement. A change in wash watercolor or drilling rate can indicate changes in soil strata. When samples are needed, the jetting bit is replaced with a sampler.
- **Percussion drilling:** Percussion drilling (or churn drilling) uses a heavy drill bit that is repeatedly dropped to crush the underlying material. Water is added to form a slurry, which is periodically removed. This method is suitable for stiff soils or weathered rocks but is ineffective in loose sands and slow in soft clays. The soil is often highly disturbed by impact, limiting its usefulness for property evaluation.
- **Rotary drilling:** Rotary drilling is a fast and efficient method for rock formations. A rotating drill bit at the end of a drill rod cuts through the ground while drilling fluid or bentonite slurry is circulated to remove the cuttings. This method can retrieve rock cores when diamond-tipped bits are used. However, in highly porous soils, excessive fluid loss makes this method uneconomical.

c) Indirect methods: Soundings and penetration tests, such as the Standard Penetration Test (SPT) and Cone Penetration Test (CPT), are widely used. In addition, geophysical techniques such as seismic refraction, electrical resistivity tomography (ERT), and ground-penetrating radar (GPR) serve as powerful indirect tools for subsurface exploration. These methods are non-invasive and capable of covering large areas efficiently, offering insight into soil layering, rock depth, water table levels, and anomalies in subsurface materials. When used in combination with direct or semi-direct methods, soundings and geophysical investigations significantly enhance the accuracy and reliability of site characterization.

- **Standard Penetration Test (SPT):** The Standard Penetration Test (SPT) is a widely adopted in-situ dynamic penetration test for evaluating geotechnical properties of soil, especially cohesionless soils. It is standardized under IS 2131:1981 by the Bureau of Indian Standards and is performed in a borehole using a standard split spoon sampler. The borehole is advanced to the required test depth, and the sampler is driven into the soil using a 63.5 kg hammer falling freely from a height of 750 mm. The sampler is driven in three successive increments of 150 mm each. The number of blows required to drive the sampler through each increment is recorded.

Figure 3.1 Rotary drilling at dam site

The SPT N-value or blow count is defined as the sum of number of blows required to penetrate the second and third increments (i.e., the last 300 mm). Blow count of the first 150 mm is disregarded as it is considered a seating drive. The N-value obtained is an index that correlates with soil properties such as relative density, angle of internal friction, unconfined compressive strength, and bearing capacity. In saturated fine sands and silts, corrections for overburden pressure and dilatancy are applied to obtain a corrected N-value (N'). SPT is also a key tool in evaluating liquefaction potential and is used in earthquake-resistant design of earthen structures. The correlation for clays is rather unreliable. Hence, vane shear test is recommended for more reliable information. This test is useful to obtain undisturbed soil sample for further laboratory investigation.

Figure 3.2 SPT test at dam site **Figure 3.3 SPT split spoon sampler**

- **Cone Penetration Test (CPT):** The Cone Penetration Test (CPT) is a widely used in-situ testing method to determine the stratigraphy, strength, and deformation characteristics of subsoil. The test is standardized under IS 4968 (Part 3):1976 – "Method for Subsurface Sounding for Soils, Part 3: Static Cone Penetration Test". In this method, a steel cone of 60° apex angle with base area of 10 cm² is pushed into the ground at a constant rate of 1 to 2 cm/s without any rotation. The cone is connected to sounding rods and is advanced into the soil using hydraulic or mechanical equipment. Subsequently, a friction jacket attached with the sounding rod is pushed to obtained frictional resistance. The resistance to penetration is measured either through a mechanical system (proving ring and dial gauge) or via load sensors in electronic CPT setups. The total resistance (known as cone resistance) is recorded at regular depth intervals, typically every 20 cm. No soil sample is retrieved in this test though.

CPT is particularly useful for soils where sampling is difficult, such as soft clays and loose sands. It provides a continuous profile of soil resistance and can be correlated with various geotechnical parameters like undrained shear strength, relative density, friction angle, and soil classification. Modern CPT systems may also include pore pressure measurement (CPTu), seismic cones (CPT), and resistivity probes for advanced characterization. This test is non-destructive, rapid, and economical for profiling large sites and is especially useful in the preliminary investigation phase. However, it is not suitable in gravelly or very dense soils where cone advancement becomes impractical.

- **Vane Shear Test (VST)**
  The Vane Shear Test is a quick and simple method used to determine the undrained shear strength of soft to very soft cohesive soils. As per IS 4434:1978, the test can be performed either in laboratory on undisturbed soil samples or directly in the field. It involves inserting a four-bladed vane vertically into the soil and rotating it slowly to induce shear. The torque required to cause failure is measured and used to estimate the soil's undrained shear strength. This method is particularly useful for clays where sampling and handling for other strength tests may disturb the natural structure. The vane, typically with a height twice its diameter, is rotated at a controlled rate using a calibrated torque device. In the field, the test is carried out at required depths, and in the lab, soil is placed in a suitable container to preserve natural conditions. The Vane Shear Test is not suitable for sandy or fissured soils and assumes uniform soil properties. Despite its limitations, it is highly effective for preliminary site investigations, foundation assessments, and embankment stability analysis in soft clayey soils.

- **Geophysical methods:** Geophysical techniques offer non-invasive and efficient methods for assessing subsurface conditions, making them highly suitable for investigation and monitoring of earthen dams. These methods provide continuous spatial coverage and help detect subsurface anomalies, weak zones, seepage paths, and stratigraphic variations that are difficult to identify using conventional boring alone. Seismic Refraction method is widely used to determine the depth and profile of different subsurface layers based on the travel time of seismic waves. It is particularly effective in identifying the interface between overburden soils and bedrock, detecting loose or weak foundation zones beneath earthen dams, and estimating elastic moduli. The method is beneficial for mapping compaction quality and locating voids or zones of differential settlement. Electrical Resistivity Tomography (ERT) maps variations in subsurface resistivity by injecting electrical current into the ground and measuring potential differences. It is extremely sensitive to changes in moisture content and soil type, making it ideal for detecting internal erosion, seepage zones, and saturation fronts within embankments and foundations. ERT profiles help in delineating heterogeneities and identifying vulnerable zones requiring remediation. Ground Penetrating Radar (GPR) uses high-frequency electromagnetic waves to detect shallow subsurface features. It is particularly effective for near-surface inspections such as assessing surface layer uniformity, detecting cracks, and identifying zones of water ingress or material discontinuity within the dam body.

3.1.2. Soil sampling

The ultimate objective of various exploration methods described earlier is not only to gather relevant information about the subsurface strata but also to obtain representative soil samples for detailed laboratory analysis. The instruments specifically designed and employed for collecting such samples are referred to as soil samplers. Depending on the required sample type—disturbed or undisturbed—different samplers are used, each suited to specific soil conditions and depths. Disturbed samples may be further subdivided as: (i) Non-representative samples, and (ii) Representative samples. Non-representative samples consist of mixture of materials from various soil or rock strata or are samples from which some mineral constituents have been lost or got mixed up. While Representative samples contain all mineral constituents of the soil, but the structure of soil may be significantly disturbed. The water content may also have changed. They are suitable for identification and for determination of certain physical properties such as Atterberg limits and grain specific gravity. Undisturbed samples may be defined as those in which the material has been subjected to minimum disturbance so that the samples are suitable for strength tests and consolidation tests. Tube samples and chunk samples are considered to fall in this category.

Soil samplers are classified as ‘thick wall’ samplers and ‘thin wall’ samplers. Depending upon the mode of operation, samplers may be classified as the open drive sampler, stationary piston sampler and rotary sampler. The split spoon sampler is basically a thick-walled steel tube, split length wise. The sampler as standardized by the IS 2131:1986 - *Standard Penetration Test* *for soils* while thin-walled samplers are standardized by the IS 2132:1986 *Code of Practice for Thin walled Tube Sampling of Soils*.

3.2. GROUND WATER LEVEL

Identifying the location of groundwater table is a critical component of any geotechnical exploration programme. Typically, groundwater levels are recorded during exploratory borehole drilling. However, in cases where perched or artesian groundwater conditions are suspected—or when drilling fluids obscure natural groundwater levels—dedicated borings may be required solely for groundwater assessment. Accurate determination involves allowing water in the borehole to reach a state of equilibrium. In permeable soils like sands, stabilization occurs rapidly, often within few hours. In contrast, in fine-grained soils such as clays or silts, equilibrium may take several days or even weeks. To monitor water levels in such conditions, standpipes or piezometers are installed. A piezometer typically consists of an open-ended perforated tube (commonly around 50 mm in diameter), which is surrounded by a gravel filter and sealed above using puddle clay to isolate the measurement zone. Long-term observations using these instruments help in accurately determining stabilized groundwater levels. In low-permeability clays, where water movement is minimal, specialized pressure measuring devices may be required to assess groundwater pressure rather than water levels.

3.3. LABORATORY TESTING OF SOIL SAMPLES

Undisturbed and disturbed samples collected from the site are tested in laboratory to find out their physical, shear strength, consolidation, compaction and hydraulic properties.

3.3.1. Index soil properties

Physical properties of any soil sample basically represent its i) Natural water/ moisture content, ii) Density/ unit weight, iii) Specific gravity, iv) Particle size analysis, v) Consistency/ Atterberg limit (for fined grained soil). For determination of water content sealed samples collected from trial pit can be used whereas for density (bulk) determination undisturbed samples are needed. Specific gravity, grain size distribution and consistency of any soils sample can be determined using disturbed soil sample collected from trial pit or borehole. Soil gradation is classification of soil based on different particle sizes contained in soil. It is an important aspect of geotechnical engineering as it indicates other engineering properties of soil such as compressibility, shear strength and hydraulic conductivity. Soil can be classified into four categories viz. gravel (>4.75 mm), sand (4.75 to 0.075 mm), silt (0.075 to 0.002 mm) and clay (<0.002 mm) based on particle sizes. Based on the increasing value of water content, soil appears in four states viz. solid, semi-solid, plastic and liquid. The limiting values of water content between these states are known as Atterberg limits, and they are Liquid Limit (LL) and Plastic Limit (PL). The difference between (LL) and (PL) is Plastic Index (PI). The consistency of soil can be determined by comparing these values with natural water content. Gradation and Atterberg’s Limits are important in deciding suitability of soil to be used as construction material for earthen dams. Density of the soil sample is used as input parameter for various geotechnical analyses.

3.3.2. Compaction properties

Soil compaction occurs when soil particles are pressed together, reducing pore space between them. Heavily compacted soils contain less total pore volume and, consequently, a greater density. Compaction of soil increases its bearing capacity and shear strength. It also reduces water seepage and hence swelling/ contraction in earthen dams. By proper compaction of soil during construction of dams, one can prevent further settlement and thus stability of the dam is increased. At field, soil can be compacted using different techniques viz. vibration, impact, kneading, applying pressure, etc. Soil material can be compacted to maximum density (MDD) at a particular moisture content known as Optimum Moisture Content (OMC). The values of OMC and MDD are useful in deciding compaction water content for achieving maximum density while constructing earthen embankments.

In laboratory, compaction tests are carried out using disturbed soil collected from trial pit to decide the value of MDD and OMC. These values are determined in laboratory from compaction tests viz. Standard Proctor and Modified Proctor tests. In the Standard Proctor test, soil is compacted by a 2.6 kg hammer falling at a distance of 310 mm into a soil filled mould. The mould is filled with three layers of soil and each layer is subjected to 25 blows of hammer. The Modified Proctor test is identical to the Standard Proctor test except it employs a 4.89 kg rammer falling at a distance of 450 mm and uses five equals of soil instead of three. The experiment is carried out for soil samples with different moisture contents. Moisture content and dry density after compaction are calculated for each sample and plotted in graph. Maximum value of dry density (MDD) and corresponding moisture content (OMC) is obtained. Comparing the in-situ density of soil in an earthen dam with its maximum bulk density that can be determined from compaction test, it can be determined how well the embankment is compacted during construction.

3.3.3. Shear strength properties

Shear strength of soil is defined as the resistance to deformation by the action of shear stress. As soil fails under shearing, determination of shear strength properties of foundation/ embankment soil is essential to avoid failures (bearing failure or slope failure). The shear strength of soil is defined in terms of two parameters viz. cohesion (c) and angle of internal friction (ϕ) between soil particles. To assess safety of dams, value of cohesion and angle of frictions of constituent soils are used as input parameters. These parameters are determined by direct shear and triaxial shear tests.

- **Direct shear test:** Direct shear test is a comparatively quick process to determine shear strength properties of soil and most suitable for granular soil. To simulate drained condition tests are performed at slow strain rate whereas for undrained condition tests are to be performed at very fast rate. Generally disturbed samples reconstituted at bulk density are used for testing. In this test, soil is stressed to failure by moving one section of the specimen container (shear box) relative to the other section in horizontal direction under different normal loads. The soil sample fails along a predetermined horizontal failure plane. At first step a normal load is applied on specimen to consolidate it. In next step under a specific normal load shear force is applied at a constant rate until the sample is fully sheared (maximum 20% strain). The relationship between Normal stress and Shear stress at failure gives the failure envelope of soil and provides the shear strength parameters (cohesion and internal friction angle).
- **Triaxial shear test:** Triaxial shear test is the widely performed and usually recommended test for determination of strength properties of soil. It is advantageous over direct shear test are as:
  (i) Specimen can be failed along any failure plain i.e. failure plain is not pre-defined.
  (ii) Confining pressure can be applied to the sample to simulating actual site condition
  (iii) Drainage of the specimen can be controlled
  (iv) Pore pressure developed within the sample during undrained conditions can be measured.

Figure 3.4 Direct shear test equipment

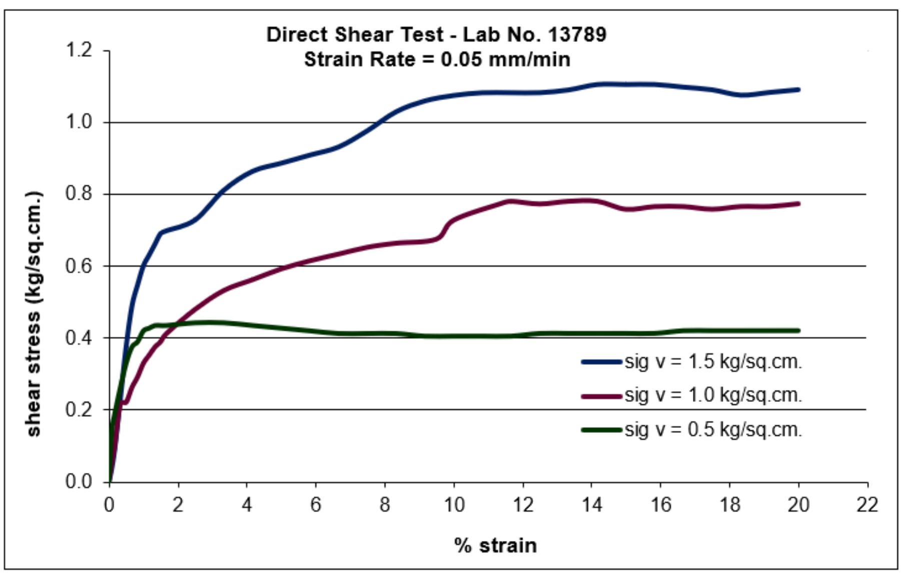


Figure 3.5 Direct shear test result – Shear stress vs. % Strain

In triaxial shear tests, a saturated cylindrical soil sample is sheared by applying vertical load up to failure under different values of confining pressures (to simulate stress condition at site) i.e. specimen is simultaneously subjected to lateral and axial forces. It is generally performed using undisturbed samples collected from boreholes. Test specimen generally has a height to diameter ratio of 2:1. However, in case of unavailability of undisturbed sample, reconstituted sample at bulk density can be used for testing. Three complete tests (on three different specimens) at three different confining pressures are needed to be performed to define a failure envelop and to get a set strength parameters i.e. cohesion and angle of friction of a soil sample.

Triaxial shear tests can be performed in drained or undrained conditions. The test is termed as drained test if drainage of water in soil samples is permitted during shearing. If water in the soil sample is not allowed to drain during sharing, the test is termed as undrained test. Also, depending upon whether the test is carried out on consolidated or unconsolidated soil sample, the tests are classified as UU (Unconsolidated Undrained), CU (Consolidated Undrained) or CD (Consolidated Drained). The choice of tests depends on the site conditions and studies to be performed with derived shear strength properties.

Unconsolidated Undrained (UU) Triaxial test

UU test approximates the behaviour of earth structure consisting of impervious soils during and immediately after construction. This test is also applicable to impervious foundation soil in which consolidation rate is slow as compared to the fill placement rate. During test the specimen is saturated initially. Immediately after saturation stage, sample is sheared under application of confining pressure and normal stress without allowing drainage of pore water.

Consolidated Undrained (CU) Triaxial test

CU test approximates the behaviour of impervious soil in embankment fill or foundation layer that have consolidated fully. In this test fully saturated sample is allowed to consolidate isotropically under chosen confining pressure. During consolidation stage, free drainage of water from sample is allowed. After achieving almost 90% consolidation, the sample is allowed to shear under deviator stress (difference between normal pressure and confining pressure) till failure without allowing any further drainage of pore water i.e. during shearing water content of the sample is kept constant which in turn increase the pore pressure of the sample. Shear strain rate should be chosen carefully (should be slow enough) to ensure uniform excess pore pressure distribution within the sample. During shearing axial load, deformation and excess pore pressure are measured. CU triaxial test provides the shear strength parameters for total stress analysis (c, ϕ) as well as effective stress analysis (c’, ϕ’) if provision for measurement of excess pore pressure is present in triaxial test equipment.

Consolidated Drained (CD) triaxial test

CD test is suitable for freely draining soils in which pore pressure do not develop which is usually a long term steady state condition in case of an earthen dam. In this test also, fully saturated sample is consolidated isotropically under chosen confining pressure allowing drainage of water. Then the sample is sheared slowly under deviator stress and during shearing drainage of pore water is allowed. The rate of shearing should be slow enough so that no excess pore pressure is developed within the sample during shearing. This means effective normal stress on soil sample during shearing is equal to the applied normal stress. CD test gives the effective shear strength properties of soil samples.

Strength parameters (c, ϕ) obtained from direct shear or triaxial tests are used as input for geotechnical analysis to assess slope stability, deformations and bearing capacity aspects of earthen dams. Also shear strength properties derived out of test conditions that closely simulate conditions at site are used for analysis. Shear strength properties obtained from CD test are generally used for stability of slope under steady seepage condition whereas CU properties are used for stability of slope under sudden drawdown conditions. Young’s modulus of soils can be determined from the stress-strain curve obtained in triaxial shear tests.

Figure 3.6 Triaxial shear test equipment

3.3.4. Consolidation properties

Consolidation of soil is a process in which the volume of a saturated soil decreases by expulsion of water under long term static load. When a load is applied in a low permeability soil, it is initially carried by the water that exists in pores of a saturated soil resulting in a rapid increase of pore water pressure. As the water begins escaping from the voids, the excess pore water pressure gets gradually dissipated and the load is shifted to the soil particles which increases effective stress on them, as a result the soil mass decreases in volume. The rate of escape of water depends on the permeability of the soil. The consolidation procedure lasts until the excess pore water pressure is dissipated.

An oedometer test (consolidation test) is carried out to determine the magnitude and rate of volume change of a laterally confined soil when subjected to different vertical pressures and different consolidation parameters viz. coefficient of consolidation (c_v), compression index (C_c), coefficient of volume change (m_v) and coefficient of compressibility (a_v) are found out from test results. The test can be performed on undisturbed or reconstituted samples.

The results from consolidation test are used to predict how soil in the field will deform in response to a change in applied load. Different consolidation parameters found out from test are used in geotechnical analysis for determining consolidation settlement characteristics of soil (clay layer) and to assess long term settlement of the structure. These parameters are also vital in designing ground improvement or accelerated consolidation techniques viz. Prefabricated Vertical Drains (PVDs), sand drains, etc. In addition, consolidation parameters can be used to calculated permeability value of clayey type of soil having very low permeability for which performing laboratory experiment is difficult.

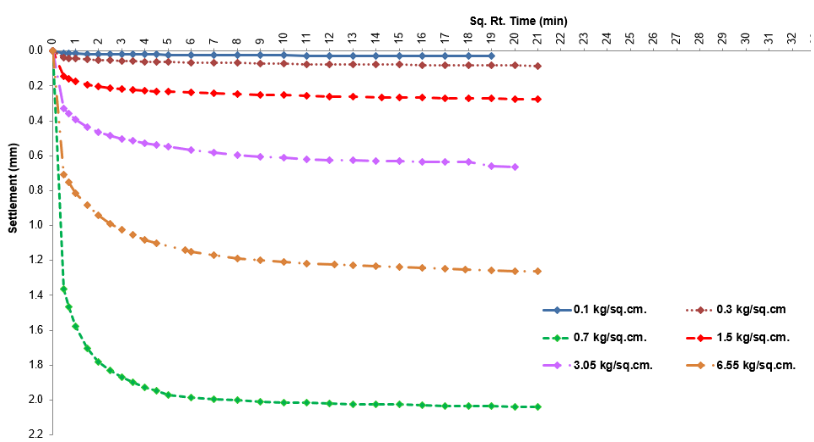


Figure 3.7 Consolidation test result – Settlement vs. Square root time

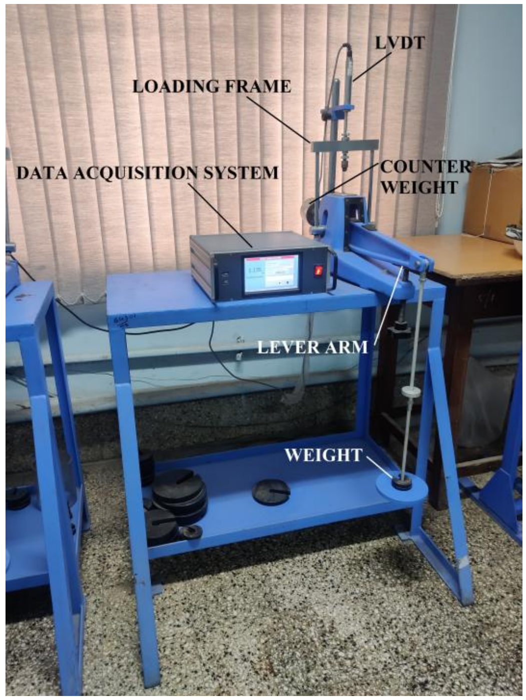


Figure 3.8 Consolidation test equipment

3.3.5. Hydraulic properties

The hydraulic conductivity or permeability of a soil is a measure of the soil's ability to transmit water through its interconnecting voids when subjected to a hydraulic gradient. Permeability test is performed in laboratory preferably using undisturbed soil samples. However, in case of unavailability of undisturbed samples, it can be performed with soil samples reconstituted either at the field density or at value of maximum dry density estimated from compaction tests. Depending on the type of soil samples, permeability tests at laboratory are conducted by constant head or falling head method to determine coefficient of permeability (k).

Coefficient of permeability of soil is an important parameter for seepage analysis through earthen dams and determines seepage discharge quantity through earthen embankments and foundation. Permeability values for soil in all zones of the embankment and in foundation strata are given as input for seepage analysis using numerical modeling. Also, calculation of uplift pressure, determination of rate of settlement of saturated compressible soil layer, etc. require a knowledge regarding the permeability value of soil.

3.4. NEED FOR WELL PLANNED GEOTECHNICAL INVESTIGATIONS

Different tests conducted at laboratory to determine properties of soil along with the relevant IS codes for laboratory testing are summarized in Table 3.1. The properties evaluated from these tests and their requirements in different geotechnical analysis are also listed.

Ensuring the safety and long-term performance of earthen dams, requires a well-planned and rigorous geotechnical investigation. These dams are subjected to varied loading conditions including hydraulic pressure, seismic events, seepage, and foundation settlement, which can lead to structural instability if not properly understood and addressed. A comprehensive geotechnical investigation serves as the backbone for designing effective rehabilitation strategies and mitigating potential failure mechanisms.

Through a combination of direct, semi-direct, and indirect methods—including borehole drilling, trial pits, in-situ testing (SPT, CPT, VST), laboratory testing, and geophysical surveys—dam authorities can develop a detailed understanding of subsurface conditions. These investigations yield critical soil and rock parameters such as shear strength, compressibility, permeability, and groundwater behavior, which are essential for analyses related to slope stability, seepage control, bearing capacity, and dynamic response of dams.

The accuracy and reliability of any geotechnical analysis are only as strong as the quality of the data collected. Hence, professional planning—starting from determining borehole locations to identifying relevant test types—is vital. Site-specific testing minimizes reliance on empirical assumptions and provides confidence in the selection of design parameters.

Furthermore, modern analytical tools such as limit equilibrium and finite element modeling require precise inputs. Soil parameters derived from field and lab tests feed into these models to simulate real-world conditions and predict performance under various scenarios. In the context of dam rehabilitation, this predictive capability is crucial for ensuring structural safety, guiding corrective measures, and prioritizing maintenance interventions. In summary, a robust geotechnical investigation is indispensable for safeguarding existing earthen dams. It ensures that rehabilitation works are informed, efficient, and durable.

Table 3.1 Laboratory tests on soil samples

Sr. No.	Parameter	Name of test	Properties	IS standard	Requirement in studies
1.	Physical Properties	Density/Unit weight	• Bulk density of soil sample		For all geotechnical stability studies
		Specific gravity	• Specific gravity of soil solids	IS-2720 (Part 2) - 1980	To calculate saturated density, dry density and degree of saturation of a soil sample
		Water/Moisture content	• Field water content of soil sample	IS-2720 (Part 3) - 1980	
		Grain size distribution	• % of Clay, silt, sand and gravel • Uniformity coefficient (C_u) • Coefficient of curvature (C_c) • Classification of coarse grained soils	IS-2720 (Part 4) - 1985	Decide suitability of soil to be used as construction material
		Atterberg's limits	• Liquid limit (LL) • Plastic limit (PL) • Plasticity index (I_p) • Shrinkage limit (SL) • Classification of fine grained soils	IS: 2720 (Part 5) - 1985	
2.	Compaction properties	Standard or Modified Proctor test	• Maximum dry density • Optimum moisture content	IS: 2720 (Part 7) – 1980 IS: 2720 (Part 8) - 1983	To estimate the degree of compaction
3.	Shear strength properties	Triaxial shear test (cohesive soil)	• Cohesion (c) • Angle of friction (ϕ)	IS:2720 (Part 11) – 1993 IS:2720 (Part 12) - 1981	➢ Slope stability ➢ Bearing capacity of any foundation

			• Young's modulus of elasticity (E) • Poison's ratio (μ)		➢ Immediate settlement of foundation upon loading
		Direct shear test (Non cohesive soil)	• Cohesion (c) • Angle of friction (ϕ)	IS:2720 (Part 13) - 1986	
4.	Consolidation properties	Oedometer test	• Initial void ratio (e_0) • Coefficient of consolidation (c_v) • Coefficient of compressibility (a_v) • Coefficient of volume change (m_v) • Compression Index (c_c)	IS:2720(Part 15) - 1986	➢ To estimate the amount of consolidation settlement and the time required for 90% consolidation
5.	Hydraulic properties	Constant head test Falling head test	• Coefficient of permeability (k)	IS:2720 (Part 17) - 1986	➢ For seepage analysis ➢ Uplift pressure

CHAPTER - 4

SEEPAGE STUDIES

J. S. Edlabadkar, Scientist 'D'

4. GENERAL

The passage of water through soil media is known as seepage. Seepage is an inevitable phenomenon in earthen dams, occurring due to hydraulic head difference on upstream and downstream sides and due to the inherent porous nature of soil. It is pertinent that seepage must be controlled in both velocity and quantity. If uncontrolled, seepage can progressively erode soil from the embankment or its foundation leading to internal erosion and piping; causing failure of earthen dams in almost 30–50% cases. Seepage significantly impacts the dam's stability by creating pore water pressure and thereby reducing effective strength of soil. A slope which becomes saturated and develops slides may be showing signs of excessive seepage pressure. Also, as the primary purpose of most earth dams is to store water during flood season and use it during lean season; excess water loss through seepage is undesirable.

Seepage through dam body can be controlled by selecting adequate soil material such as impervious clay in the hearting zone and compacting it to optimum density. Seepage through foundation is a major concern, especially for a dam which is sited over weak/ pervious foundation. Implementation of foundation seepage mitigation measures becomes utmost important in such cases. While different remedial measures viz. foundation grouting, cut-off-wall below COT, upstream cut-off-wall, upstream impervious horizontal blanket, etc. can be adopted to arrest foundation seepage; it is important to select and optimize the most effective site specific remedial measure in terms of seepage discharge reduction. At times, combination of more than one measure is required for obtaining desired results.

In view of the above, determining seepage characteristics of earthen dams for their existing conditions is important and often the first step in assessing a dam's safety and stability. Seepage analysis is conducted for an existing dam, irrespective of whether or not there are visible signs of seepage at site. Analyses using numerical modeling are a great tool to solve seepage problems and determine the optimum site specific solutions for seepage mitigation. The present chapter describes the basics of seepage mechanism through soil as porous medium and how seepage analysis of an earthen dam is conducted, mostly using numerical modeling techniques. The significance of different seepage parameters and their potential impact of dam stability are also discussed.

Figure 4.1 Excessive seepage on downstream berm of earthen dam

Figure 4.2 Seepage near downstream toe of earthen dam

4.1. BASICS OF SEEPAGE FLOW

Seepage in soil media is mathematically analyzed using Darcy's law, which describes the flow of water through porous media. It states that the flow rate of a fluid (q) is directly proportional to the pressure gradient (change in pressure over distance, Δp) and inversely proportional to the fluid's viscosity (μ). In simpler terms, the more pressure difference and the smaller the distance (L), the faster the fluid will flow, where permeability of soil (k) is a numerical constant. The soil particles, soil structure and water are assumed incompressible in Darcy's law.

$$q = -\frac{k}{\mu L}\Delta p$$ *Darcy’s Law*

Further, the flow of water through soil is governed by Laplace’s equation which is the fundamental equation used to describe steady-state seepage flow through saturated, isotropic and homogeneous soil. It essentially combines Darcy's law and the continuity equation to model the movement of water within a porous medium. Solving Laplace's equation allows engineers to determine various seepage parameters such as flow rate, discharge quantity, hydraulic head, pore pressures, exit gradient, uplift pressure and potential issues like piping or heaving in soil structures.

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial x^2} = 0$$ *Laplace’s Equation*

4.2. PROBLEM DEFINITION AND BOUNDARY CONDITIONS

To solve seepage problems, saturated soil mass considered in the analysis must be defined by: (i) boundaries, (ii) permeability and (iii) external hydraulic heads imposed upon the structure. The nature and location of these boundaries are determined by soil exploration program, assumptions based on engineering judgment and conditions imposed by the proposed design. Generally, seepage analysis problems associated with dams involve four possible types of boundaries viz. (i) impervious boundary, (ii) entrance and exit, (iii) surface of seepage and (iv) line of seepage as shown in Fig. 4.3. These conditions are defined below

(i) Impervious boundary - The interface between saturated, pervious soil mass and adjacent materials such as very low permeability soil or concrete is approximated as an impervious boundary. It is assumed that no flow takes place across this interface.

(ii) Entrances and exits - The lines defining area where water enters or leaves the pervious soil mass are known as entrances or exits, respectively. Entrances and exits are also called reservoir boundaries.

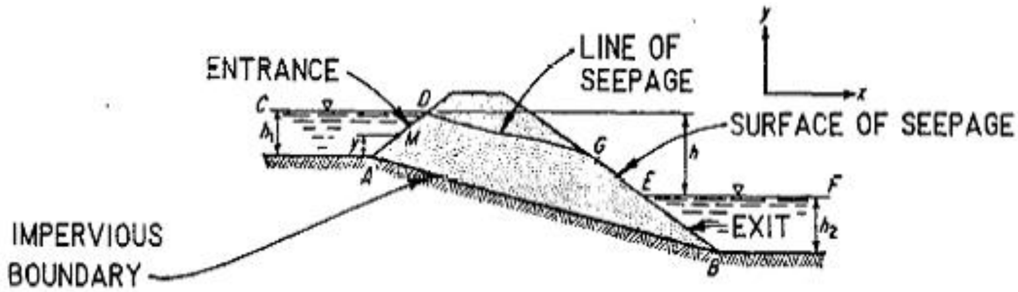


Figure 4.3 Unconfined flow through earthen dam

(iii) Surface of seepage - The saturated pervious soil mass may have a boundary exposed to the atmosphere and allow water to escape along this boundary. Pressure along this surface is atmospheric.

(iv) Line of seepage - Line of seepage, also known as the free surface, is located within the pervious soil where water is at atmospheric pressure (Fig. 4.3). The line of seepage is not known until flow distribution within the pervious soil is known.

4.3. METHODS FOR SOLUTION OF LAPLACE’S EQUATION

After definition of boundary conditions, solutions to steady-state, laminar flow seepage problems are obtained by solving the Laplace's equation to determine desired parameters. Several methods have been developed to solve exactly or approximately the Laplace's equation. In earlier days sand models, electrical analogy methods, viscous flow methods were used to determine flow conditions. Analytical methods and graphical methods were also developed and used to solve seepage problems. Several methods have been developed by various researchers. Harr (1962) made use of transformations and mapping to transfer the geometry of a seepage problem from one complex plane to another. Pavlovsky (1936, 1956) developed an approximate method called the Method of Fragments. Closed form solutions exist for simpler seepage conditions. However, these methods are complex and cumbersome. The flow net method is most widely used method that can be adapted to many of the seepage problems found in earthen dams and other hydraulic structures. In the flow net method, Laplace’s equation is represented by sets of two mutually perpendicular curves viz. flow lines and equipotential lines. However, this method is also time consuming since flow net is to be drawn graphically by trial and error method. Numerical methods, such as Finite Difference Methods (FDM) and Finite Element Methods (FEM) are today widely used to solve Laplace’s equation. With the advent of computers, numerical methods have gained popularity due to its ease of adapting to complex flow situations and site conditions. Several computer programs solving seepage problems by numerical modeling are available commercially. In CWPRS, softwares PLAXIS and FLAC are used to tackle seepage problems in earthen dams. This note briefly describes the numerical methods applied for solving seepage problems in earthen dams.

4.3.1. Numerical methods

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

a) Finite difference method

This method solves the Laplace’s equations by approximating them with a set of linear algebraic equations. The flow region is divided into a discrete rectangular grid with nodal points which are assigned values of head (known head values along fixed head boundaries or points, estimated heads for nodal points that do not have initially known head values). Using Darcy's law and the assumption that the head at a given node is the average of the surrounding nodes, a set of ‘N’ linear algebraic equations with ‘N’ unknown values of head are developed (‘N’ equals to number of nodes). Simple grids with few nodes can be solved by hand. Normally, ‘N’ is large and relaxation methods involving iterations and the use of a computer must be applied. Itasca's FLAC (Fast Lagrangian Analysis of Continua) software is a powerful FDM tool for simulating seepage in earthen dam problems. It utilizes the finite difference method to model the behavior of soil and rock masses, allowing for the simulation of steady-state and transient fluid flow, including free surfaces.

b) Finite element method

This method is an alternate method of numerical solution and is also based on grid pattern (not necessarily rectangular) which divides the flow region into discrete elements and provides ‘N’ equations with ‘N’ unknowns. Material properties, such as permeability, are specified for each element and boundary conditions (heads and flow rates) are set. A system of equations is solved to compute heads at nodes and flows in the elements. PLAXIS software, is a finite element analysis tool, widely used in geotechnical engineering to model and analyze various aspects of subsurface environments, including groundwater seepage. It allows users to simulate groundwater flow in both saturated and unsaturated soils, which can be further used to assess the stability of structures.

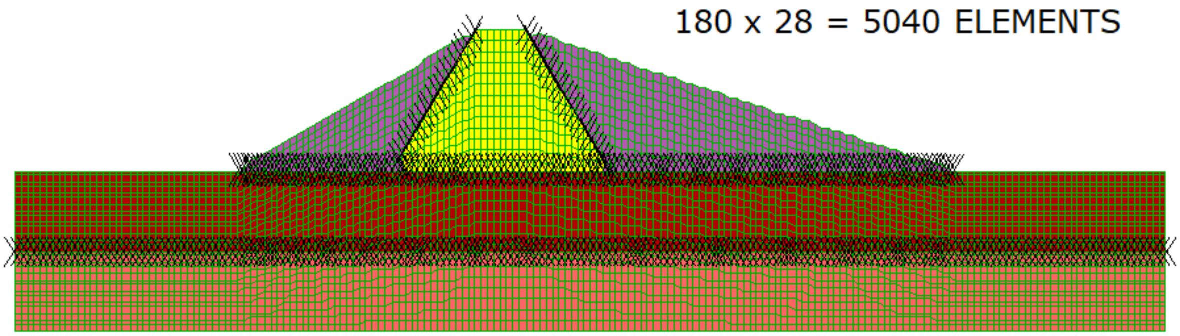


Figure 4.4 Discretization of geometry in finite difference method (Software FLAC)

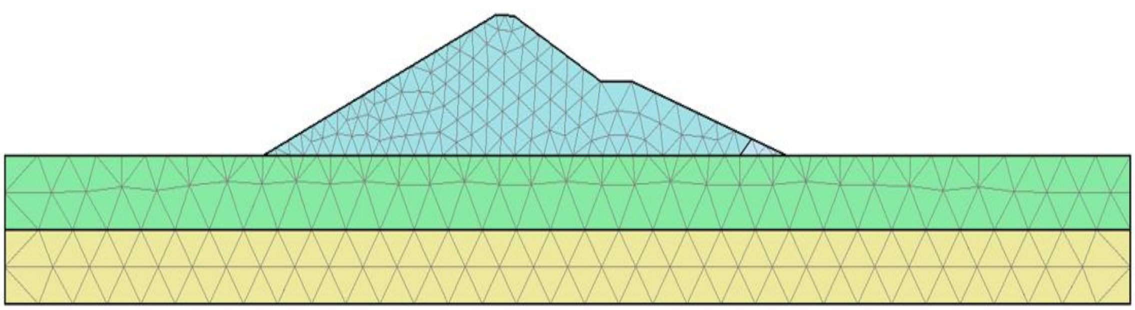


Figure 4.5 Discretization of geometry in finite element method (Software PLAXIS)

4.4. SEEPAGE PARAMETERS AND THEIR IMPLICATIONS ON DAM STABILITY

The primary aim of seepage analysis of an earthen dam is to determine characteristics and values of various seepage parameters such as phreatic line, pore pressure, hydraulic head, discharge quantity, exit gradient, etc. Each of these parameters and their values in critical zones of the dam cross-section have serious implications on safety of the dam. The significance of these parameters is elaborated below.

4.4.1. Phreatic line

For a safe dam section, it is desired that the phreatic line should drop up to dam base level in hearting zone and emerge out on downstream side through horizontal filter; indicating that the downstream zone is not in saturated condition. Contrary to this situation, an elevated phreatic line (one which does not drop up to dam base level in hearting zone) indicates that soil in downstream casing zone is in saturated condition inducing pore pressures in that region. Increased pore pressure reduces effective shear strength of soil, thereby affecting stability of downstream slope.

Fig. 4.6 illustrates results of numerical simulation of seepage for steady state condition for an earthen dam. The results show an elevated phreatic surface intercepting downstream slope near top berm. This indicates an unsafe situation and probable threat to safety of downstream slope of the dam. The results of seepage analysis can be correlated with site observations. For example in the above case, if the phreatic line is seen to be intercepting the downstream slope, some visible indications such as accumulation of water, dampness, slushiness at top berm or on the slope are likely to be observed at site. During site visit of the dam, these indications were clearly visible. Water was seen to be accumulated on top downstream berm of the dam (Fig. 4.7).

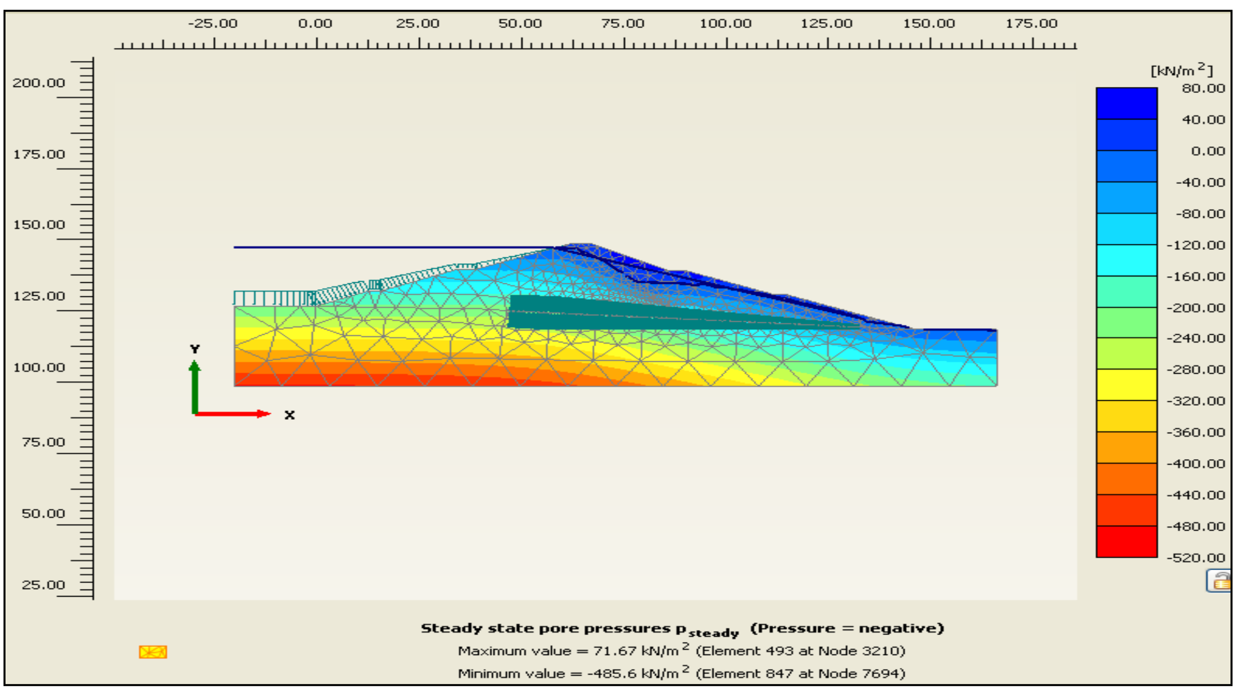


Figure 4.6 Seepage analysis by software PLAXIS (elevated phreatic surface)

Figure 4.7 Seepage water accumulated on top berm at EL 130.5 m

4.4.2. Pore pressure

Pore pressure is one of the significant parameters that affects stability of the dam. Increasing pore pressure reduces effective strength and hence weakens the soil, making it more susceptible to failure. High pore pressures can lead to problems like piping, where soil particles are carried away by seepage water, creating channels. In essence, seepage influences pore pressure and consequently the effective strength of soil. Understanding this relationship is crucial for stable geotechnical design and performance review of an earthen dam. Fig. 4.8 shows pore pressure distribution in a dam cross-section for steady state seepage condition, obtained from numerical model studies. The soil zone below phreatic line is in saturated condition thus giving rise to maximum pore pressure, the value of which depends upon hydraulic head at each nodal point. Further, stability of the dam in terms of factor of safety is determined with respect to this pore pressure distribution. If the factor of safety is within permissible limits (1.5 and 1.3 for steady state and sudden drawdown conditions respectively), the dam is inferred to be safe.

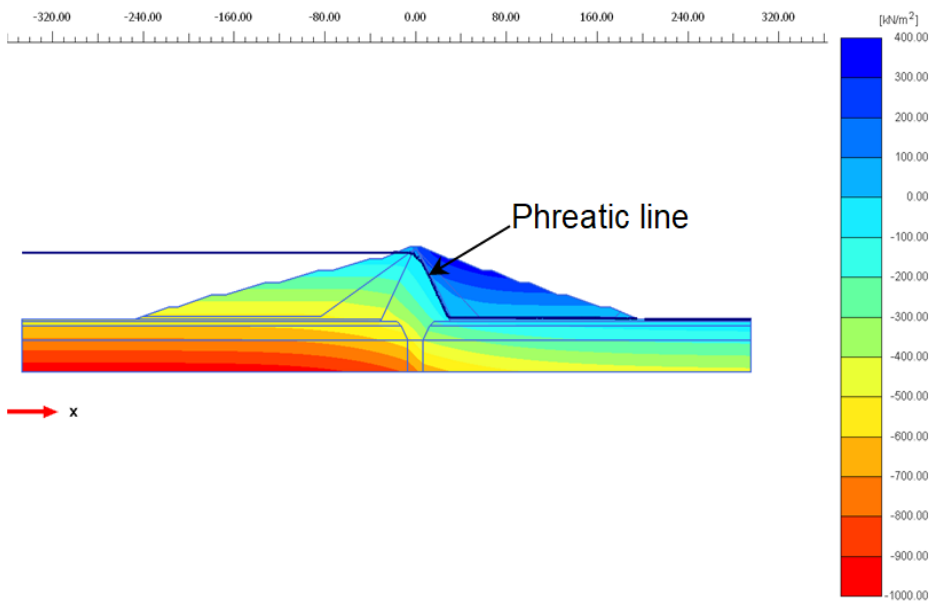


Figure 4.8 Pore pressure contours for steady state condition in an earthen dam

Regular monitoring of pore pressure values at site by installing piezometers at critical locations is crucial. Pore pressure readings from piezometers should be monitored periodically and compared with design values at respective locations for similar conditions, to check for any deviation. Any change in values should be noted, cause of the same should be found out and acted upon as per requirement. Also, change in pore pressure with respect to change in reservoir water level should be monitored.

Pore pressure in sudden drawdown condition

Sudden drawdown is one of the most critical operating conditions for an earthen dam. The sudden drawdown condition refers to rapid lowering of reservoir water level, which can significantly impact stability of the upstream slope. This occurs when the reservoir water level drops at a fast rate, thus disrupting the external hydrostatic pressure stabilizing the slope. The drawdown rate is faster than the rate of dissipation of pore pressure in the soil mass and hence the unbalanced pore pressure leads to potential destabilizing effect causing slope failure. This transient effect of lowering the reservoir water level, with dissipation of pore pressure depending upon soil permeability, can be well modeled by numerical modeling methods.

Fig. 4.9 shows the phreatic line at equilibrium condition for a dam subjected to different drawdown rates obtained from finite element seepage analysis using software PLAXIS. As can be inferred from the figure, the upstream dam slope is safest with lowest position of phreatic line which is corresponding to the slowest drawdown rate of 370 days. As the rate of drawdown increases to maximum (0.074 day) the phreatic line gets elevated. The position of phreatic line on upstream side denotes the extent of dissipation of pore water pressure in the upstream zone. Elevated phreatic line denotes that maximum portion of the upstream zone is in saturated condition and thus under high pore pressure, indicating a critical condition. As against this, a lower phreatic line indicates a comparatively safer condition with maximum dissipation of pore pressure in upstream zone of the dam. Numerical modeling thus provides a great tool to model the transient sudden drawdown condition and its consequent effect on dam stability, depending upon variable factors such as the rate of drawdown. These studies also help in determining the optimum rate of drawdown for a dam, in which the upstream slope is vulnerable to failure.

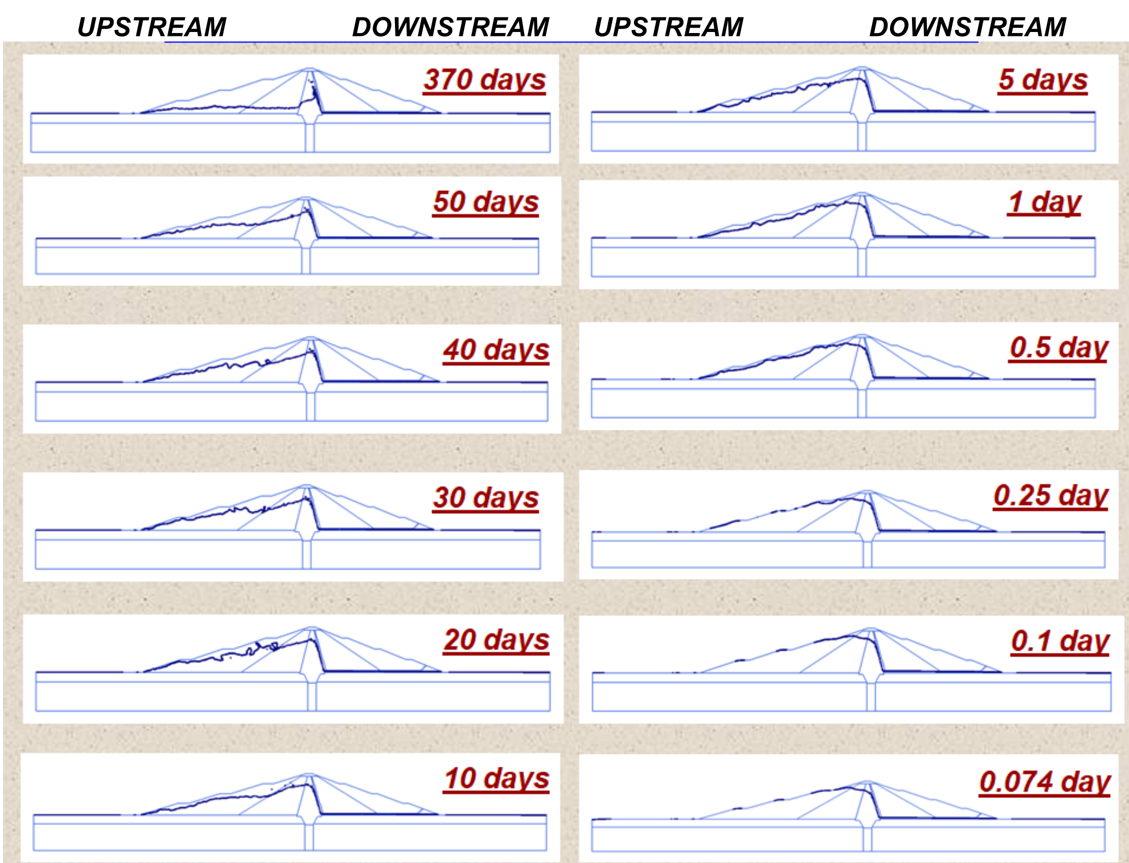


Figure 4.9 Upstream phreatic line for different drawdown rates (slow to fast)

4.4.3. Hydraulic head and exit gradient

High hydraulic head induces high pore pressure, thereby reducing effective strength of soil. The value of hydraulic head is of particular importance at certain critical locations of an earthen dam such as at exit points near downstream toe. The escape or exit gradient, is the rate of dissipation of head per unit length in the area where seepage is exiting the porous soil media. High hydraulic head in the exit area, such as downstream toe leads to high exit gradient which can give rise to undesirable conditions such as piping, sand boils, etc.; which can potentially destabilize the dam. Thus, the hydraulic head at these critical locations should be determined from results of seepage analysis and should be further compared with permissible values.

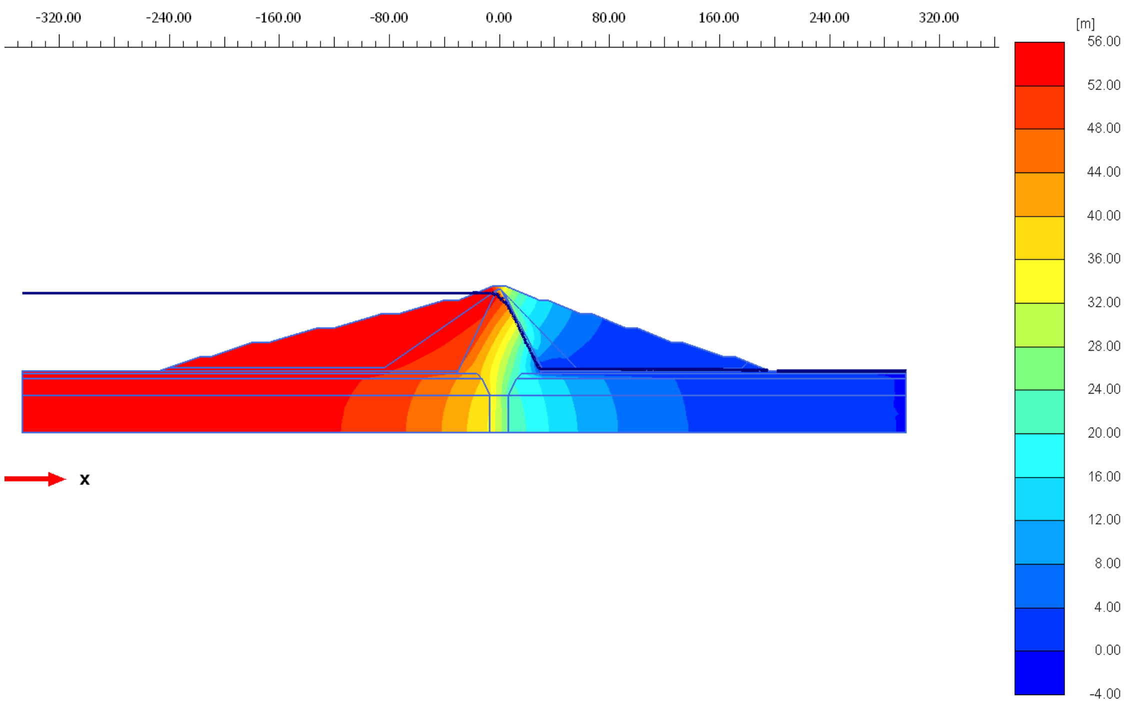


Figure 4.10 Hydraulic head contours (steady seepage condition)

4.4.4. Seepage quantity

Quantity of seepage discharge obtained from numerical modeling gives a fair idea about the quantum of water flowing through the dam body as well as its foundation due to seepage, based on which the need for seepage mitigation measures can be assessed and appropriate remedial measures can be selected. It is however, difficult to correlate the seepage discharge values obtained from analysis with site measurements due to several reasons. The first and the foremost being that numerical models are always a rudimentary representation of site conditions and seldom its exact duplication. Secondly, so many assumptions are involved in developing a numerical model, right from geometry creation to selection of design input parameters. Also, there are limitations in modeling the in-homogeneities and structural deficiencies of soil strata in the numerical model. Measurement of actual seepage discharge at site can also be a challenging task as it requires well maintained network of surface drains, toe drains, etc. and installation of appropriate discharge measuring devices. It is important that all seepage discharge at sites should be collected at the locations of measurement devices for accurate measurements.

However, notwithstanding above limitations; seepage analysis by numerical modeling does give a fair idea about the quantum of seepage, its distribution (through dam body or foundation), its concentration in specific areas and also its potential implications on dam safety. Although there are no standard values for permissible seepage limits through earthen dams, every dam authority should decide upon the value, on case to case basis considering practical and financial aspects. However, in no case the seepage limits should be so high such that safety of the dam is compromised. In case the limiting seepage discharge values are not available, CWPRS considers the values given in Table 4.1 as reference for assessing the need and deriving optimum seepage mitigation measures.

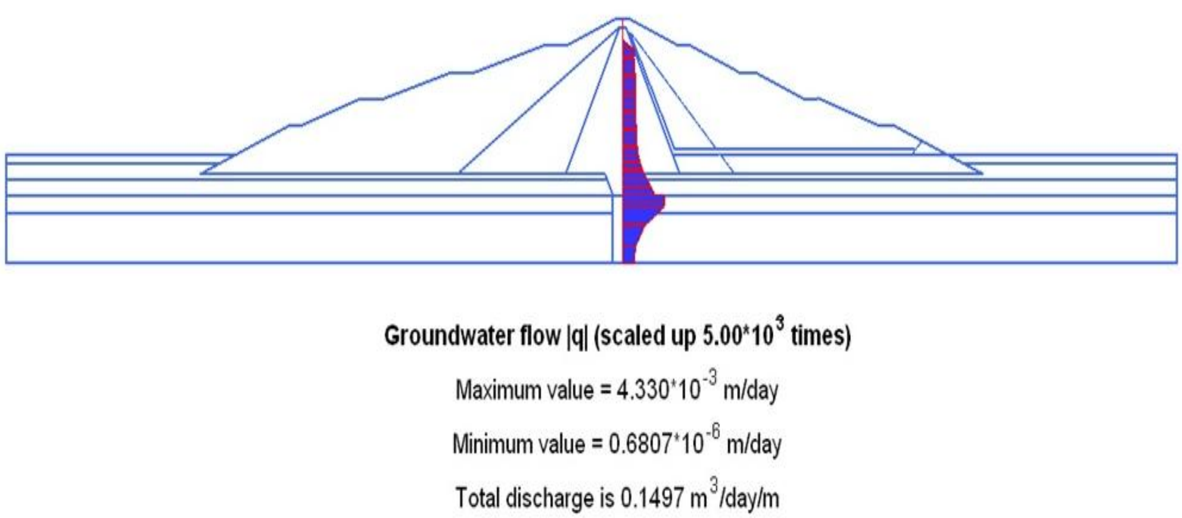


Figure 4.11 Quantity of seepage discharge during steady state condition

Table 4.1 Typical seepage losses from earthen dams (Quies, 2002)

Dam height (m)	Seepage, liter/day/meter, (Liters/minute/meter)	
	O.K.	Not O.K.
< 5	< 25 (0.02)	< 50 (0.03)
5 - 10	< 50 (0.03)	< 100 (0.07)
10 - 20	< 100 (0.07)	< 200 (0.14)
20 - 40	< 200 (0.14)	< 400 (0.28)
> 40	< 400 (0.28)	< 800 (0.56)

Seepage analysis by numerical modeling thus forms the basis for safety and stability assessment of the dam. As mentioned earlier, seepage analysis for existing condition is conducted for all earthen dam studies referred to CWPRS, irrespective of whether there are visible signs of seepage at site or not. This analysis paves way for conducting further studies to assess stability of the dam in its existing condition. Seepage studies also help to assess the need for seepage mitigation measures. Several mitigation measures are available, however selection of appropriate and optimum measures, depending upon site specific conditions can be decided based on seepage studies. Also, optimization of the remedial measures such as deciding the location and depth of cut-off-wall, length of upstream blanket, etc. with their corresponding efficacy in terms of seepage reduction can also be determined. Seepage analysis also provides values of critical parameters such as pore pressure and discharge quantity which can be monitored at site by instrumentation.

CHAPTER - 5

STRUCTURAL STABILITY ANALYSIS

Abhijit Khot, Assistant Research Officer

5. GENERAL

Upon conducting seepage analysis, the next step in comprehensive safety assessment of an earthen dam is to determine its structural stability. There are basically two ways to determine structural stability of an earthen dam viz. (i) to determine probable deformations in the dam and (ii) to determine the stability in terms of Factor of Safety of slopes. Although the performance of a dam is often dictated by allowable deformations, quantitative prediction of displacements is seldom carried out routinely. Instead, safety assessments by use of limit equilibrium methods in which the performance of a slope is evaluated in terms of its Factor of Safety (FS) are very common.

Various methods, including empirical, limit equilibrium, finite element and numerical simulations can be made use of to analyze slope stability for determining its Factor of Safety (FS). The significant advantage of limit equilibrium methods is that they take into account all major factors that influence the shearing resistance of soil. In addition, they are simpler than deformation analyses. However, because the actual stress-strain relations are not used, the limit equilibrium methods do not result in calculation of expected deformations. The Factor of Safety calculated using limit equilibrium methods cannot be measured. However, there are many instances in which precise deformations in a slope are of little concern, provided the material stays in place. Also, as suggested by various researchers, it is appropriate to undertake analyses with limit equilibrium methods; however, the empirical nature of the design criteria should be borne in mind.

The present Indian Code Standards, also mention limit equilibrium methods and the corresponding limiting safety criteria. As such, it becomes easier to directly draw inferences and conclusions about safety of slopes, in terms of Factor of Safety values based on the limiting criteria. In view of the above, static structural safety of earthen dams is assessed in CWPRS by limit equilibrium methods, unless otherwise governed by other necessities to determine stresses, strains and corresponding deformations. In the present chapter, various aspects of structural stability of embankment dams viz. the methods of analysis along with interpretation of results and their potential implications on dam safety, are discussed.

5.1. LIMIT EQUILIBRIUM METHOD OF STABILITY ANALYSIS

In limit equilibrium method the assumption of failure involves identifying potential failure surfaces along which the soil mass will slide, causing failure. The equilibrium of a soil mass tending to slide down under the influence of gravity is investigated. The common failure planes include planar, wedge, log spiral, circular or other shapes depending on the soil type and slope geometry. However, circular failure planes are common because they represent a simple, yet effective, geometry for calculating the driving and resisting forces on a slope. While planar and other non-circular failure surfaces can also be analyzed, circular failure surfaces is a widely used approximation and is also a reasonable assumption for most soil slopes.

Limit equilibrium methods are based on the comparison of forces, moments or stresses resisting movement of the soil mass with disturbing forces which causes motion of the soil mass. The procedures involve method of slices. In slip circle method, the soil mass is assumed to slide along a circular plane. The sliding mass is divided into number of vertical slices. For each slice, resisting and driving forces are calculated. Resisting forces are due to shear strength of soil mass which is derived from cohesion ‘c’ and angle of internal friction ‘ϕ’ between soil grains. Driving forces are due to weight of rubble and soil mass. Total resisting and driving forces are obtained by their summation for each slice and FS is calculated. Various trials of potential slip circles are required to be taken. Circle with lowest FS is known as the critical slip circle. The stability of a slope depends upon its geometry; properties of soil material used for construction of the dam and external forces to which the dam is subjected.

The basic assumption in limit equilibrium method is that Coulomb’s failure criterion is satisfied along the assumed failure surface. A free body is taken from the slope and starting from known or assumed values of forces acting upon the free body, the shear resistance of soil necessary for equilibrium is calculated. The calculated shear resistance is then compared to the estimated or available shear strength to give an indication of Factor of Safety (FS). Three static equilibrium conditions considered are: (1) equilibrium of forces in vertical direction, (2) equilibrium of forces in horizontal direction, and (3) equilibrium of moments about any point. Various limit equilibrium methods use at least one static equilibrium equations to calculate FS.

There are several methods given by different researchers depending upon above criteria. These methods differ in assumptions made in their formulations and inter-slice boundary conditions. Some of the methods are Swedish slip circle, Bishop's simplified, modified Bishop's, Spencer, Janbu's, Morgensten – Price, ordinary method of slices, etc. Some methods use all equilibrium equations and satisfy all of them ex. Morgenstern and Price and Spencer's method, whereas some methods use and satisfy only one equilibrium condition ex. ordinary method of slices and simplified Bishop's method. In general, the methods which satisfy both force and moment equilibrium criteria are considered to be more accurate and are recommended to be used.

Performing analytical calculations for slope stability is cumbersome due to large number of trials of slip circles required to be undertaken. However, wide variety of software with automatic search of critical slip circle are available. These software also incorporate different methods of slip circle analysis given by different researchers.

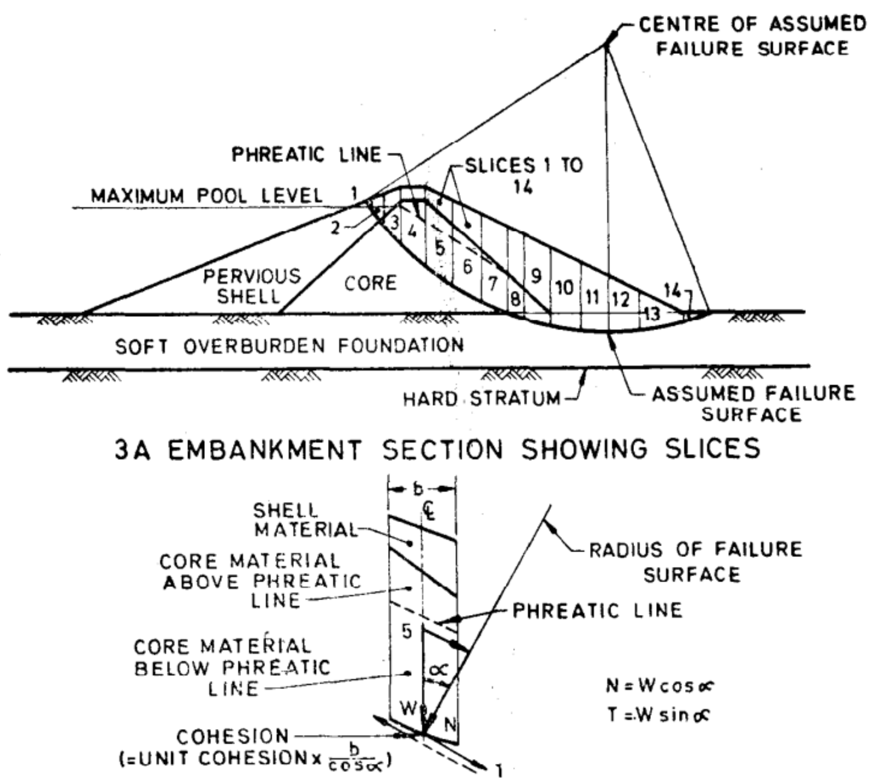


Figure 5.1 Limit equilibrium slip circle method of slices for slope stability

$$FS = \frac{\Sigma S}{\Sigma T} = \frac{\Sigma[\, C + (N - U)\tan\phi \,]}{\Sigma W \sin\alpha}$$

Where:

FS:	Factor of Safety	ϕ:	Angle of shearing resistance
S:	Resting or stabilizing force	W:	Weight of slice
T:	Driving or actuating force	α:	Angle made by radius of failure surface with vertical at centreline of slice
C:	C X (h / cos α)	c:	Unit cohesion
N:	Force normal to the arc of slice	B:	Width of slice
U:	Pore water pressure		

(Ref. IS 7894-1975)

5.1.1. Assumptions, advantages and limitations of limit equilibrium method

In limit equilibrium method, the stress system is assumed to be two-dimensional. The stresses in third direction (perpendicular to the section) are considered to be negligible or zero. Stability of a slope depends upon the phreatic condition and pore pressures in soil mass. Thus, before conducting slope stability analysis, it is assumed that the seepage conditions are known. The pore water pressure and phreatic conditions derived from seepage analysis, as explained in Chapter 4, forms the basis of slope stability analysis.

Limit equilibrium methods have several advantages over other methods of stability assessment. LEM is computationally less demanding and faster to execute compared to numerical techniques such as FEM or FDM which involve more complex numerical simulations. LEM also requires fewer input parameters, such as slope geometry, soil properties and external forces; so less assumptions are required to be made in case any properties are not available. The method is well suited for analyzing complex geometries and layered soil profiles, as it also allows for consideration of non-circular failure surfaces. LEM can be used in conjunction with other methods, such as FEM, to provide a more comprehensive analysis. The method is also well established and commonly used in practice

However, the method also has certain limitations. The method relies on assumptions about failure surface and inter-slice forces, which may not accurately represent actual site conditions. The method does not establish detailed stress-strain behavior of soils, which can be a limitation in certain complex scenarios. However, notwithstanding the above, LEM is still the most widely used and proven technique for assessing slope stability of earthen dams.

5.2. NUMERICAL MODELING (FEM, FDM) - STRENGTH REDUCTION METHOD

With advance computing technology, the use of the finite element and finite difference based software in calculation of stability have been developed. As the name suggests, in this method, shear strength parameters i.e. cohesion and friction angle are reduced until slope failure is induced. The Factor of Safety (FS) can be calculated at the point of failure as the ratio of the soil's shear strength parameters (effective cohesion and the tangent of the effective friction angle) to those required to bring the slope to failure. Shear strength reduction involves iteratively weakening the slope through a simultaneous reduction in effective strength parameters.

The method offers several advantages over traditional limit equilibrium methods (LEMs). It provides more realistic stress distributions, identifies critical failure mechanisms, and doesn't require assumptions about inter-slice forces that significantly impact the accuracy of LEMs. The method can account for stress-strain relationships, unlike LEMs which often rely on simplified assumptions about stress distribution. The analysis automatically identifies the critical failure mechanism, including the shape and location of the slip surface, without requiring a user to guess the failure surface. The method is well-suited for complex slope geometries, layered soil profiles, and non-homogeneous soil properties, where traditional LEMs may struggle. With increasing availability of high-speed computing power, the Strength Reduction method (SSR) is becoming increasingly efficient and practical.

The SSR method can be used with various finite element and finite difference software packages, offering flexibility in implementation. SSR analysis not only provides a Factor of Safety (FS) but also offers insights into the stress and strain distributions, failure mechanisms, and displacement patterns within the slope. However, though the FEM and FDM based methods have been commonly used for deformation analysis of embankments and other geotechnical problems; they are still not widely used for stability analysis of slopes as compared to the conventional limit equilibrium methods.

Earthen dams are prone to settlement over time due to sinking or consolidation of soil material in dam body and foundation due to weight of the structure and due to applied loads. This settlement can result in loss of freeboard, reduction in the dam's storage capacity and if excessive, can also potentially compromise structural integrity of the dam. FEM and FDM based numerical modeling techniques offer a great tool to estimate these settlements so that timely remedial measures can be implemented to avoid any mishaps.

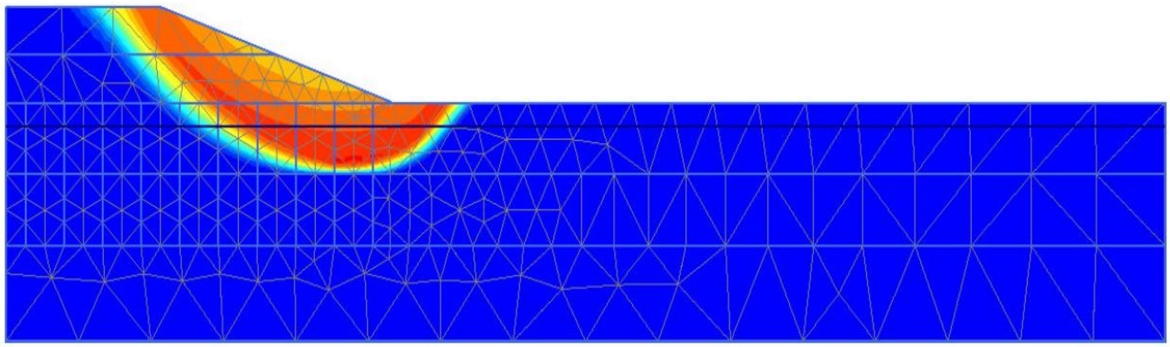


Figure 5.2 Failure surface by strength reduction method

5.3. Critical CONDITIONS OF STABILITY

The primary purpose of slope stability analysis by limit equilibrium method is to provide a quantitative measure of stability of dam slopes in terms of FS. It is essential that the dam slopes are stable under all critical loading conditions during various phases of construction and reservoir operation. These critical conditions can be different for upstream and downstream slopes. As per guidelines in IS 7894-1975 the critical loading conditions are: construction condition with or without partial pool, reservoir partial pool, sudden drawdown, steady seepage, steady seepage with sustained rainfall and earthquake condition. Table 5.1 lists the critical loading conditions specific to upstream and downstream slopes with minimum required value of FS.

Table 5.1 Critical loading conditions and minimum required Factor of Safety

No.	Loading condition	Critical slope	Minimum FS
1	Construction condition with, or without partial pool	Upstream and downstream	1.0
2	Reservoir partial pool	Upstream	1.3
3	Sudden drawdown a. Maximum head water to minimum with tail water at max b. Maximum tail water to minimum with reservoir full	 Upstream Downstream	 1.3 1.3
4	Steady seepage with reservoir full	Downstream	1.5
5	Steady seepage with sustained rainfall	Downstream	1.3

6	Earthquake condition a. Steady seepage b. Reservoir full	Downstream Upstream	1.0 1.0

(Ref: IS 7894:1975)

5.4. SLOPE STABILITY ANALYSIS

As mentioned earlier, seepage through the dam significantly impacts its stability by creating pore water pressure and thereby reducing effective strength of soil. Thus, seepage analysis by numerical modeling is often the first step in conducting slope stability analysis. For earthen dams referred to CWPRS for safety and stability assessment; seepage analysis is conducted initially using software PLAXIS followed by slope stability analysis by limit equilibrium method using software TALREN. Software TALREN has the facility of importing the results of seepage analysis viz. pore pressure and phreatic line from software PLAXIS. Seepage analysis followed by slope stability analysis is thus carried out for upstream and downstream slopes for various critical conditions as listed in Table 5.1.

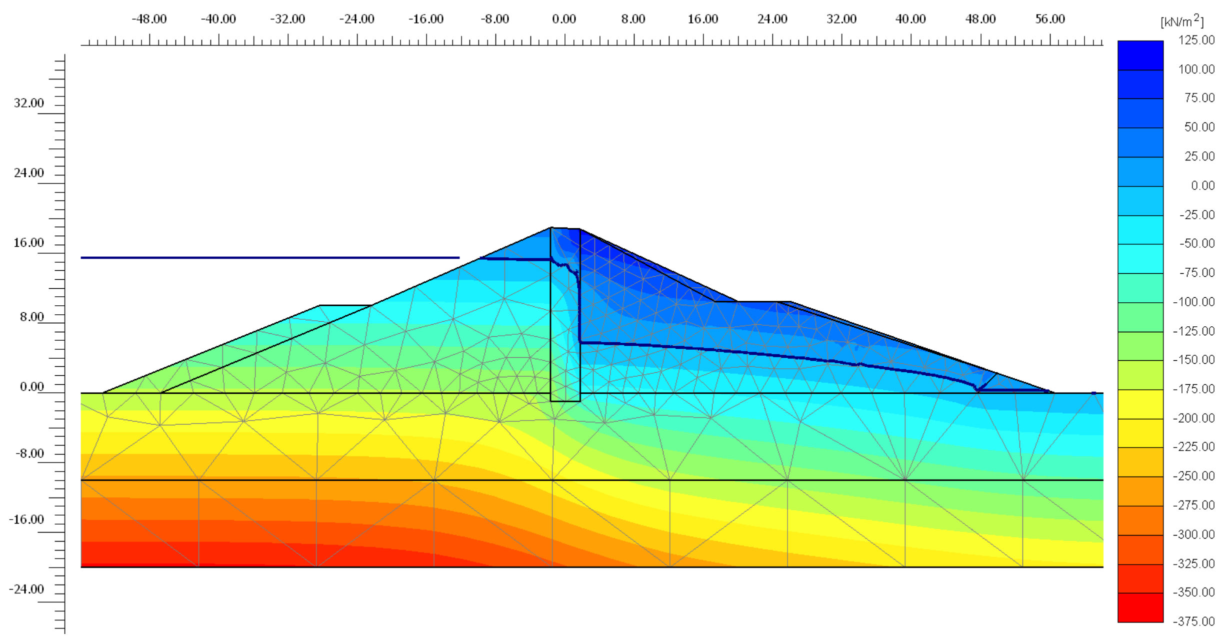


Figure 5.3 Phreatic line generated from seepage analysis in PLAXIS 2D

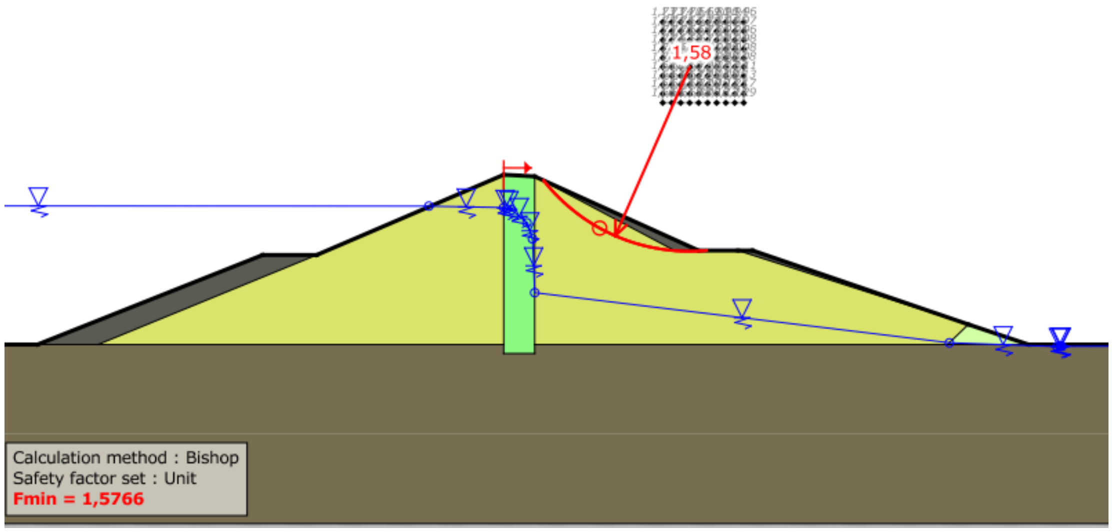


Figure 5.4 Phreatic line imported in software TALREN from PLAXIS

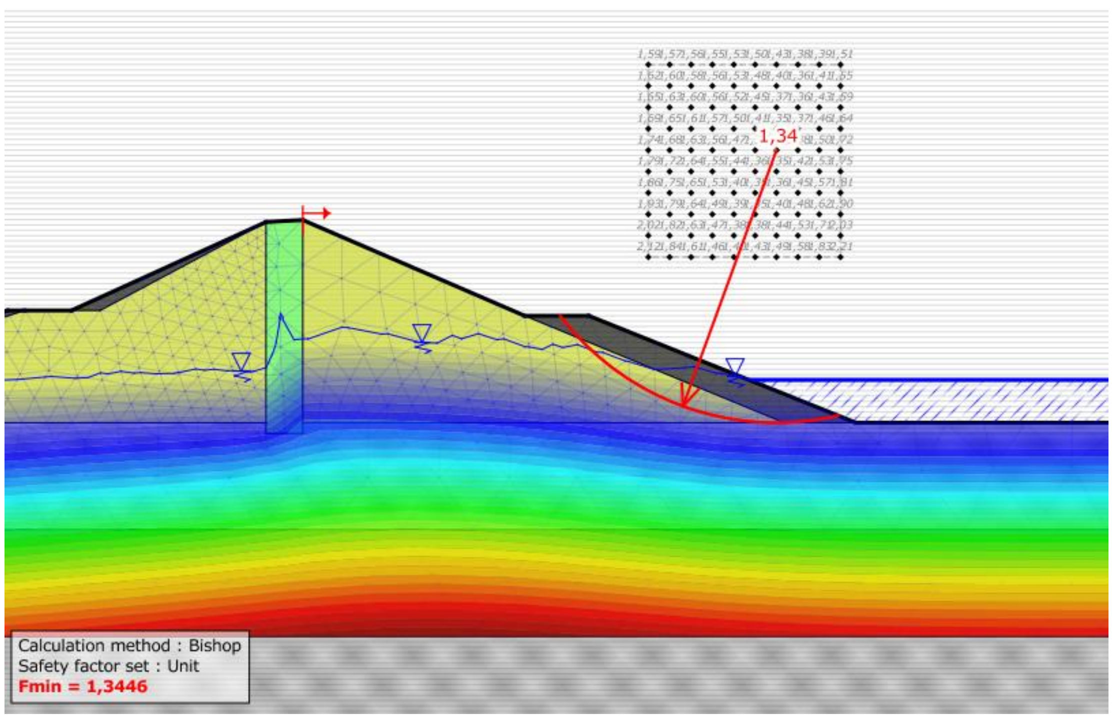


Figure 5.5 Pore pressures imported in software TALREN from PLAXIS

A geometrical model of the earthen dam with various zones viz. hearting, casing, filter, rock toe, etc. is created in software TALREN for slope stability analysis. Foundation strata with different soil/ rock layers is also modeled. Input properties viz. density, cohesion and internal friction angle are assigned to each soil/ rock zone. The model is run for various trials of slip circles and FS is determined for each circle. The circle with lowest FS is termed as the critical slip circle.

The FS of critical slip circle is compared with the minimum required value for corresponding conditions. The slope is termed as unsafe if FS obtained is lower than the minimum required value as listed in Table 5.1.

5.4.1. Selection of dam cross-sections for stability analysis

Generally, for an earthen dam, seepage and slope stability analyses are conducted for various cross-sections along its length. The number of cross-sections for conducting stability analysis are decided depending upon various factors such as length of the dam; extent, quantum and locations of distresses; significance and risk factor involved in eventuality of dam failure; variation in geometry of dam cross-section; heterogeneity in soil material used for construction; variation in foundation conditions; construction quality; history of unprecedented conditions such as floods; earthquakes, etc. Fig. 5.5 shows a figure indicating locations of five cross-sections selected for stability analysis based on above criteria for the 10 km long Kangsabati – Kumari dam in West Bengal.

5.4.2. Finalizing design soil properties

Once the dam cross-sections and chainages are selected for analysis, the next step is to finalize input design parameters. Geotechnical investigations conducted at site including boreholes, trial pits, borrow area surveys, laboratory and in-situ test results, etc. form the basis for selection of input design parameters. Various geotechnical investigations are elaborated in Chapter 3. Initially the available data with project authorities is reviewed to assess its sufficiency for conducting analysis of various selected cross-sections. If required, additional geotechnical investigations are recommended to be conducted at site. Complete guidance for geotechnical investigations viz. location and number of boreholes and trial pits, laboratory and field tests to be conducted, etc. is provided to project authorities. Few confirmatory tests on selected soil samples are also conducted at CWPRS laboratory.

Based on the already available test reports and those of additional investigations, the final input design parameters are selected. However, the test results usually give a wide range of values for each parameter. From this range, design values of different parameters for a soil type are finalized. Design value is the one which is adopted in calculations of safety analyses. The value is generally selected as average of all test results. With additional conservatism, design value can also be selected such that 75% of test results are above it. In either case, offshoot values are however neglected. Example of calculating design value of friction angle from a range

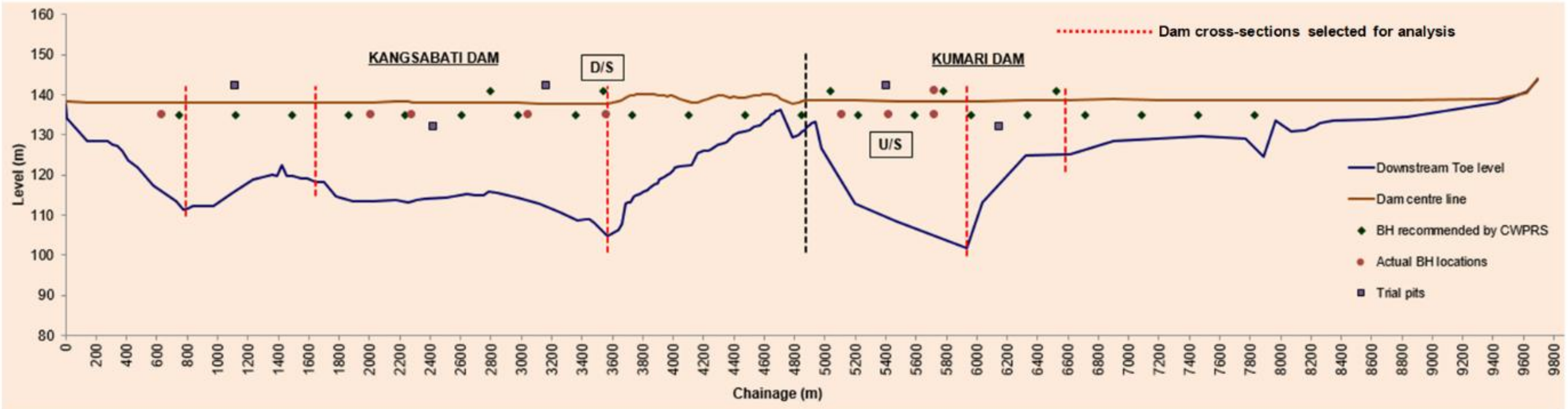


Figure 5.6 Locations of dam cross-sections selected for analysis

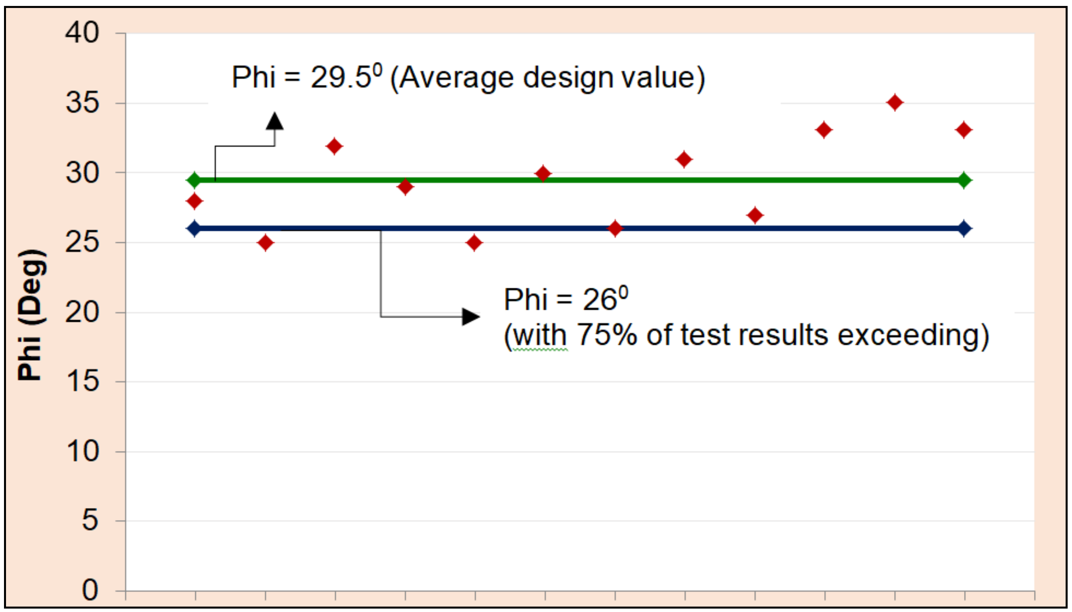


Figure 5.7 Design value from test results

of test results is shown in Fig. 5.7. From the figure, it is seen that the average design value works out to be 29.5° while design value with 75% results exceeding, works out to be 26°. On similar lines, design value of each parameter required for analysis is calculated for all soil types in dam body as well as in foundation. Sample design input values of each parameter for different zones of an earthen dam are shown in Table 5.2.

Table 5.2 Representative design input parameters for stability analysis

Sr. No.	Parameter	Casing	Hearting and COT	Filter	Rock-toe	Foundation	
						Layer 1	Layer 2
1	Bulk density (kN/m^3)	19.42	20.11	18.00	20.00	20.11	19.92
	Saturated density (kN/m^3)	20.26	20.33	18.00	20.00	20.33	20.23
2	Cohesion (kN/m^2)	18.24	83.35	0.00	0.00	83.35	15.69
3	Friction Angle (deg.)	32.83	4.95	30.00	40.00	4.95	34.50
4	Young's Modulus of elasticity (MPa)	190.00	6.00	50.00	200.00	6.00	200.00
5	Poisson's ratio	0.28	0.32	0.30	0.28	0.32	0.28
6	Permeability (m/sec)	5.922E^{-5}	13.63E^{-8}	4.63E^{-4}	1E^{-3}	13.63E^{-8}	1.33E^{-9}

5.4.3. Slip circle analysis

Upon finalizing the dam cross-sections and design input parameters for analysis, slope stability is conducted by limit equilibrium method using software TALREN. The geometry of dam cross-section along with foundation strata is created in the software. Design parameters are assigned to each soil/ rock zone in the dam body and foundation. Results of seepage analysis for corresponding condition viz. steady seepage, sudden drawdown, etc. are imported in the form of pore pressure or phreatic line. Various trials of slip circles are taken to determine the critical slip circle with lowest Factor of Safety (FS).

5.5. INFERENCES FROM RESULTS AND NEED FOR REHABILITATION MEASURES

The critical FS determined as described above is compared with minimum required value of safety corresponding to each critical condition as listed in Table 5.1. If the FS values are less than the minimum required values, the dam slope is designated as unsafe for that particular condition, and it is inferred that rehabilitation measures are required to be implemented to achieve structural safety of the dam.

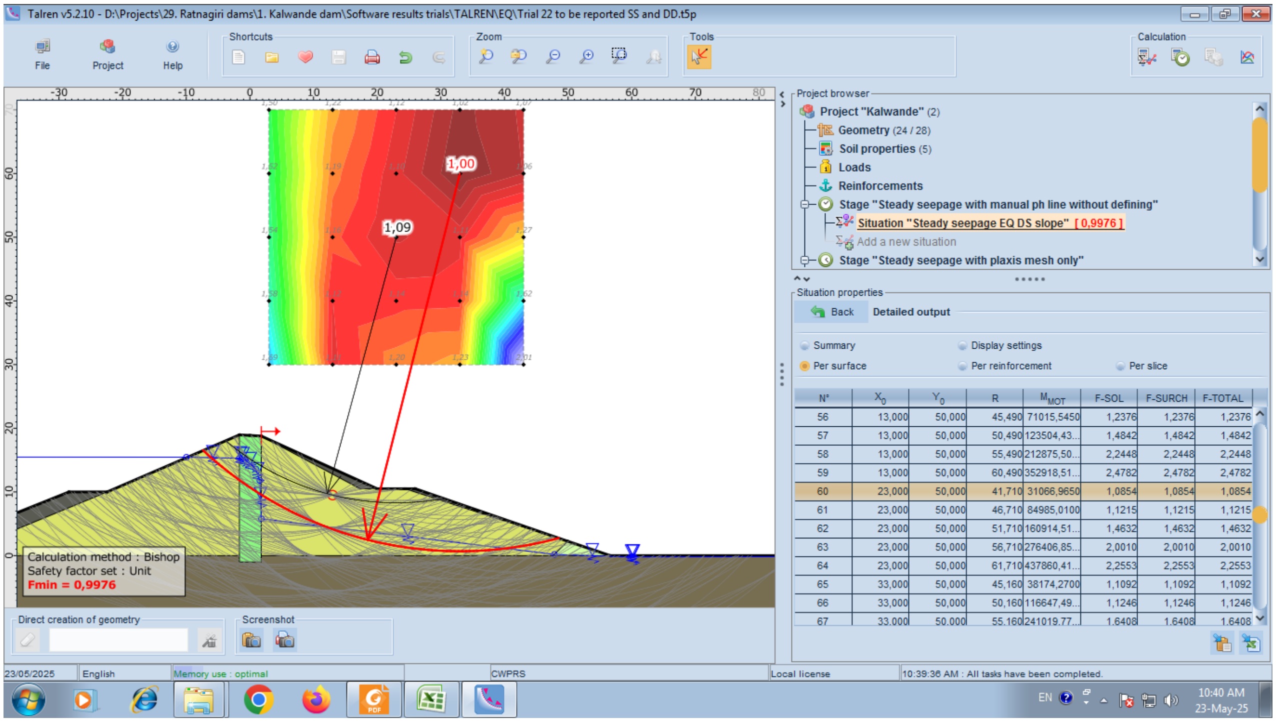


Figure 5.8 Search for critical slip circle with lowest Factor of Safety

A number of rehabilitation measures are available for earthen dams. Details of the same are discussed in Chapter 7. However, selecting the most suitable remedial measures is a challenging job as every problem is unique owing to its site-specific characteristics. Effective application of investigatory methods and analysis tools described in this chapter not only help in identifying the cause of problems but also to evolve suitable remedial measures for attaining dam safety.

CHAPTER - 6

DYNAMIC STABILITY

J. S. Edlabadkar, Scientist 'D'

6. GENERAL

Earthquakes are a major cause of damage and failure in earthen dams. There are reported incidences of failures in earthen dams during the 7.6 M_w Bhuj earthquake in Gujarat on 26th January 2001. Severe damages such as significant displacements, lateral spreading, cracking, slumping, heaving, etc. have occurred in various dams viz. Chang, Shivlakha, Suvi, Tapar, Fatehgarh, Kaswati, Rudramata, etc. during the Bhuj earthquake. Damages in earthen dams occur as a result of large reduction in stiffness and strength of soil due to dynamic earthquake loading. Understanding the behaviour of earthen structures under seismic loading has increased considerably in the past 30 years as a result of field observations, extensive laboratory testing of soil samples under cyclic loading and development of numerical modeling procedures to simulate dynamic loading and its effect on soil behaviour. The present chapter elaborates upon the various methods for assessing seismic stability of earthen dams along with determination of dynamic soil properties.

6.1. OVERVIEW OF SEISMIC ANALYSIS METHODS

Till recently, seismic stability of earthen dams was computed by means of limit equilibrium method viz. the pseudo-static method which is an extension of static limit equilibrium procedure as explained in the previous chapter. In pseudo-static method, the effect of earthquake is represented by a constant, static inertia force acting at the centroid of failure mass and which is equivalent to the corresponding horizontal and vertical accelerations. In this method, safety of the dam is measured in terms of a numerical value known as the Factor of Safety (FS). Being the simplest method, this has limitation of not being able to represent the highly non-uniform dynamic nature of earthquake loading, as well as the true non-linear hysteretic behaviour of soil. Also, research has shown that, this method of analysis is unreliable for soils that build up large pore pressures or show more than 15% degradation of strength due to earthquakes. It has been obse-

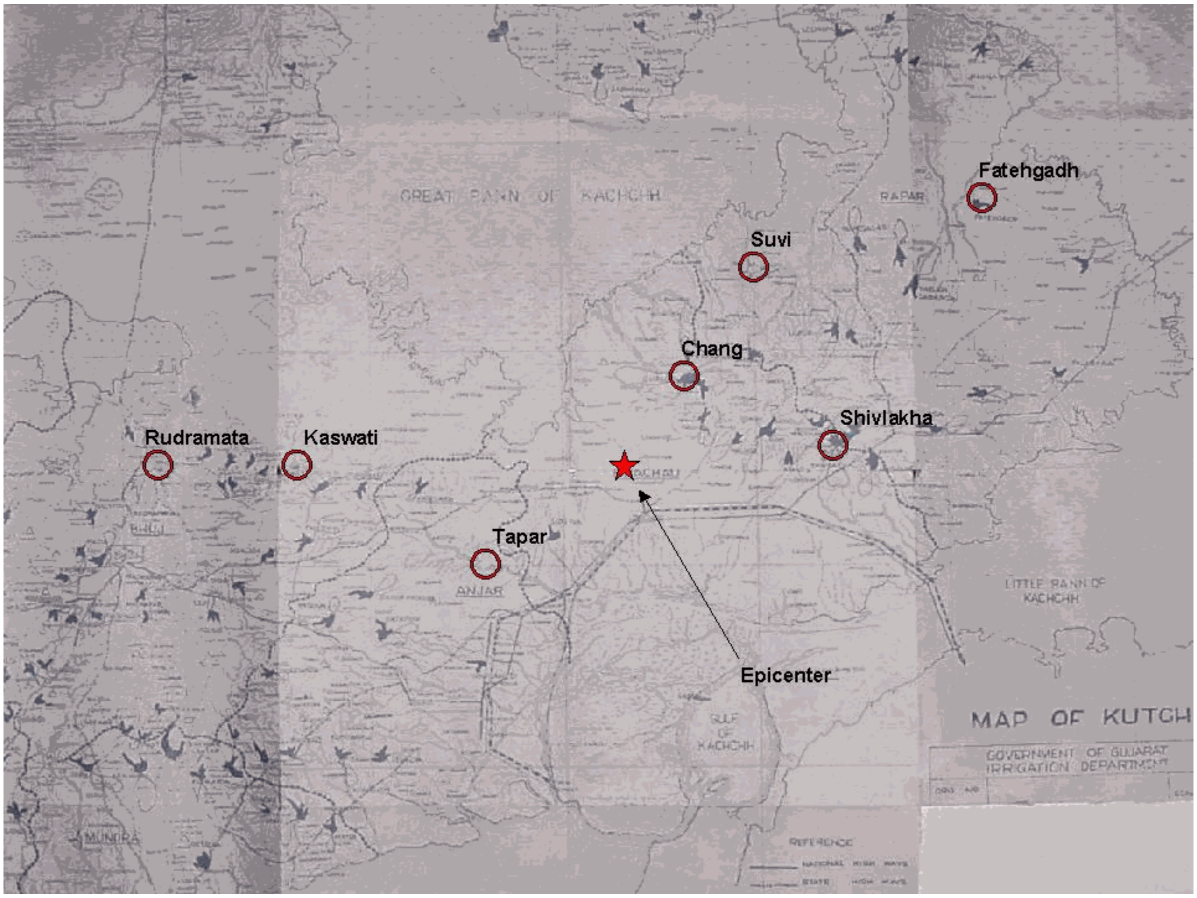


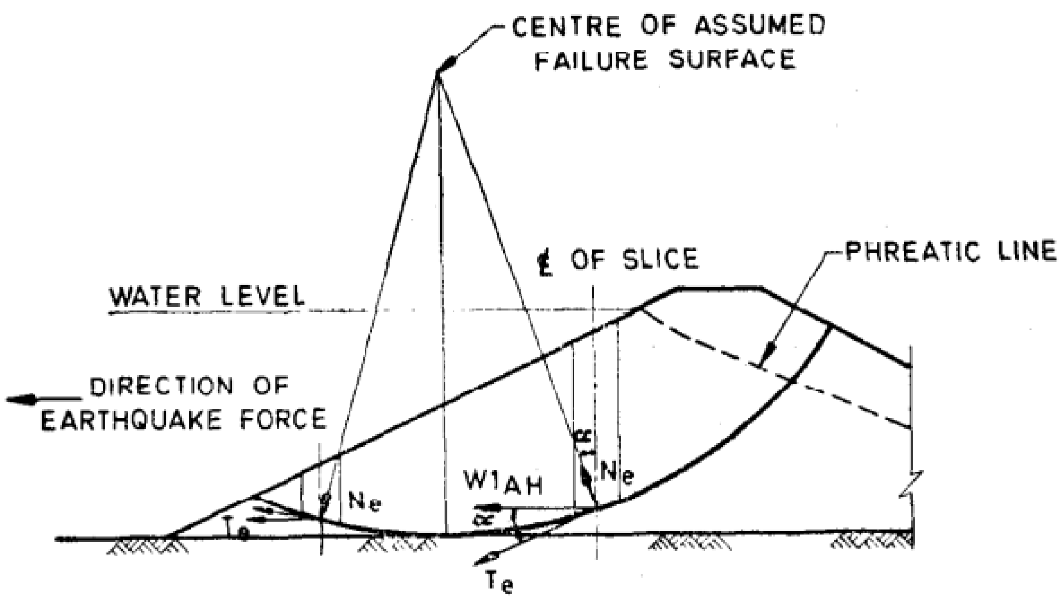
Figure 6.1 Location of earthen dams damaged during Bhuj earthquake

(Source:https://geerassociation.org/components/com_geer_reports/geerfiles/Bhuj_2001/Dams%20Web%20Page.htm#Shivlakha%20Dam)

Figure 6.2 Cracking at Shivlakha dam, Gujarat

Figure 6.3 Widespread liquefaction, cracks and sand boils at Chang dam, Gujarat

rved that number of dams which were analyzed by the pseudo-static approach and having FS more than 1 have failed during earthquakes; ex. Sheffield dam, Lower San Fernando dam, Upper San Fernando dam, etc. Thus, the pseudo-static approach is known to provide only an index of relative stability of earthen dams during earthquakes.



$W1$: Saturated unit weight of the material in the slice (below phreatic line)
A_H : Horizontal earthquake coefficient
T_e : Tangential component of earthquake force (W1.cosα.AH)
N_e : Normal component of earthquake force (W1.sinα.AH)

Figure 6.4 Pseudo-static limit equilibrium slope stability method

(Source: IS 7894 - 1975)

As a result, the dynamic analysis procedures for evaluating seismic response of earthen dams have evolved over the years. Today, the methods range of simplest (pseudo-static) to highly complex having capability of modeling the dynamic nature of earthquake loading and non-linear response of soil. Serviceability of an earthen dam after earthquake largely depends upon the amount of deformation it has undergone during earthquake. Relatively advanced methods of seismic response analysis also provide information on amount of stress, strain, deformation and pore pressure developed in the soil mass due to dynamic loading, which is not possible in pseudo-static method. The accuracy in results of these advanced methods however depends upon their capability to model the earthquake loading and dynamic response of soil as closely as possible. Some of the methods of seismic analysis of earthen dams are briefly described below.

6.1.1. Empirical methods

These methods are based on empirical charts or equations derived from site observations of existing dams during earthquakes. The methods are useful for rapidly predicting dam behaviour due to earthquake motion by empirical equations or charts. Methods given by researchers viz. Jansen, Swaisgood, Pells and Fell, Singh and Debasis, etc. are some of the commonly used empirical methods.

6.1.2. Simplified methods

Simplified methods are generally based on the Newmark’s analogy. Newmark used the analogy of a block resting on an inclined plane to predict its displacement due to earthquake. When a block on an inclined plane is subjected to a pulse of acceleration that exceeds yield acceleration (K_y), the block will move relative to the plane. Yield acceleration is the value at which the FS reduces to unity. Relative displacement of the block depends on amplitude as well as frequency of that pulse. Final cumulative displacement is the summation of relative displacements during all instances when the actual acceleration has exceeded yield acceleration.

Researchers used this analogy to determine displacements in earthen dams during earthquakes by comparing the sliding block with soil mass. In sliding block method, the block is assumed to be rigid. However, actual soil slopes are compliant and they deform during earthquake. Their response depends on geometry and stiffness of the soil mass as well as on the amplitude and frequency of earthquake motion. For a slope composed of very stiff soil and subjected to low frequency motion, lateral displacement in the failure mass will be nearly in phase and the rigid block assumption is approximately satisfied. Thus, the sliding block analysis is based on assumption of rigid-perfectly plastic stress strain behaviour on a planar surface. In reality, soils rarely behave as perfectly plastic materials. They exhibit strain hardening or strain softening behaviour after yielding. Movement of slope on a non-planar failure surface tends to flatten the slope thereby reducing the driving forces. As a result, the yield acceleration should increase due to changes in geometry of the unstable soil. However, for most slopes this effect does not become significant until large displacement has occurred.

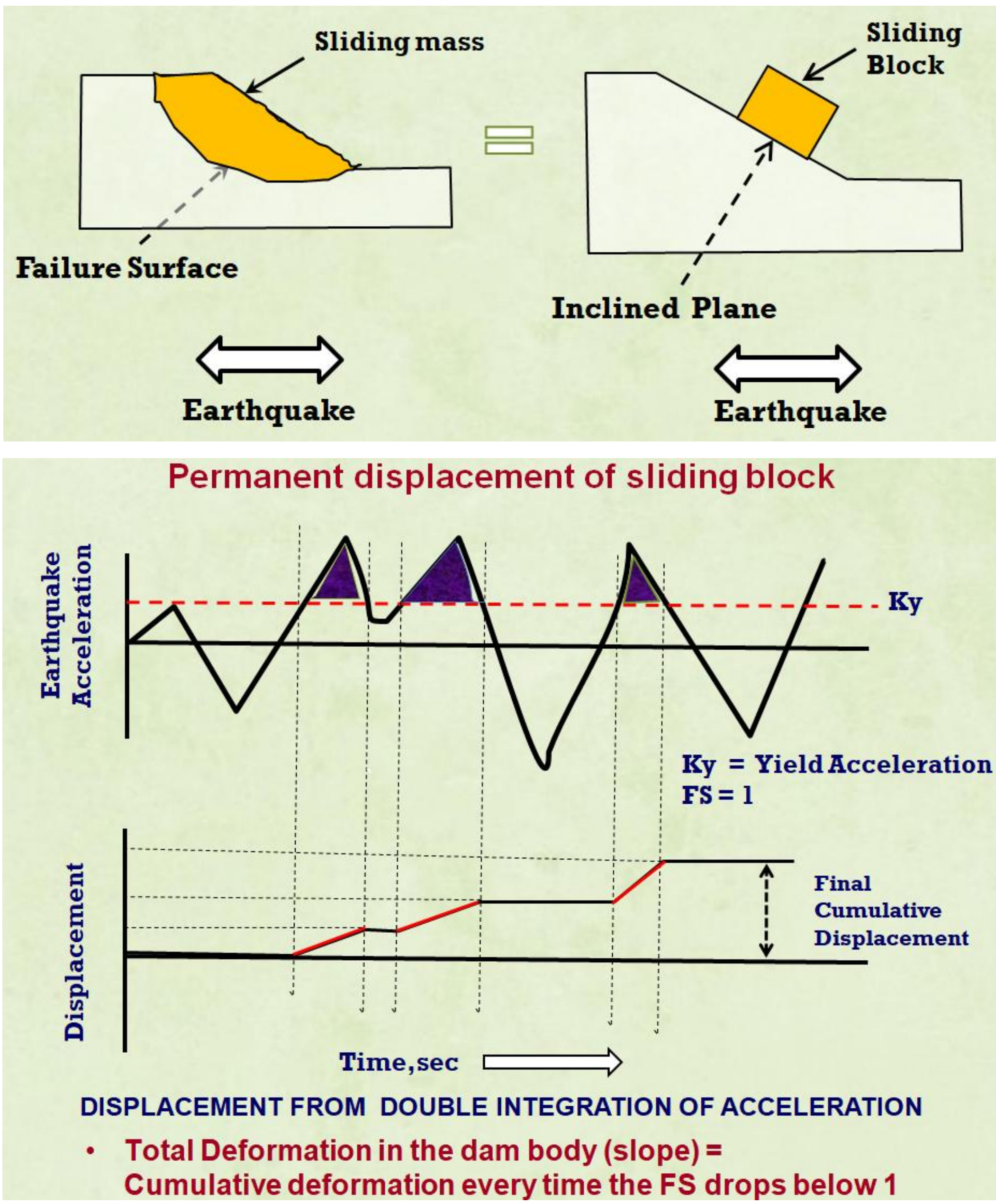


Figure 6.5 Schematic representation of Newmark's sliding block analogy

Various researchers viz. Sarma, Franklin and Chang, Ambraseys and Menu, Yegian et. al., Watson-Lamprey and Abrahamson, Jibson, Saygili and Rathje, Rathje and Antonakos, Hynes-Griffin and Franklin, Bray and Rathje, Bray and Travasarou, Makdisi and Seed, etc. have given different simplified methods. Makdisi-Seed refined Newmark's method by computing the variation of permanent displacement with ratio of yield acceleration and peak crest acceleration and earthquake magnitude (M); by subjecting several real and hypothetical dams to recorded and synthetic earthquake ground motions for given magnitudes.

6.1.3. Advanced methods

These methods are capable of covering a wide range of complexity in modeling earthquake motion and soil response. They comprise of dynamic analyses using total and effective stress methods, linear and non-linear soil response, coupled deformation modeling capability and sophisticated pore pressure models.

a) Total stress concept

Total stress methods are based on the total stress concept in which pore pressure development during an earthquake is not calculated. Therefore, these methods are used in situations where seismically induced pore pressure is negligible. These methods are basically of two types:

(i) Equivalent linear: In this method the non-linear soil response under cyclic loading is approximated as linear and hence the name equivalent linear. The output acceleration/ shear stress-time history from equivalent linear analysis can be used in a simplified deformation analysis by the Newmark method for calculating deformations. Prof. Seed developed a procedure for estimating earthquake induced slope deformation from the results of equivalent linear analyses. The cyclic shear stresses induced in each element is computed from dynamic Finite Element analyses. Dynamic response of the dam is determined by solving the equation of motion for site specific earthquake loading. Using the results of cyclic laboratory tests, the computed cyclic shear stresses are used to predict the strain potential, expressed as shear strain, for each element. Deformations are then estimated as the product of average strain potential along a vertical section through the slope and height of that section.

(ii) Nonlinear: This method utilizes the actual nonlinear in-elastic stress-strain behaviour of soil using direct numerical integration in time domain to compute development of permanent strain during earthquake. The accuracy of nonlinear analysis mostly depends upon the stress-strain or constitutive models used.

b) Effective stress concept

Effective stress methods are most comprehensive in estimating dam deformations during earthquake loading. This approach allows for modeling of various parameters viz.: (i) pore pressure development, (ii) loss of strength and stiffness due to increase in pore pressure and (iii) residual strength after liquefaction. The effective stress methods can be fully coupled, semi-coupled and uncoupled. The fully coupled prediction of pore pressure under cyclic stresses is extremely complicated and challenging.

These approaches of determining permanent deformation in earthen dams are considered acceptable for practical purposes rather than calculation of only FS. Conducting seismic analysis for estimating dam deformations requires input of various dynamic soil parameters. A brief about these parameters is given below.

6.2. DYNAMIC SOIL PROPERTIES

Mechanical properties of soil associated with response to seismic excitation are known as dynamic soil properties. For carrying out seismic analyses of earthen dams the primary input parameters are: unit weight (γ_{dry}, $\gamma_{bulk,}$ γ_{sat}), Poisson’s ratio (ν), coefficient of earth pressure at rest (k_0), shear modulus (G), damping (D), variation of ‘G’ and ‘D’ with shear strain (υ), cyclic strength, etc. Shear modulus and damping have the largest effect on values of soil response, hence compatible values of these parameters are chosen corresponding to the shear strain level induced in soil during earthquake. Successful applications of methods for seismic response depend upon use of correct estimates of these strain dependent parameters.

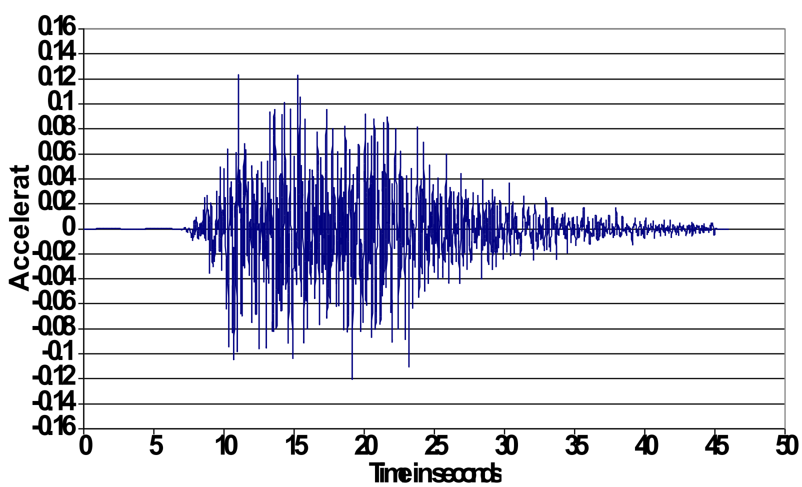


Figure 6.6 Horizontal earthquake record of Maximum Credible Earthquake

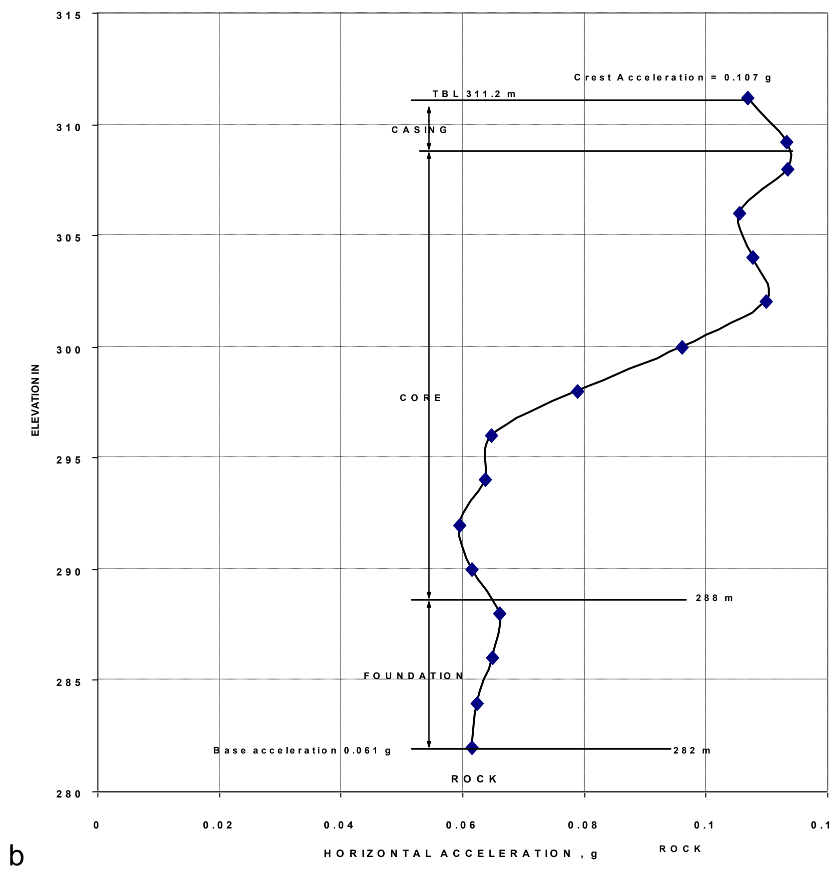
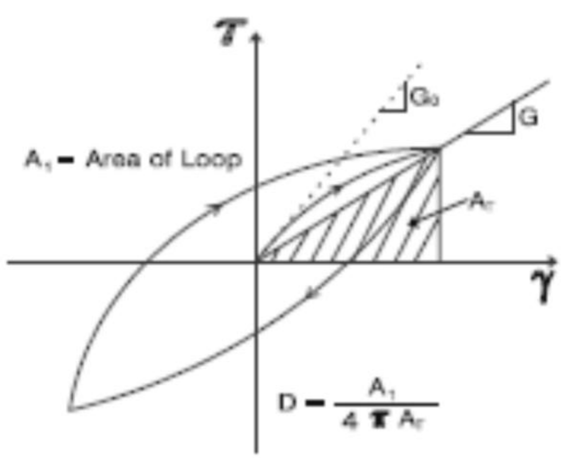
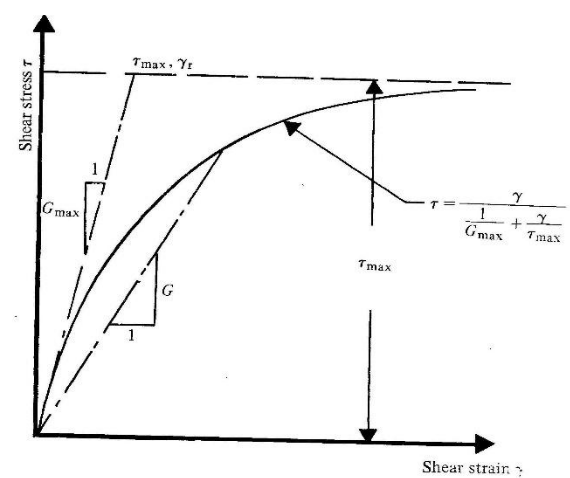


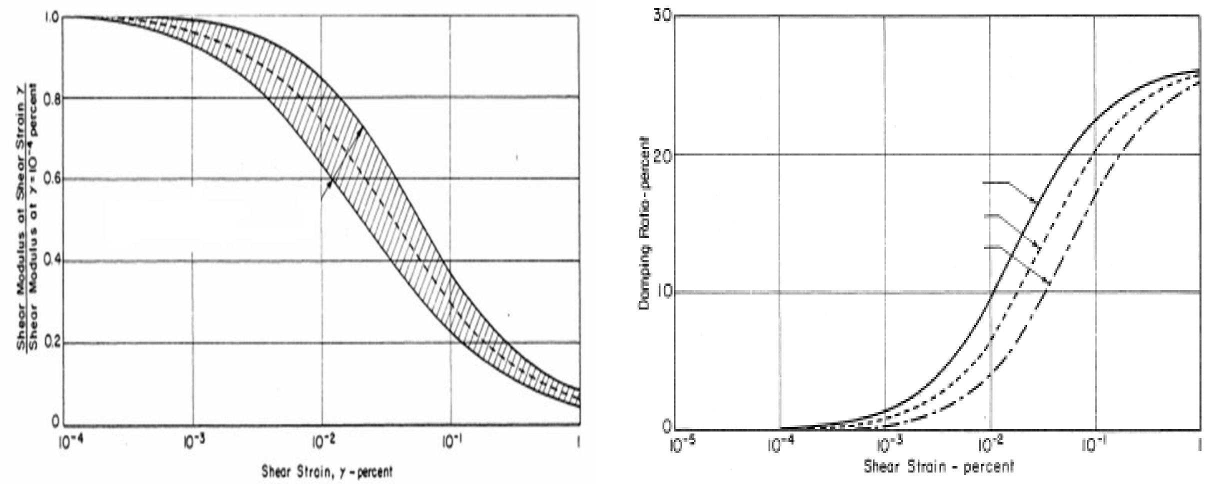
Figure 6.7 Horizontal acceleration determined from numerical modelling





(a) **(b)**

Figure 6.8 (a) Dynamic shear modulus (G) and Damping (D) (b) Variation of Shear stress with Shear strain



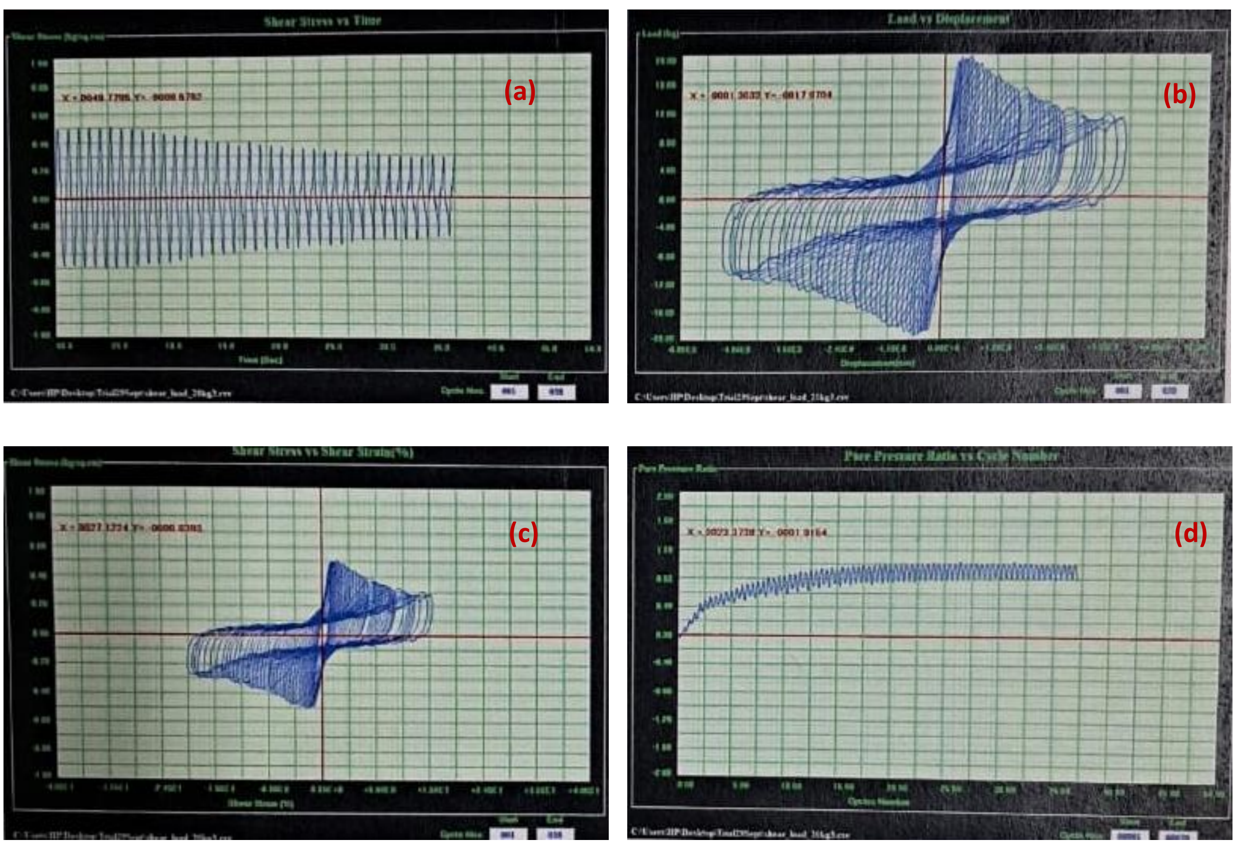
(a) (b)

Figure 6.9 (a) Variation of Shear modulus with Shear strain (b) Variation of Damping with Shear strain

6.2.1. Determination of dynamic soil properties

Various field and laboratory tests are conducted at the dam site and on soil samples extracted from boreholes and trial pits to determine dynamic soil properties. These properties are further used in conducting dynamic stability analysis of the dam. The shear strain amplitude capability ranges for different field and laboratory tests vary. Field tests mostly comprising of geophysical methods such as seismic reflection, seismic refraction, suspension logging, steady state vibration (Rayleigh wave test), seismic downhole/ up hole test, seismic cross-hole test, spectral analysis of surface waves, seismic cone test, etc. are used for determination of shear modulus/ shear wave velocity at low strain levels. Laboratory tests such as bender element, resonant column, torsional shear, cyclic triaxial, cyclic simple shear, shake table, etc. help in estimation of a realistic range of dynamic soil properties ranging from low to high strain. Determination of dynamic properties is a critical, at the same time an extremely important task in geotechnical earthquake engineering problems. Some of the test methods them are listed in Table 6.1. Selection of testing techniques, however, requires careful consideration and understanding of the specific problem.

Figure 6.10 Laboratory cyclic shear test equipment



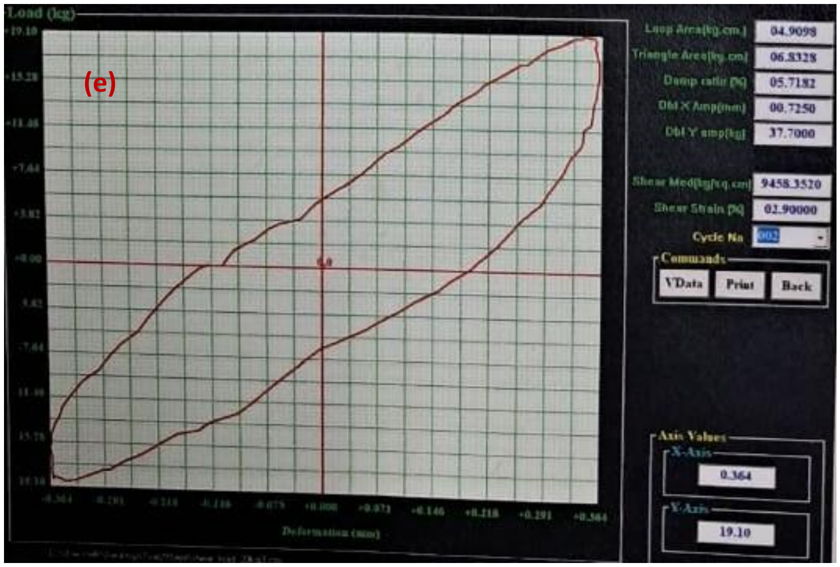


Figure 6.11 Results of cyclic simple shear test on soil sample (a) Shear stress vs. Time (b) Load vs. Displacement (c) Shear stress vs. Shear strain (d) Pore pressure ratio vs. No. of cycles (e) Load vs. Deformation for Cycle No. 2

Field tests are to be conducted with minimum disturbance to the soil structure, thus preserving the effects of fabric and aging on measured properties; however these are majorly restricted to determination of soil properties at low strain levels. Field testing has the advantage of determination of soil properties in its natural in-situ stress but is associated with relatively poor determination of material damping. Laboratory tests have the advantages of having good control of stress history, stress path and stress–strain levels. Also large database is available to validate the testing results. However, sampling disturbances during laboratory testing affect the representativeness of in-situ soil conditions. Specimen dimensions, as compared to natural deposits, may also vary the response to real earthquake excitations. Dynamic soil properties, determined by any of these testing methods, are influenced by factors such as confining pressure, stress history, shearing strain amplitude, number of loading cycles, degree of saturation and drainage conditions, etc.

6.3. NEED FOR SEISMIC SAFETY ASSESSMENT AND CHOICE OF METHODS

With the advent of Dam Safety Act 2021, it is pertinent that seismic safety of existing dams is ensured to avert disasters. Most of the existing dams are ageing and require rehabilitation measures. Moreover, old dams were designed and constructed when seismic design practices were very preliminary and seismic designs were generally carried out using basic methods. Seismic zoning pattern of India has also been revised. In view of the above, it is important that necessary measures to assess seismic stability of dams by advanced methods are implemented and safety of the dams is ensured.

Table 6.1 Field and laboratory tests for determination of dynamic soil properties

Field tests	
Low strain tests	High strain tests
1) Seismic reflection 2) Seismic refraction 3) Suspension logging 4) Steady state vibration (Rayleigh wave test) 5) Seismic downhole test (Up hole test) 6) Seismic cross hole test 7) Spectral analysis of surface waves 8) Seismic cone test	1) Standard Penetration tests 2) Cone penetration tests 3) Dilatometer test 4) Pressure meter test
Laboratory tests	
Low strain tests	High strain tests
1) Resonant column tests 2) Ultrasonic pulse tests 3) Piezoelectric bender elements	1) Cyclic triaxial tests 2) Cyclic direct simple shear 3) Cyclic torsional shear
Special tests	
Physical modeling is employed to simulate response of the full scale prototype structure 1) Shaking table tests 2) Centrifuge tests 3) Hydraulic gradient similitude tests	

In this chapter, an overview of various methods for assessing seismic performance of earthen dams is presented. The field of earthquake engineering is evolving and the methods range from basic, empirical, simple to complex and advanced. All these methods come with their own drawbacks and even the most complex ones can give a range of results due to approximations required to be made and due to their limitations in accurately modeling earthquake excitations and soil response. In addition, site specific challenges only add up to complexity of the problem. As such, it is important that appropriate methods for estimating seismic deformations in earthen dams are used.

Usually, the seismic stability of an earthen dam is carried out in stages. Initially the Pseudo static analysis by seismic coefficient method is carried out. Depending upon the Factor of Safety (FS) values for different operating conditions, further requirement of detailed studies is assessed. If the FS is found to be marginal or lower than requirement, then next stage of studies i.e. Deformation analysis using Newmark method is carried out. If the analysis indicates acceptable order of deformation for Design Basis Earthquake (DBE), then further analysis may not be required. However, if the deformation is large, then detailed dynamic analysis may be carried out. For small and medium height earthen dams, deformation analysis is sufficient. In case of large earthen dams, detailed dynamic analysis is recommended. Using advanced methods of seismic analysis entail high cost, considerable time and sophisticated investigations for determination of dynamic soil properties. Hence, the choice of method of analysis depends on following factors: importance of the project, its risk and hazard potential, probability of occurrence of high seismic events in the region (seismic zone), soil type used for construction of the dam (whether the soil is potentially liquefiable and prone to strength degradation when subjected to cyclic stresses), availability of good quality and sufficient data including design drawings, geotechnical investigation reports, dynamic soil properties, etc.

Based on the studies, the need for rehabilitation of the dam for seismic retrofitting is assessed. The best suitable rehabilitation methods is also selected from a range of available methods such as adding berms and buttresses, excavate and replace weak soil with material having high strength and seismic resistance, in-situ densification, in-situ strengthening by addition of admixtures, increase freeboard, provision of additional drainage for dissipation of pore water pressure, etc. The rehabilitation measures are discussed in detail in Chapter 7.

CHAPTER - 7

REHABILITATION MEASURES

Dr. Tanusree Samanta, Scientist 'C'

7. GENERAL

For various reasons a dam needs to undergo rehabilitation. All dams are subjected to natural aging, environmental degradation, operational stresses and evolving hydrological conditions over time which necessitate their rehabilitation. Old dams sometimes may not meet modern safety standards (for static as well as seismic conditions) and performance criteria, which also necessitate reassessment of their safety and implementation of rehabilitation measures. Over a period of time, various distresses can appear at the dam site such as uncontrolled seepage, sand boils, piping, cracks, settlements, sinkholes, slope failure, erosion, etc. The reasons for these distresses can be varied, ranging from design deficiencies, use of improper construction material, poor construction quality, inadequate maintenance and operation standards, etc. Various distresses and their probable causes are discussed in detail in Chapter 2. If the distress remains untreated they can lead to severe conditions leading to instabilities, and even failure or breach of the dam. Effective rehabilitation not only ensures continued safe operation of the dam but also contributes to the sustainability and resilience of water resource systems in the face of growing challenges.

The rehabilitation process may involve a range of interventions — from minor repairs and retrofitting to major structural modifications or even partial reconstruction. Unlike new construction, rehabilitation work must address the unique challenges of working with aging infrastructure, often while maintaining continued operation of the dam and reservoir. Several rehabilitation measures are available, however selecting the appropriate method considering the cause, nature and extent of distress, keeping in mind the site specific characteristics; is a skilled job. Also, for an earthen dam, due to heterogeneity and complex behavior of soil material there is no thumb rule of selecting the rehabilitation measures. Hence, finding the optimized and most suitable rehabilitation measure is a task and requires thorough studies backed by sound investigations at site.

7.1. REHABILITATION MEASURES FOR SEEPAGE MITIGATION

The primary objective of constructing any dam is to store and regulate water effectively. However, excessive seepage through the dam body or foundation leads to loss of stored water, which indirectly can be termed as a failure. More critically, seepage accompanied by internal erosion or migration of soil particles poses a serious threat to structural integrity and safety of the dam. If left unaddressed, such conditions can lead to piping, sinkholes, or even dam failure. Therefore, controlling seepage is of paramount importance and should be addressed promptly upon detection. A variety of remedial measures are available to improve water-tightness and reduce excessive seepage, both through the dam body and foundation. Various seepage mitigations measures that are commonly used are described below.

7.1.1. Seepage mitigation through dam foundation

Earthen dam can be constructed on any type of foundation, be it rock or soil. The quantum of seepage occurring through foundation depends on the pervious nature of foundation. Weathered rocks with joints, fractures are more prone to seepage. Joints and fractures act as conduits for water flow, especially when they are interconnected. In soil, the permeability is influenced by factors like soil texture, porosity and gradation. Poorly graded or gap graded alluvial soils, sands and gravelly type of soils are more prone to seepage. To prevent foundation seepage, generally earthen dams are provided with Cut-off-Trench (CoT) during construction. The CoT can be positive or partial depending upon permeability of the strata.

a) **Positive CoT:** A total seepage barrier that reaches the impermeable layer in foundation, intercepting all the pervious layers, is termed as positive CoT. In order to accomplish this, a trench is dug and then backfilled with compacted impervious soil material. It is recommended that base level of the cutoff should be at least one-fourth the maximum difference between reservoir and tail water elevations, but not less than 6 m, in order to guarantee an adequate seepage cutoff. If the foundation material beneath the cutoff is expected to be marginally impervious, the cutoff width should be greater. To ensure that impermeable backfill is properly placed and compacted in the CoT, the trench excavation must be kept dry at all times during construction.

b) **Partial CoT:** In this case, CoT does not extend through the full depth of pervious foundation layer, but only partially penetrates it. The purpose of partial CoT is to reduce seepage and uplift pressure by increasing the length of seepage path and reducing hydraulic gradients, in conditions where positive cutoff is not economically or technically feasible. Partial CoT is used when depth of pervious strata is very high to justify full cut off. To increase effectiveness, it is generally used in conjunction with other seepage control measures.

In addition to the above, seepage mitigation measures that can be adopted during rehabilitation of existing earthen dams are discussed below:

a) **Cut-off–wall:** Vertical cutoff wall is a useful technique for reducing under seepage through foundation when the pervious foundation is very deep. It provides an impermeable vertical curtain like barrier to reduce seepage through permeable foundation. For a new dam, the cut-off-wall is generally constructed beneath CoT. However, in case of an existing dam, depending on the feasibility, the cutoff may be constructed on either upstream or downstream side. In case of small or low height dams, cut-off-wall construction from crest through the dam body can also be thought of. The cut-off-wall can be constructed with various techniques viz. (i) Earth (soil–bentonite mix) backfilled slurry trench, (ii) Cement–bentonite or soil-cement-bentonite mix slurry trench, (iii) Concrete (Precast panels or cast in-situ RCC), (iv) Deep soil mixing, (v) Secant piling, (vi) Sheet piling, (vii) Jet grouted piles, etc. Choice of the construction technique depends on a number of factors such as site conditions, heterogeneity, permeability of sub-surface strata, geological features, depth of overburden, etc.

b) **Grouting:** For reduction of seepage through foundation, particularly in rocky strata, grouting is an effective method. The primary objective of foundation grouting is to fill joints, fractures, fissures, bedding planes, cavities or any other openings in the rock mass, thereby reducing its permeability and preventing excessive water loss. For grouting to be effective and to ensure its quality, consistency and long-term performance; it should be carried out at site by following strict compliances.

c) **Upstream impervious blanket:** To reduce seepage through foundation, an impermeable layer of compacted clay is laid at base of the dam from hearting/ core towards the reservoir. The thickness and length of clay blanket are crucial factors in determining its efficiency in seepage mitigation. While increasing thickness beyond a certain value has limited impact, increasing the length significantly reduces seepage. However, the thickness and length can be optimized based on the expected desired results in terms of reduced seepage discharge.

d) **Relief Wells:** These are vertical wells installed near downstream toe of the dam to permit ingress of seepage water and relieve uplift pressure. It is used as a seepage control measure when pervious foundation has artesian conditions (water under pressure) and hydraulic gradient beneath the dam is high enough to cause boiling at downstream toe. Relief wells intercept high-pressure seepage water in the foundation by providing path for safe exit of seepage water and thus reducing chance of piping. Moreover, where positive CoT is not feasible, relief wells are used to collect seepage water and relieve excess pore pressures.

7.1.2. Seepage mitigation through dam body

Seepage through the dam body typically results from use of unsuitable, pervious soil material for construction that lacks adequate clay content to achieve desired degree of imperviousness. Seepage through dam body also results from design deficiencies or poor construction quality. Inadequate compaction during construction can create weak zones, which allow excessive seepage and initiate internal erosion. Uncontrolled seepage, particularly when combined with insufficient or malfunctioning drainage systems, can lead to a rise in the phreatic line. This, in turn, causes saturation of the downstream slope, reducing shear strength of soil and potentially leading to slope instability. For seepage mitigation through dam body, following remedial measures are generally implemented.

a) **Impermeable layer/ membrane on upstream slope:** Seepage through dam body can be reduced by providing upstream clay or geo-membrane lining with suitable cover as protective layer (Fig. 7.1). This will reduce or prevent seepage water from entering the dam body and provide additional protection when the internal core alone is not sufficient. For clay lining, suitable clay material with low permeability and adequate plasticity should be used. In present times, geo-membrane which is a thin, flexible, synthetic sheet typically made of plastic polymers like HDPE, PVC or EPDM is used as lining instead of traditional clay lining. However, during installation of geo-membrane proper care should be taken such as preparing the surface, welding of joints, anchoring the edges in anchor trenches at crest and toe, etc. These measures will ensure continuity of the geomembrane lining without any gaps for passage of seepage water.

b) **Diaphragm wall:** Cement-bentonite, plastic or concrete diaphragm walls are commonly constructed through or adjacent to the dam’s core to serve as an impermeable barrier that blocks seepage flow. These walls effectively lower the phreatic line, helping to keep the downstream slope unsaturated and thereby enhancing its stability.

c) **Grouting in dam body:** Cement-bentonite or chemical grouting in the dam body will reduce overall permeability of the dam and thus reduce seepage discharge. Permeation grouting is

Figure 7.1 Upstream geomembrane lining

Figure 7.2 Grout holes for grouting through dam body

generally recommended in dam body where grout is injected into soil to fill voids and cracks, creating a cemented mass. This process increases soil strength and stiffness increasing overall stability of dam, as well as reducing its permeability. During grouting in dam body, care should be taken to control the pressure such that it ensures effective grout penetration without causing displacement or disturbance to the existing soil structure. In any case, hydraulic fracturing due to excessive pressure should be avoided and any change in volume of existing soil mass or bulging should be prevented. Also care should be taken to avoid reduction in efficiency of downstream filter arrangement (chimney and horizontal drain) due to grouting.

d) **Provision of drains and filters:** In all earthen dams, drains and filters are provided to collect seepage water and provide it a safe exit to the downstream toe-drain. While designed to have sufficient capacity for collecting estimated seepage, drains should also satisfy the filter criteria to prevent migration of fine soil particles of hearting zone into voids of casing material. Thus the filter should be designed to satisfy drainage as well as filter criteria which are seemingly conflicting. The grain size of filter material should be large enough to drain out seepage water but at the same time it should be small enough to prevent migration of hearting material. A properly designed filter in the dam body restricts occurrence of elevated phreatic surface and also prevents internal erosion which can eventually lead to piping.

Local slushiness or piping observed on the downstream slope indicates improper functioning of the drainage/ filter system. To reduce the elevation of phreatic surface, following remedial methods may be adopted:

i. Construction of vertical drains on downstream side joining the horizontal drain. This will drain the perched water on downstream side.
ii. Providing a new filter section with berm at appropriate level of downstream slope covering the entire affected area. This section will enable safe exit of seepage water while at the same time provide loading to the downstream slope increasing its stability.

e) **Seepage control above FRL:** Seepage occurring through the dam body above the Full Reservoir Level (FRL) is often caused by capillary action. This can be mitigated by extending the core vertically up to 1 m above the Maximum Water Level (MWL) through open excavation and constructing a continuous additional core above the existing one.

The above-mentioned measures are effective in controlling seepage through dam body and foundation, thereby reducing the risk of internal erosion and piping. Depending on site conditions, requirements and feasibility; a combination of multiple remedial measures can be implemented. However, such combinations should be based on thorough geotechnical investigations of the dam site.

7.2. REHABILITATION MEASURES FOR STRUCTURAL STABILITY

Structural stabilization for an earthen dam is essential to ensure its long-term safety and structural integrity. To address these challenges, a variety of rehabilitation techniques are employed. Some of the commonly used techniques are discussed below.

7.2.1. Upstream and downstream slope stabilization

The upstream and downstream slopes of a dam are vulnerable to instability due to several factors such as seepage, rapid drawdown, rainfall infiltration, poor compaction and seismic activity. To mitigate these risks, various slope stabilization techniques are applied, tailored to specific site conditions and underlying causes of instability.

a) **Flattening of slope:** This method involves reducing the slope angle and increasing the number and width of berms. Flattening of slope decreases destabilizing forces, thereby increasing the Factor of Safety (FS) against slope failure.
b) **Improvement of overall shear strength:** The slopes can be stabilized by grouting with sand-cement mixture or any other suitable grout mix, which enhances shear strength of soil material and improves overall stability.
c) **Soil reinforcement:** Geosynthetic materials such as geogrids or geocells can be used to reinforce weak soils in the slopes. These materials provide additional tensile strength and help improve slope stability by interaction with soil material.
d) **Retaining structures:** In areas where slope flattening is not feasible due to space constraints or where the original slope profile must be retained, structural supports such as gabion walls, masonry retaining walls, or reinforced earth walls can be constructed at the toe to provide lateral support and ensure slope stability. Gabion walls or retaining walls may be constructed at toe of the slope to provide extra support. These structures help resist lateral earth pressures, particularly in steep or critical areas.
e) **Provision of shear key:** For weak foundation layers, slope stability can be enhanced by providing a shear key in foundation adjacent to the dam toe. The shear key acts as a barrier that increases shear resistance and reduces the risk of deep-seated slope failures. However, its design and construction requires careful attention. Depth of the shear key

should be sufficient to penetrate and intercept weak or soft foundation layers prone to sliding. Additionally, the width and depth must be adequate to resist the anticipated sliding forces. For foundations with weak or soft strata, a shear key constructed near the downstream toe can intercept potential sliding planes. The shear key should extend through full depth of weak layer and should be adequately designed to resist expected sliding forces.

f) **Horizontal filters:** To facilitate rapid dissipation of pore water pressure developed during drawdown, horizontal filters can be installed within the upstream slope.

g) **Installation of drainage system:** To lower the phreatic line and permit safe passage of seepage water, additional drains can be installed on the downstream side. The drains will also help in dissipating excess pore pressures developed due to elevated phreatic line in downstream zone of the dam.

h) **Surface drainage system:** Rainwater infiltration can saturate the dam body, reducing shear strength of soil. To prevent this, an effective surface drainage system should be installed to collect and safely drain the rainwater to the toe drain, minimizing infiltration into the dam body.

Among all the above remedial measures, flattening of slopes is the most common and cost effective remedial measure. It requires only additional quantity of suitable soil material. However, flattening of slope increases overall base width of the dam, which may not be feasible at locations where space is limited. In such cases, a combination of slope flattening and other stabilization measures such as soil reinforcement, retaining structures, or drainage improvements, can be adopted.

At this point, it is important to emphasize that when slope flattening is carried out using additional earth fill, proper bonding between the existing slope and newly placed soil material must be ensured to maintain structural integrity and to avoid potential slip surfaces at the interface. To achieve effective integration, following precautions should be observed:

- Loose, soft or organic materials from the slopes should be removed.
- Existing slope surface should be scraped. Scraping helps to roughen the surface, which improves mechanical interlock between the existing soil and additional fill.
- In some cases, benching (stepped excavation) is done along with scraping for better stability.
- Suitable fill material (same or better than existing one) should be used.
- Proper compaction in layers should be carried out maintaining optimum moisture content.

- Field supervision, quality control and compaction testing should be carried out to ensure that design specifications are met during construction.
- After earthwork, slope should be protected with proper slope protection works as discussed in the subsequent para.

Figure 7.3 Soil Compaction

7.2.2. Slope surface protection measures

The upstream slope of an earthen dam is exposed to wave action, fluctuating water levels and potential erosion, making protection essential. Whereas, the downstream slope surface is primarily vulnerable to rainfall-induced erosion, surface runoff and seepage related instability. To protect slopes from the effects of these conditions, various measures can be implemented as given below.

a) **Upstream slope:** For upstream slope the most common protection method is stone pitching, which is a layer of hand-placed or dumped rock laid over a filter layer to resist wave impact and prevent soil erosion. Beneath the pitching, a filter layer or geotextile is provided to prevent migration of fine particles from the embankment into the pitching. To repair any disturbed slope protection following measures are adopted:

- Partial repair with same material as in original dam construction
- Enlarging the width of upstream protection layer by dumping selected rockfill along the entire upstream slope

- Replacement of affected slope protection

b) Downstream slope: One of the most common protection methods for downstream slope is turfing or grass cover, which helps bind the soil, reduce surface erosion and improve slope appearance. For steeper or erosion-prone areas, jute or coir matting is often used during vegetation establishment. In cases where surface runoff is more intense, stone pitching or riprap may be applied in localized areas to prevent gully formation. Additionally, well-designed surface drainage systems, including cross drains, longitudinal drains, and toe drains, are essential to safely collect and divert rainwater away from the slope. These measures collectively enhance the slope's resistance to erosion and ensure long-term stability and safety of the dam.

When the downstream slope surface shows signs of distresses such as surface erosion, gully formation, thick vegetation, animal borrows, etc. timely remedial measures are critical to restore stability and prevent further damage.

In addition to above, no vegetation in the form of trees, shrubs should allowed to be grown on the slope. Such vegetation if already present should be removed carefully along with their root system. If not removed and disposed off properly away from the dam; the decaying root system will create voids in the dam body, diminishing overall density of the casing material. If surface erosion or rain gullies are formed, the damaged areas should be cleaned, backfilled with suitable earth material and compacted in layers. Also proper protection measures in form of turfing or stone pitching should be installed to protect the repaired surface.

Figure 7.4 Installation of stone pitching for upstream slope protection

Figure 7.5 Downstream slope protection

7.2.3. Filling of fractures and cavities

Fractures and cavities in an earthen dam are serious signs of distress that may arise due to differential settlement, desiccation cracking, animal burrows, internal erosion (piping), or seismic activity. Use of poor quality construction material, improper compaction and inefficient drainage system are main reasons behind this. If left unattended, these defects can compromise the dam’s structural integrity. Following methods are generally employed to treat the cavities or cracks formed in dam body:

- **Initial assessment:** The extent and depth of cracks or cavities should first be assessed through careful visual inspection to formulate appropriate repair strategy.
- **Removal of loose material:** Any loose, eroded, or unstable material within the cracks or cavities should be carefully excavated to expose sound and stable surrounding soil.
- **Backfilling and compaction:** The cleaned cavity should be backfilled with suitable soil material by placing, spreading, and compacting in thin layers with proper moisture control to ensure strength and continuity with the existing dam body.
- **Crack sealing with clay slurry:** Cracks should be flushed with pressurized water to remove debris and ensure a clean surface. They should then be filled with a thick slurry of natural clayey soil. After the slurry dries, the surface should be sealed by trenching to a depth of 0.3 to 0.6 m, followed by filling and re-compacting with appropriate soil material.
- **Sinkhole treatment:** Any sinkholes observed in the dam body should be backfilled using a dense mixture of well-graded materials such as sand, gravel, pebbles, boulders, and rock fragments. Prior to filling, the area should be secured—preferably by installing sheet

piles around the sinkhole perimeter. The backfill must be placed and compacted thoroughly in accordance with design specifications.

7.2.4. Raising of dam crest

Raising of a dam's crest refers to increasing height of the dam, typically to enhance flood storage capacity, improve freeboard (safety margin above maximum water level), comply with updated safety or design standards and to account for settlement or degradation of the original crest over time. This process is common in rehabilitation or modernization of aging earthen dams. Raising the dam height requires modification and strengthening of both the upstream and downstream slopes. The modified dam cross-section must be thoroughly assessed to ensure stability under new conditions. Materials used for raising should be compatible with the existing dam zones to maintain structural integrity and prevent interface problems.

During upstream slope strengthening, special precautions must be taken, including careful removal of the slope protection layer and addressing any sediment deposits that may be present. The existing crest and slopes should be scarified to ensure a good bond between old and new materials. Additionally, the core and filter zones must be extended vertically to correspond with the new crest elevation, maintaining continuity in seepage control and drainage.

7.2.5. Restoration of bond between junctions

Loss of bonding between earthen and concrete structures (ex. at the interface between embankment and spillway, outlet conduit, or retaining walls) is a critical and most commonly occurring defect in earthen dams. It can result in seepage, piping, settlement, or even structural instability, especially during high reservoir levels or seismic events. Repairs for restoring the bonds between earthen and concrete/ masonry structures can be carried out in following manner:

- Excavation and thorough cleaning of affected interface to remove all loose materials, vegetation, and debris
- Sealing of interface using appropriate techniques such as cement-bentonite or epoxy grouting, bituminous coatings, to restore bonding
- Installation of properly designed filter and transition zone between the earthfill and concrete surface using graded sand, gravel, and drainage material to prevent soil migration and internal erosion
- The adjacent earth fill must be reconstructed in thin layers and compacted using proper moisture content, with benches or key trenches formed to ensure effective interlocking

- Improvements to the drainage system at interfaces help in safe management and discharge of seepage water. Without proper drainage, the seepage can become concentrated, increasing the risk of piping and structural failure

7.3. REHABILITATION MEASURES FOR SEISMIC RESTORATION

To ensure seismic stability of earthen dams is critical, especially in seismically active zones, as these structures are vulnerable to cracking, slope failure, liquefaction, and excessive deformation during earthquakes. Remedial measures for seismic retrofitting of dams focus on strengthening the dam to resist seismic forces by improving material properties, enhancing drainage and ensuring safe dissipation of excess pore pressure generated during seismic shaking. Hence, while designing the dam its stability under seismic force should be assessed, along with stability under static forces. Required modification in term of slope flattening, constructing stabilizing berms or buttresses to resist lateral spreading and reinforcing critical zones with geosynthetics to improve confinement and shear strength, etc. can be executed in the dam for ensuring stability during earthquake.

Another phenomenon related to earthquake is liquefaction in which saturated, loose, cohesionless soil within the dam body or foundation loses its strength and behaves like a liquid. To mitigate this risk, remedial measures focus primarily on improving the strength and drainage characteristics of vulnerable zones comprising of potentially liquefiable soils. Common approaches include removing and replacing liquefiable soils with well-compacted, non-liquefiable materials, or applying ground improvement techniques such as vibro-compaction, stone columns, dynamic compaction, or soil mixing to densify and stabilize the existing soil. Additionally, installing vertical or horizontal drainage systems like sand drains or relief wells helps dissipate excess pore water pressure generated during seismic shaking.

This chapter of the technical memorandum outlines the commonly used remedial measures to address various problems typically encountered in earthen dams. However, sometimes when severe issues crop up at site, adopting rehabilitation measures do not significantly help in improving safety of the dam. In such cases reconstruction of the entire damaged portion is the most viable solution instead of implementing remedial measures.

7.4. SELECTION OF SUITABLE REHABILITATION MEASURES

The prerequisite for selecting site-specific remedial measures is having comprehensive information about existing material properties of the dam body and foundation, as well as the current dam cross-section. Additional data such as design drawings, geological and foundation stratification, information on existing Cut-off-Trench (CoT) and curtain grouting, in-situ properties recorded during construction and records of any previously implemented rehabilitation measures should be reviewed. The properties of existing soil material in the dam body are typically determined by geotechnical investigations conducted on-site. These investigations include in-situ tests such as Standard Penetration Test (SPT) and permeability tests, as well as laboratory testing of disturbed and undisturbed soil samples collected from boreholes as explained in Chapter 3. To accurately determine the existing dam cross-section; design drawings, borehole data and survey records are often correlated and used in further analyses.

As already discussed, a number of remedial measures are available but it is often a task to choose most suitable remedial measures for a specific problem. Various studies as discussed in previous chapters viz. Chapter 4 (Seepage analysis), Chapter 5 (Structural safety assessment) and Chapter 6 (Dynamic stability assessment) are essential to be carried out to formulate the most optimum remedial measure. Most often, remedial measures adopted for seepage mitigation may not be sufficient to improve structural health of the dam and vice versa. As such a combination of methods may be required to be adopted at site to mitigate seepage as well as improve structural health of the dam.

CHAPTER - 8

INSTRUMENTATION AND MONITORING

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8. GENERAL

An effective instrumentation and monitoring program is a cornerstone of dam safety management. By equipping an earthen dam with appropriate instruments, engineers can quantitatively monitor key indicators of dam performance and detect evolving problems early. This is critical for preventing failures that could endanger downstream communities and infrastructure. Instrumentation provides data to verify design assumptions, observe the dam's behavior during construction/ filling, and inform maintenance or rehabilitation decisions. In the Indian context, the Central Water Commission (CWC) and Bureau of Indian Standards (BIS) have established comprehensive guidelines and codes for dam instrumentation (ex. IS 7436 (Part 1): 1993, Guide for Types of Measurements for Earth and Rockfill Dams). Adhering to these standards ensures that instrumentation is planned and installed correctly and that the data collected can be relied upon for dam safety assessments. This chapter covers the basics of instrumentation and monitoring for earthen dams, focusing on practical field applications in seepage monitoring, pore pressure measurement, slope stability, settlement, and internal erosion detection, along with guidance on data acquisition and interpretation.

8.1. KEY PARAMETERS TO MONITOR IN EARTHEN DAMS

Instrumentation plans should target the critical aspects of an embankment dam's performance. According to standards and dam safety guidelines, the principal parameters to monitor include: pore water pressures, seepage flows, internal movements (deformations), external movements (settlement and surface displacement), and indicators of internal erosion. Each of these is briefly introduced below, along with its significance for dam safety.

8.1.1. Seepage

All earthen dams experience some seepage through the dam body and foundation. Controlled seepage is usually harmless, but increasing or muddy seepage is an alarming signal. Measuring the rate and clarity of seepage water helps to indicate if internal erosion (piping) or clogging of drains is occurring. Excessive or concentrated seepage can lead to loss of material from the dam (internal erosion) or uplift pressures in the foundation.

8.1.2. Pore water pressure

Pore water pressure or piezometric level within the dam and foundation is arguably the most important measurement in embankment dams. It defines the seepage regime and saturation line, directly affecting effective stress in the soil. High pore pressures reduce shear strength and can trigger slope instability or even liquefaction in extreme cases. Monitoring pore pressure allows engineers to verify that drainage systems are performing and that the phreatic line remains within design limits, particularly during first filling and rapid drawdown events. It also helps identify potential piping zones and evaluate the effectiveness of cutoffs or drainage provisions.

8.1.3. Slope stability (deformations)

The stability of slopes (upstream and downstream) can be evaluated by measuring lateral and vertical deformations. Horizontal movements might indicate shear deformations or incipient sliding surfaces, while vertical movements (settlements or heave) reflect consolidation or movement of material. Movements within normal, expected ranges confirm the dam's stability, whereas accelerating or unexpected deformations may foreshadow slope failure. Thus, instruments that track the internal deformation profile of an embankment dam and any differential movements are essential for slope stability monitoring.

8.1.4. Settlement

Earthen dams undergo settlement during and after construction as the fill and foundation consolidate. Uniform, gradual settlement is expected; however, differential settlement (uneven or localized sinking) can cause cracking, which in turn may lead to concentrated leakage or internal erosion. Monitoring settlement of the dam crest, slopes, and internal zones over time ensures that total settlements are within design predictions and helps detect any unusual distortions of the structure. Settlement data is also vital when raising dams or performing rehabilitation, to gauge how the existing structure is behaving.

8.1.5. Internal erosion (piping indicators)

Internal erosion refers to the removal of soil particles by seepage flow (piping), often through cracks or poorly compacted zones. It can progress unseen inside the dam until failure is imminent. Because there is no single "piping meter," internal erosion must be inferred from other measurements – increases in seepage flow rate, appearance of turbidity (muddy water) in seepage, drops in internal piezometric levels if a cavity forms, or abnormal localized settlements. Early detection is key and instrumentation is used to monitor these proxy indicators. In recent years, techniques like fiber-optic temperature sensing have also been used to detect seepage-induced cooling or warming in the dam body, but traditional instruments remain the main tools in routine dam safety monitoring.

Each of the above parameters is linked with specific instruments and observation methods, discussed in the next para. A well-designed instrumentation scheme will cover all these aspects to provide a complete picture of the dam's health.

8.2. INSTRUMENTATION FOR SEEPAGE MONITORING

Seepage measurement in earthen dams is typically done by collecting and measuring the water that emerges from drainage systems or downstream areas. Seepage weirs (V-notch or rectangular weirs) are commonly installed at the outlet of drainage galleries, toe drains, or relief wells to measure flow rates. Water passing over the notched weir is observed and recorded – by measuring head (water depth) over the notch, engineers can calculate the discharge using standard formulas. IS codes recommend V-notch weirs with a free-fall condition and tranquil approach flow for accuracy. In small dams or for simplicity, calibrated containers or flumes can also be used to gauge seepage flow. Indian standards (IS 7436-1:1993) recommend using V-notch or rectangular weirs at suitable points on drainage outlets to measure seepage. The head over the notch is observed and the discharge is calculated by an appropriate formula.

In addition to flow rate, seepage water quality should be observed – the presence of sediments ("cloudy" water) may indicate internal erosion. Simple instruments like turbidity meters or even a daily visual check of a container can track changes in seepage clarity. CWC guidelines emphasize that any sudden increase in seepage rate at constant reservoir level, or new seepage emerging at the toe, warrants immediate investigation.

To ensure standardized monitoring, BIS has issued IS 14750:2000, a code of practice for installation and observation of seepage measuring devices in dams. This standard covers good practices for constructing measuring weirs, collection systems, and maintaining records of seepage. Key points include providing proper approach channels to weirs, keeping weir plates clean, and measuring at consistent time intervals. It’s important for field engineers to correlate seepage readings with reservoir levels and rainfall. For example, higher seepage during reservoir surcharges is normal, but if seepage does not recede as reservoir is drawn down, it could indicate a problem. Multiple measuring points may be installed to isolate seepage from different zones (ex. under the core vs. through abutments). The data from seepage monitoring, combined with internal piezometer data, gives a powerful indication of the dam’s seepage performance and any evolving internal erosion. In summary, seepage monitoring procedures involve regularly measuring flow at weirs (ex. daily to weekly in monsoon, monthly in dry season), noting any discoloration or changes, and comparing trends over time. According to CWC’s instrumentation guidelines, all observed seepage should be recorded in an inspection register and any anomaly should trigger a detailed inspection of the dam.

8.3. INSTRUMENTATION FOR PORE PRESSURE

Piezometers are the primary instrument for pore pressure monitoring in earthen dams. A piezometer generally consists of a sensor (or open tube) placed at the desired depth within the embankment or foundation, which measures the water pressure (pore pressure) at that point. By installing piezometers at various locations and elevations, engineers can map the phreatic surface (line of saturation) through the dam and see how it changes with reservoir level and time. Indian Standard IS 7436 (Part 1) highlights that pore pressure measurement enables the seepage pattern to be known and allows assessment of internal erosion and slope stability concerns. Piezometer data thus serves multiple purposes– indicating potentially dangerous conditions (excess pressures), verifying design assumptions about drainage, and evaluating effectiveness of seepage control measures like filters or cut-offs.

Proper installation of piezometers in earth fills and foundations provides quantitative data on pore pressure distribution and variation with time. It helps reveal the seepage pattern, identify zones of potential piping, and check the effectiveness of seepage control measures. The data can indicate dangerous conditions affecting stability and allows post-construction behavior to be monitored. There are several types of piezometers used in embankment dams, each with its own design and application considerations:

8.3.1. Standpipe (open tube) piezometers

It is a simple, reliable device consisting of a porous filter tip connected to a riser pipe that extends to the surface. The standpipe fills with water up to the piezometric elevation at the tip location. The water level is measured with a dip tape or automatic logger to indicate pore pressure. Standpipes are rugged and have long successful performance records. They require manual readings and can be slow to respond to rapid pressure changes (because water must flow in/out of the tube). Proper installation (including saturating the filter and backfilling with a sand filter and bentonite seal above) is crucial to avoid creating unintended drainage paths. IS 7356 (Part 1):2002 is a dedicated code of practice for installing and maintaining porous tube (standpipe) piezometers in earth and rockfill dams. It provides guidelines on borehole drilling, filter pack placement, and piezometer protection.

8.3.2. Hydraulic (twin tube) piezometers

These devices use twin fluid-filled tubes running from a diaphragm tip to a hydraulic gauge at the surface. The water pressure at the tip is transmitted through an incompressible fluid to a Bourdon gauge or pressure transducer. They are advantageous where direct access is difficult, as the calibrated part (the gauge) remains at the surface. IS 7356 (Part 2):2003 covers the installation of twin-tube hydraulic piezometers. Hydraulic piezometers respond faster than standpipes and can be read remotely if equipped with transducers, but they are sensitive to any leaks or temperature changes in the tubing.

8.3.3. Pneumatic piezometers

These use a gas (usually nitrogen or air) system. A pneumatic piezometer tip contains a flexible diaphragm; to take reading, gas is pumped down one tube until it slightly deforms the diaphragm, at which point the gas pressure (indicated by a gauge at the surface) equals the water pressure at the tip. Pneumatic piezometers have no internal fluid that can drain, so they interfere less with construction and are often used in earth fill during placement. However, they require careful calibration and tubing of adequate length/ diameter; they can also suffer delays if tubing is long. Pneumatic types are not as commonly used now as vibrating wire systems.

8.3.4. Vibrating wire piezometer

These modern electronic piezometers have a tensioned wire inside a fluid-filled housing; the pore pressure changes tension in the wire, and the instrument outputs a frequency signal that can be read with a portable readout or logged automatically. Vibrating wire piezometers are highly sensitive, can measure negative pressures, and offer remote reading capability. They require electrical cables for data transmission. IS 12949:1990 covers the installation of electrical (vibrating wire) piezometer cells in earthen dams. Vibrating wire piezometers are particularly suitable for impervious or clayey zones where traditional standpipes may respond slowly, and they are often used when continuous or frequent monitoring is needed (ex. during first reservoir filling or in high-risk dams). They do, however, require good lightning protection and calibration checks.

Common instruments for pore-water pressure in embankments include standpipe (open tube) piezometers, pneumatic piezometers, and vibrating wire piezometers. For example, standpipes are simple and reliable but require longer time for stable reading and can be prone to clogging, whereas vibrating wire piezometers allow rapid readings with electronic data capture. Regardless of type, piezometers should be strategically located within the dam cross-section. Indian and international guidelines recommend placing them at critical points: near the upstream toe (to monitor uplift in the foundation), within or just downstream of the core (to track the internal phreatic line), and near the downstream toe or drain exit (to ensure the exit gradients are not excessive). They are often installed in clusters at different elevations along a vertical line to profile the pore pressure distribution through the height of the dam. Care must be taken to install piezometer tips within the zones they are intended to measure (for instance, within the clay core or in foundation strata) and not inadvertently short-circuiting clay core layers. This is addressed by proper grouting and sealing around the instrument borehole per IS 7356 procedures.

8.4. INSTRUMENTATION FOR SLOPE STABILITY (LATERAL DEFORMATION)

To monitor slope stability, deformation is measured within the embankment and along its slopes. The primary instrument for internal horizontal deformation is the inclinometer (also known as a slope indicator). An inclinometer system consists of a grooved casing installed vertically (or along a known alignment) into the embankment fill or abutment, and a sensitive probe that is lowered inside the casing to record the tilt (angle) at various depths. By taking readings over time, the lateral deflection profile can be determined, showing how much and where the dam might be shifting laterally. Inclinometers can detect very small movements (in the order of millimeters) and are crucial for early warning of shear deformations or instability planes developing in the dam or foundation.

Installation of inclinometers in earthen dams typically occurs during or post construction by drilling a borehole and inserting the inclinometer casing, which is then backfilled (often with a sand-cement grout mix that is flexible enough to deform with the ground). The casing has longitudinal grooves that ensure the probe stays oriented. According to CWC's Guidelines for Instrumentation of Large Dams, inclinometers should be placed in areas of suspected movement – for example, in the downstream shoulder near the dam axis where the highest embankment section is, or near abutment contacts if there is potential for differential movement. More than one inclinometer might be installed: a common approach is to have one at the dam's maximum section and others at regular distances along the length (ex. a few hundred meters apart) to cover different profiles.

Reading an inclinometer involves lowering the probe to bottom of the casing and recording tilt as the probe is pulled upward in increments (typically 0.5 m). The tilt in two orthogonal directions (longitudinal and transverse to the casing grooves) is measured so that the vector of movement can be understood. Data are then converted to lateral displacement versus depth. Modern inclinometers may use digital MEMS sensors and can even be left in place for continuous monitoring, but the traditional manual approach is common in practice for periodic checks. The key sign of trouble is if one notices a localized increase in displacement (a kink in the deformation profile) that grows with time – this may indicate a developing slip surface in the embankment or foundation.

Other instruments complement inclinometers for slope stability monitoring: extensometers can directly measure displacement across a crack or joint (though more common in concrete structures), and tiltmeters can measure rotation of structures like retaining walls or steep abutments. On an earthen dam, surface deformations are often tracked with survey markers (see next para on settlement). If an inclinometer is not available, a cheaper method to gauge lateral movement is to install alignment pins or collimation targets along the slope and periodically survey them, though this is less precise.

It is worth noting that significant lateral movement in an embankment is usually accompanied by vertical movement. Thus, a comprehensive deformation monitoring scheme will involve both inclinometers (for horizontal movement) and settlement monitoring (for vertical movement). Indian Standards like IS 7436-1:1993 advise that horizontal and vertical displacements at the surface of the dam should be measured, for example by fixed monuments and periodic surveying. Any acceleration in horizontal displacement rates is a serious concern – as a rule of thumb, if movement vs. time plots show an increasing trend, engineers should investigate and perhaps lower the reservoir until stability is confirmed.

8.5. INSTRUMENTATION FOR SETTLEMENT (VERTICAL DEFORMATION)

Settlement monitoring addresses how much the dam and its foundation are compressing or subsiding over time. A number of instruments and techniques are employed for this purpose:

8.5.1. Settlement points/ monuments

These are fixed reference points on the dam (often on the crest or berms) that can be surveyed periodically with leveling equipment or total stations. In their simplest form, they may be metal pins or benchmarks embedded in concrete blocks. The change in elevation of each point over time indicates how much settlement has occurred at that location. A series of such points along the crest can reveal differential settlement (one point settling more than another). This method falls under geodetic monitoring and is a standard practice recommended by IS codes for all dams. Typically, surveys are done more frequently in the first year after construction (ex. monthly or quarterly) and annually thereafter.

8.5.2. Deep settlement gauges (internal settlement devices)

These instruments measure settlement at depth within the embankment or at the foundation interface, which is valuable for understanding how the core or foundation is compressing. One common type is the cross-arm settlement device (also called settlement profiler). As described in IS 7500:2000 “Code of practice for installation and observation of cross arms for measurement of internal vertical movement in earth dams”, this system involves a vertical pipe with sliding cross-arms at various depths that extend out to the pipe wall. As the embankment settles, each cross-arm (initially aligned with reference marks) moves down with the soil. A sensing probe or tape is inserted in the pipe to detect new positions of the cross-arms, thereby giving the settlement at those depths. This technique allows engineers to see how settlement is distributed through height of the dam, which is important for identifying if excessive compression is occurring in a particular layer or zone.

8.5.3. Foundation settlement plates/ base plates

To measure settlement of foundation under the dam, settlement plates or base plates can be placed at foundation level before construction. These are typically steel plates or discs that serve as reference points; as the foundation soils compress, the plates move downward. Their positions can be tracked via riser pipes or by probing from the surface. IS 8226:1976 is a code of practice for installation of base plates for foundation settlement monitoring. For example, a base plate might have a pipe extending to the dam surface where precise leveling can determine how much the plate (and thus foundation) has moved. Significant foundation settlement can affect the dam’s slope or cause cracks, so these measurements are crucial if the dam is on soft or compressible ground.

8.5.4. Hydrostatic settlement gauges

An advanced method uses the principle of communicating vessels – a series of liquid-filled tubes and sensors that measure differential elevation. Known as hydrostatic leveling systems (HLS), they can provide continuous monitoring of settlement between points (ex. between the dam crest and a fixed point on the abutment). Vibrating wire settlement cells also exist, where a fluid reservoir and a pressure sensor are used to infer elevation changes. These are typically employed in large dams or where continuous remote monitoring of settlement is needed.

8.5.5. Magnetic settlement devices

These devices involve magnetic rings installed around a central access pipe at various depths. A portable probe with a magnet sensor is lowered, and it detects the elevation of each ring. As the soil settles, the rings move down, and their positions indicate the settlement profile. This is conceptually similar to the cross-arm device but uses magnetic sensing.

Which method to use depends on the dam size and importance. For most training purposes, surveyed settlement points and cross-arm gauges are emphasized, as they are widely used and straightforward to interpret. According to instrumentation guidelines, vertical movement readings from internal devices like cross-arms should be taken at regular intervals (ex. monthly during early operation) and then every few years once deformations stabilize. Surface settlement points may be surveyed annually. All measurements are compared against predicted settlement from design. A rule of thumb is that postconstruction settlement should be a small fraction of the dam height (a few percent at most, distributed over years). If measured settlement exceeds expectations or if uneven settlement is causing cracks or offsets (for instance, settlement at the center of the dam much more than at the abutments), it should prompt further analysis and remedial measures (such as filling cracks, relief wells if due to consolidation of saturated zones, etc.).

Settlement often correlates with reservoir loading; some additional settlement can occur when the reservoir is filled (due to increased effective stress in the core and foundation).

Instruments should capture this, and the dam’s design should have accounted for it. Ensure that any instruments like cross-arm pipes are protected from construction equipment and that readings are corrected for any reference movement. A stable or slowing rate of settlement over time is usually a good sign, whereas ongoing or accelerating settlement might indicate material collapse (possibly from internal erosion or wetting of collapsible soils) and must be investigated.

8.6. INTERNAL EROSION DETECTION AND MONITORING

As noted earlier, internal erosion (piping) is often a hidden threat in earthen dams that instrumentation can help uncover indirectly. There are a few strategies and tools to specifically target internal erosion detection:

8.6.1. Turbidity and flow monitors

The clearest indication of internal erosion is sediment appearing in the seepage water. Installing a turbidity meter or simply regularly sampling the seepage flow (from weirs or relief wells) for sediment content can provide early warning. Some dams have automated turbidity sensors or even sediment traps to quantify how much material is being carried out. A rising trend in sediment load, combined with increasing seepage rate, is a red flag for piping. As part of instrumentation, a weir box can be fitted with a small sample port and a turbidity sensor that logs water clarity over time.

8.6.2. Piezometer patterns

Internal erosion can create voids or pathways that alter the pore pressure distribution. For example, if a concentrated leak develops, nearby piezometers might show a drop in pressure (since water is now escaping, local head is reduced) or an atypical response to reservoir level changes. By closely analyzing piezometric data, engineers may detect anomalies indicative of internal erosion. One recommended practice is to install piezometers near potential problem zones such as along suspected crack lines (ex. interface between core and abutment). If one piezometer consistently reads lower pressure than others in the section, it could be located in a preferential seepage path. Conversely, a sudden increase in a piezometer reading might mean a filter has clogged downstream, causing a backup of pressure – potentially due to piping material accumulating. Thus, comparing multiple piezometers in a cross-section is important. IS guidelines suggest locating them in pairs (upstream and downstream of core) so that gradients can be computed.

8.6.3. Drain outlet monitoring

Often dams have toe drains or blanket drains emptying into an inspection gallery or a drain outlet. Equipping these with simple instruments like flowmeters and pressure gauges can help. For instance, if a drain gets partially blocked by eroded material, the water might start to back up – a pressure gauge on the drain outlet could show increased pressure that normally should be near zero. Some advanced systems use acoustic devices listening for changes in flow noise that might indicate air entry from a developing pipe, though these are not yet common.

8.6.4. Geophysical and advanced methods

Modern dam safety programs sometimes include geophysical surveys (like electrical resistivity tomography or self-potential methods) to identify zones of higher permeability or flowing water inside the dam, which could indicate internal erosion. Fiberoptic cables running through the dam can detect temperature changes caused by seepage (because seepage water temperature is often different from the soil). These techniques, while beyond the basics, can complement classical instruments. The CWC has noted in its guidelines that emerging technologies can enhance internal erosion detection, but regular seepage and piezometric observations remain the first line of defense.

In summary, instrumentation for internal erosion is about integrating the data from seepage measurements, piezometers, and visual inspections. Field engineers should be trained to recognize subtle changes: for example, a small but steady increase in seepage over weeks, or a drop in a downstream piezometer reading not explained by reservoir decline, may be the first sign of trouble. All such observations should trigger closer inspection of the dam (walkover surveys to find any new wet spots or sinkholes on the downstream face, etc.). Indian dam safety guidelines emphasize developing alarm criteria for internal erosion indicators – for instance, specifying that if seepage flow exceeds a certain threshold (based on historical max) or if any muddiness is observed, it should be reported immediately and expert analysis sought.

8.7. TYPICAL INSTRUMENTATION LAYOUT IN AN EARTHEN DAM

To design an instrumentation scheme, one must decide which instruments to install, how many, and where. IS 7436-1:1993 (Guidelines for choice and location of instruments in embankment dams) provides criteria: instruments should be placed to monitor the zones of maximum stress/ strain, likely seepage paths, and any suspected weak points. This usually translates to instrumenting the maximum cross-section of the dam (the deepest valley section where the dam is highest) as a primary section, and additional sections towards the abutments especially if the geology or stratigraphy changes. For a long dam, sections every few hundred meters may be instrumented. Longitudinally, instruments like settlement monuments might be spaced along the crest.

In the cross-section view, a typical layout might include: piezometers within the core (at multiple elevations), piezometers in the foundation (upstream and downstream of any cutoff, to measure uplift), an inclinometer near the downstream toe extending into the foundation, earth pressure cells at the base of the core or beneath the downstream slope (to measure total stresses), and surface monuments on the crest and downstream slope (for settlement and alignment). The downstream toe area will have a lined drainage collection or toe drain leading to a V-notch weir for seepage measurement. An example schematic is shown in Fig. 8.1, illustrating a cross-section of an embankment dam with some typical instruments in place.

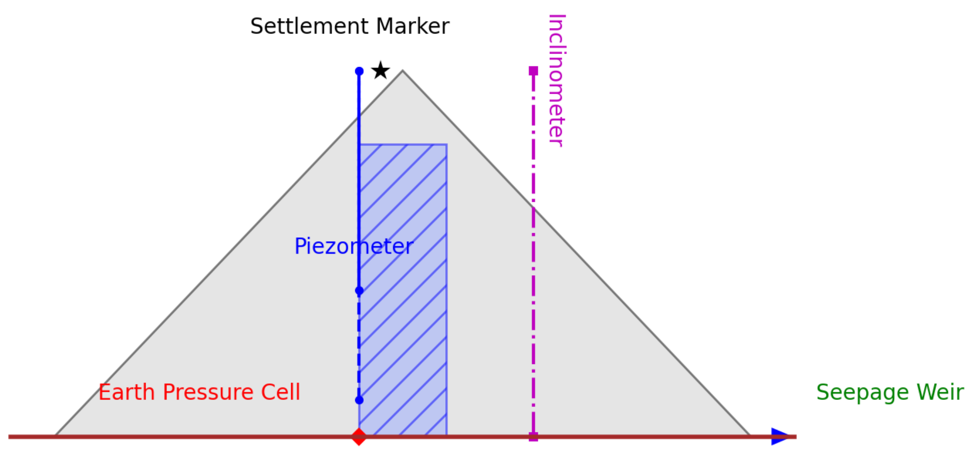

Figure 8.1 Schematic cross-section of embankment dam with typical instrumentation

Piezometers (blue) are installed in the impervious core and foundation to measure pore pressure. An inclinometer (magenta) near the downstream side monitors lateral deformation. A settlement marker (★) is set on the crest for periodic leveling. An earth pressure cell (red) at the base of the core measures total stress. Seepage is collected at the toe and measured via a V-notch weir (green). The ground line is shown in brown. Such an arrangement is in line with IS/ CWC guidelines, focusing instruments at critical sections of the dam.

In designing the layout, practical considerations are taken into account: accessibility (instrument locations should be reachable for readings or maintenance, ex. not too far down a steep slope unless remote reading is available), protection (installations must be protected from construction damage and from subsequent erosion or traffic – usually piezometer risers are enclosed in lockable boxes or manholes), and redundancy (important parameters may be measured by more than one instrument for reliability). A robust design also considers how data will be collected – for example, routing cables from VW piezometers and pressure cells to a central readout unit or terminal house on the dam.

CWC's Guidelines for Instrumentation of Large Dams (2018) recommend that instrumentation layouts be included in the initial dam design drawings and updated as needed during construction. For existing dams, an instrumentation retrofit should similarly be planned out in drawings and verified on site. The guideline also suggests grouping instruments by purpose: for instance, having a dedicated "pore pressure section," a "deformation section," etc., although in practice many instruments overlap in usage. Finally, when laying out instruments, engineers must plan for a baseline reading (initial readings) as soon as instruments are in place (before reservoir filling) to have a point of comparison. They should also ensure reference benchmarks (for surveys, etc.) are established on stable ground away from the dam.

8.8. DATA ACQUISITION, INTERPRETATION, AND DECISION-MAKING

Installing good instruments is only half the battle – timely data collection and proper interpretation are what turn the instrument readings into useful dam safety information. Field engineers and dam safety staff should establish clear procedures for how often to take readings, how to record and analyze them, and how to respond to the findings:

8.8.1. Reading frequency

This depends on the instrument type and the dam's situation. During construction and first filling of the reservoir, readings are typically frequent (ex. piezometers weekly or even more often if filling rapidly; deformation measurements monthly). IS and international guidelines often provide tables of recommended frequencies. For example, the US Bureau of Reclamation suggests piezometer readings twice monthly during construction and monthly in first year of operation. After the dam's behavior has stabilized (say a few years of consistent readings), the frequency might be reduced (piezometers perhaps monthly or quarterly, deformation surveys annually). However, after extreme events (unusual floods, earthquakes) or if any abnormal reading occurs, immediate or extra reading should be taken. All scheduled and unscheduled readings should be logged. It is good practice to take synchronized readings – for instance, measure reservoir level, all piezometers, seepage flows, and weather data on the same day so the data can be correlated.

8.8.2. Data management

Readings should be entered into a database or spreadsheet and plotted over time. The dam monitoring team should plot individual instrument trends and also cross-plot related data (ex. pore pressure vs. reservoir elevation). Modern dam safety programs use software that can generate automated alerts if reading exceeds predetermined thresholds. In India, the Dam Health and Rehabilitation Monitoring Application (DHARMA) platform (under DRIP) is an example of a database where instrument data can be stored and analyzed. Regardless of the system, the key is consistency and accuracy: if done manually, the same measurement techniques and personnel training should be used to reduce errors.

8.8.3. Baseline and thresholds

Every instrument reading should ideally be compared to a baseline (initial reading or expected value). Design documents often provide anticipated values, for example, the phreatic line location, or predicted settlement. Thresholds are set such that if an instrument exceeds a certain value, it triggers action. Thresholds are commonly categorized (ex. Normal, Alert, Alarm levels). For instance, if a piezometer reads a pressure head that is 1.2 times higher than expected at full reservoir, one might declare an Alert (increased monitoring frequency) and at 1.5 times, an Alarm (initiate emergency procedures). These numbers are dam-specific; the CWC guidelines recommend developing threshold values from initial observations and past performance of the dam.

8.8.4. Interpreting trends

It is not just the absolute value but the trend that matters. Stable readings over a long period suggest the dam is performing as expected. Gradual changes often correlate with seasonal or reservoir changes – ex. higher piezometer levels in monsoon when the reservoir is full, and lower in summer. These are normal if reversible. However, accelerating or nonlinear trends are of concern. For ex., if settlement started at 2 cm per year and has grown to 5 cm per year, or if a piezometer that used to fluctuate between 5–7 m head suddenly rises to 10 m, something has changed in the dam system. A major part of interpretation is also comparing instruments: if one piezometer rises but others in the area do not, the issue might be localized near that instrument (perhaps a filter clog there or instrument malfunction). But if multiple instruments concur (ex. several piezometers and increasing seepage flow all suggest higher seepage), then the likelihood of a problem is high.

8.9. MAINTENANCE OF INSTRUMENTS

Sometimes strange data is simply due to instrument malfunction (ex. a clogged piezometer tip, a sheared inclinometer casing, or a shifted survey benchmark). Regular maintenance and verification are needed. Standpipe piezometers should be slug tested or flushed if suspected clogged; vibrating wire instruments should be checked against manual measurements occasionally. Calibration of sensors (like load cells, pressure transducers) must be done as per manufacturer’s schedule. Field engineers should keep notes of any maintenance because that can affect readings (for example, if you flush a piezometer, the next reading might be temporarily lower until it re-equilibrates).

8.10. INTEGRATION WITH INSPECTIONS

Instrumentation data should always be reviewed in tandem with visual inspections of the dam. Instruments can pick up what the eyes can’t see internally, and conversely, inspectors might notice a wet area or crack that the instruments haven’t captured. A classic example: increasing seepage flow was recorded (instrument data) and on inspection a new wet spot was found on the downstream toe with a small whirlpool (sign of piping exit). Neither alone gives the full story, but together they confirm a serious issue. Thus, training workshops emphasize correlating instrument data with field observations and other information (reservoir ops log, rainfall, etc.).

8.11. DECISION MAKING

Dam safety professionals must know how to act on the data. CWC’s dam safety guidelines outline that if an instrument reaches alarm level, the dam owner should convene a Dam Safety Review Panel or experts immediately. Lesser abnormalities might trigger more frequent readings and analysis. Ultimately, the data feed into decisions such as: does the reservoir need to be lowered as a precaution? Is relief well installation necessary to alleviate pressure? Should grouting be done to address suspected internal erosion? Because instrumentation reduces uncertainty, it enables targeted interventions rather than blind conservatism. For example, if instruments show the dam is performing well within safe limits, the operation can continue normally; if not, emergency preparedness plans are activated. One must also be aware of false alarms vs. real alarms – sometimes a single spurious reading can cause panic. Best practice is to confirm abnormal data (take a re-reading, ensure the instrument is working properly) promptly.

Instrumentation and monitoring thus provide a basis for evaluating dam safety over time. A well-instrumented earthen dam will have data on seepage, pore pressures, deformations, and more, which together paint a comprehensive picture of the dam’s behavior. By using the instrumentation intelligently (from installation to data analysis), one can identify distress at an early stage and implement remedial measures before any hazard escalates. Moreover, comparing actual performance data with design predictions helps improve future dam designs and validates analytical methods. In summary, for practical field application it should be ensured that instruments are installed correctly and protected, readings as scheduled, records are maintained, and data is interpreted in context. Thus, instrumentation is a powerful tool in dam safety when combined with engineering judgment and regular inspections.

CHAPTER - 9

CASE STUDIES

Neeraj Mansingh Meena, Scientist ‘B’

9. GENERAL

This chapter presents selected case studies pertaining to earthen dams, focusing on their rehabilitation through comprehensive geotechnical investigations and analysis.

9.1. ISLAMPUR EARTH DAM, MAHARASHTRA

The Kurha - Vadhodha Islampur irrigation project was designed to irrigate approximately 25,898 hectares of land across Muktainagar taluka of Jalgaon district and Sangrampur and Jalgaon-Jamod talukas of Buldhana district in Maharashtra. Seepage studies were conducted in two sections of the earthen dam: Stretch I (Ch. 990 m to 1290 m) and Stretch II (Ch. 1650 m to 1920 m) to determine appropriate cut-off-trench (CoT) depths and analyze seepage characteristics. 2-dimensional seepage analysis using PLAXIS software was conducted for four different cases (A and B in Stretch I, C and D in Stretch II). The analysis aimed to determine seepage characteristics of the design dam cross-sections and recommend appropriate remedial measures for seepage mitigation. Results indicated total seepage values of 0.7102, 0.7738, 0.2699, and 0.7169 m^3/day/m for cases A, B, C, and D respectively—all exceeding permissible values.

Several remedial measures were analyzed, including foundation rock grouting, Cut-off-wall installation below CoT & Upstream horizontal blanket in combination with a 6.0 m deep cut-off-wall. Analysis with foundation rock grouting reduced total seepage discharge to 0.2494, 0.2742, 0.04178, and 0.04582 m^3/day/m for cases A, B, C, and D respectively. For cases C and D, this brought discharge within the lower permissible seepage limit of 0.2 m^3/day/m, but cases A and B still exceeded the upper permissible limit. Cut-off-walls of varying depths were tested as an alternative solution. Based on these studies, the following optimum seepage mitigation measures were recommended:

Case A: 35 m deep cut-off-wall below CoT up to RL 267.10 m

Case B: 30 m deep cut-off-wall below CoT up to RL 265.96 m

Case C: 5 m deep cut-off-wall below CoT up to RL 270.28 m

Case D: 15 m deep cut-off-wall below CoT up to RL 271.49 m

The recommendations were based on permissible seepage values specified in the Handbook of Geotechnical Investigations and Design Tables by Burt Look. The study suggested that project authorities might adopt alternative remedial measures based on different permissible values, considering economic aspects, site-specific constraints, and construction limitations.

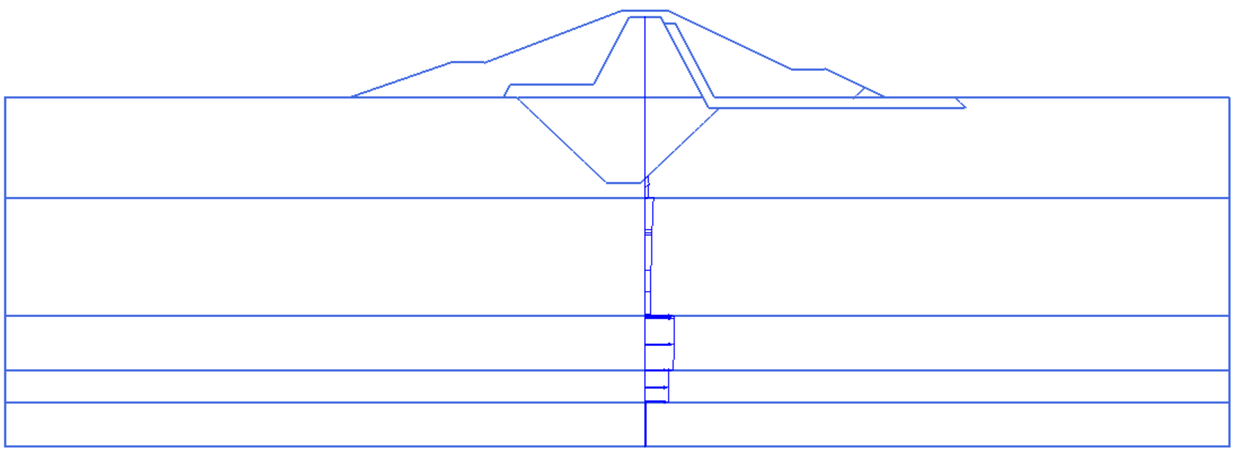

Figure 9.1 Seepage flow vectors at design dam cross-section

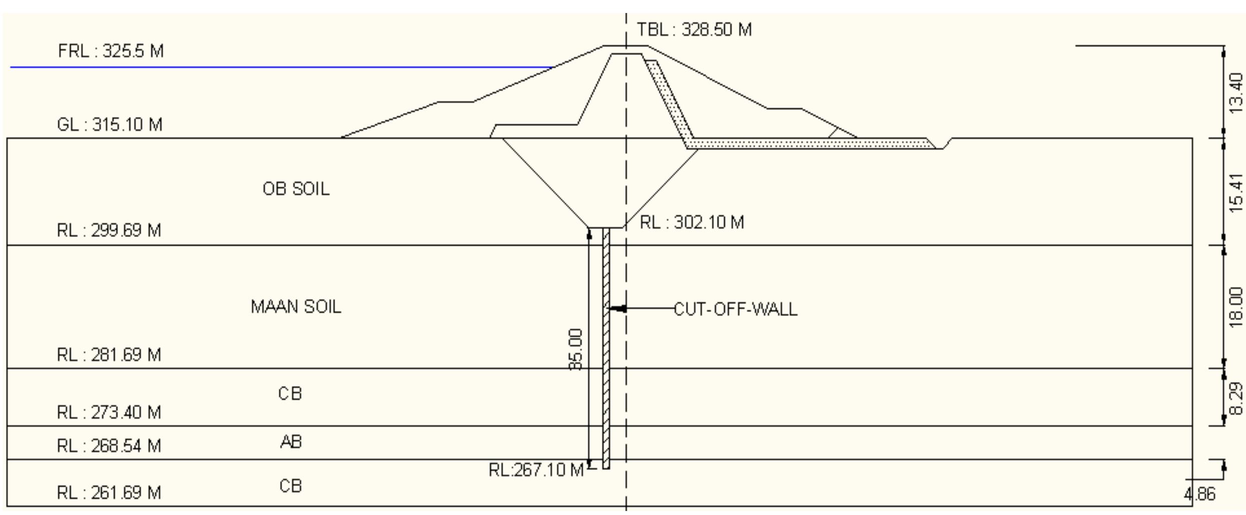

Figure 9.2 Dam cross-section with 35 m deep cut-off-wall below CoT

9.2. KANGSABATI DAM, WEST BENGAL

The Kangsabati-Kumari dam (also known as Mukutmanipur dam) is in Bankura district, West Bengal. Constructed in 1965 near the confluence of the Kangsabati and Kumari rivers, it is the second largest earthen dam in India with a total length of 10.4 km and maximum height of 41.0 m. The Central Water and Power Research Station (CWPRS) conducted seepage analysis on three dam sections on the Kangsabati side:

- Section I (Chainage 117, Ht. 32.813 m)
- Section II (Chainage 26, Ht. 26.748 m)
- Section III (Chainage 54, Ht. 19.973 m)

The analysis employed FE software PLAXIS 2D for seepage and limit equilibrium methods for slope stability as per IS 7894. Results showed seepage discharge quantities of 103.144 m³/day, 452.930 m³/day, and 230.550 m³/day for Sections I, II, and III respectively. The total discharge for the entire dam length was calculated at 786.620 m³/day. For a dam of height 20 m to 40 m, discharge should fall between 0.2 m³/day/m (allowable value) and 0.4 m³/day/m (limiting value). Given the total dam length, the desired value was calculated at 957.072 m³/day and the limiting value at 1914.14 m³/day. Although the discharge value from the analysis was within acceptable limits, the study recommended monitoring actual discharge through installation of drainage and seepage measurement devices.

Slope stability analysis revealed that existing dam sections I and II were unsafe for steady seepage and sudden drawdown conditions, requiring modifications. Due to practical limitations for increasing dam base width, recommendations included flattening of slopes, adding berms, increasing berm width, and providing shear keys with gabion fill as needed. Optimum shear key dimensions were also specified. Section III was found to be safe for both conditions and required no modifications.

The critical values of Factor of Safety (FS) for the modified sections were:

- Modified Section I (Ch.105 to Ch.121): 1.54 for steady seepage & 1.31 for sudden drawdown
- Modified Section II (Ch. 19 to Ch.41, Ch. 55 to Ch.105, Ch. 121 to Ch.128): 1.58 and 1.31 respectively
- Existing Section III (Ch. 0 to Ch.19, Ch. 41 to Ch.55, Ch. 128 to Ch.157): 1.55 and 1.61 respectively

Additional analysis determined the maximum safe reservoir water level against downstream slope failure under steady seepage conditions. Results indicated that even with half the water head (corresponding to water level RL 119.483 m), the existing dam Section I remained unsafe, with critical FS value increasing only marginally from 1.13 to 1.17.Alternative slope stability measures were also analysed for Shear key with soil-cement grout as fill material (effective in increasing FS) and Rock anchored concrete toe wall (found ineffective) Optimum dimensions for the shear key with soil-cement grout fill were recommended based on slope stability analysis. The study advised determining grout composition, pattern, spacing of grout holes, and grouting pressure through trials to achieve design values for density, cohesion, and friction angle of the grouted soil in the shear key.

Since the dam is in seismic zone II according to India's seismicity map, pseudo-static seismic analysis was conducted. Modified dam sections I and II with soil-cement grout fill shear keys and existing section III showed critical FS values above 1.0 for steady seepage and reservoir full conditions as per IS 7894, confirming safety under earthquake conditions. The study recommended installing piezometers to monitor pore pressure and maintaining the dam site according to Central Water Commission (CWC) guidelines.

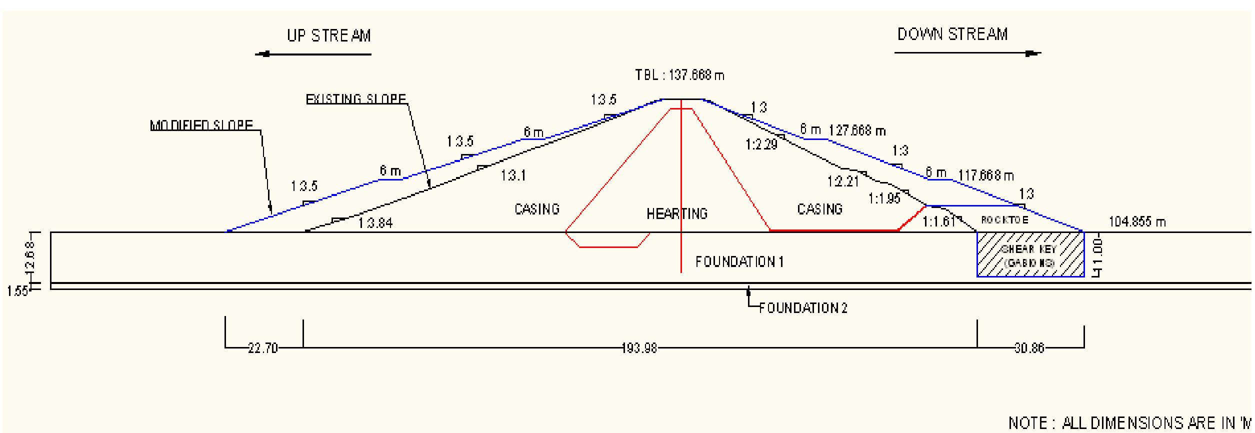

Figure 9.3 Modified Section I dam (with shear key of gabion material)

9.3. DUDHAWA DAM, CHHATTISGARH

A 24.7 m The Dudhawa dam, located in Kanker district, Chhattisgarh, was constructed as part of the Mahanadi irrigation project in 1962. The dam features a 2907 m long earthen embankment with a height of 24.7 m. Since its construction and initial water storage, the dam has experienced seepage and sand boil problems, preventing its use at full capacity. Although temporary remedial measures were implemented periodically, a comprehensive study became necessary. CWPRS conducted detailed field investigations and mathematical model analysis for seepage and stability at two critical sections: Ch. 40-42 m and Ch. 83-84 m.

Seepage analysis utilized finite element software with actual prototype boundary conditions, including reservoir and tail water levels. To assess filter performance, numerous piezometers were installed at the site, primarily along the dam toe and at the two critical cross-sections. The analysis incorporated piezometric head measurements from the dam body and foundation. Permeability coefficients for different dam zones were derived from laboratory permeability tests. Slope stability analysis incorporating seepage forces showed that both affected sections had factor of safety values significantly below the requirements specified in IS 7894. Seepage analysis also revealed unsatisfactory performance of the internal drainage system, including the horizontal filter and rock toe, necessitating remedial measures.

The study recommended downstream slope flattening or toe loading with an additional berm to counterbalance actuating forces and increase stabilizing forces. A well-designed, adequately constructed drainage system beneath the loading berm was also recommended to safely discharge seepage flow downstream. Mathematical modeling indicated a need for 1.5 m thick rubble filled loading on the downstream slope, extending from the downstream toe to 20 m in the downstream direction. This remedial measure increased the factor of safety of the dam sections to 1.5, meeting Bureau of Indian Standards (BIS) requirements.

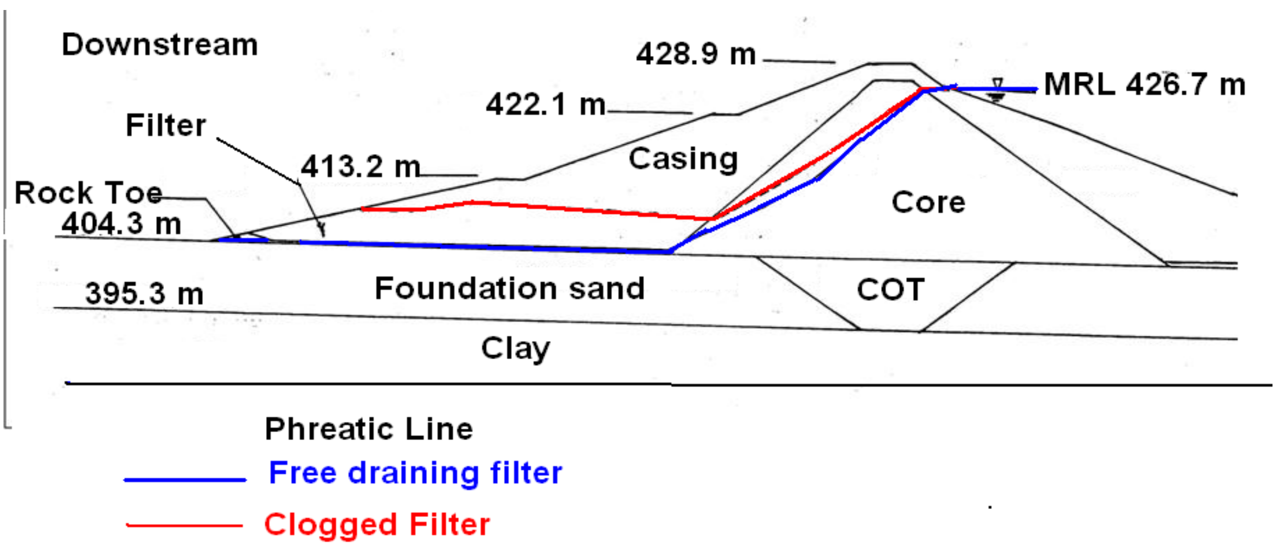

Figure 9.4 Unsatisfactory functioning of internal drainage system

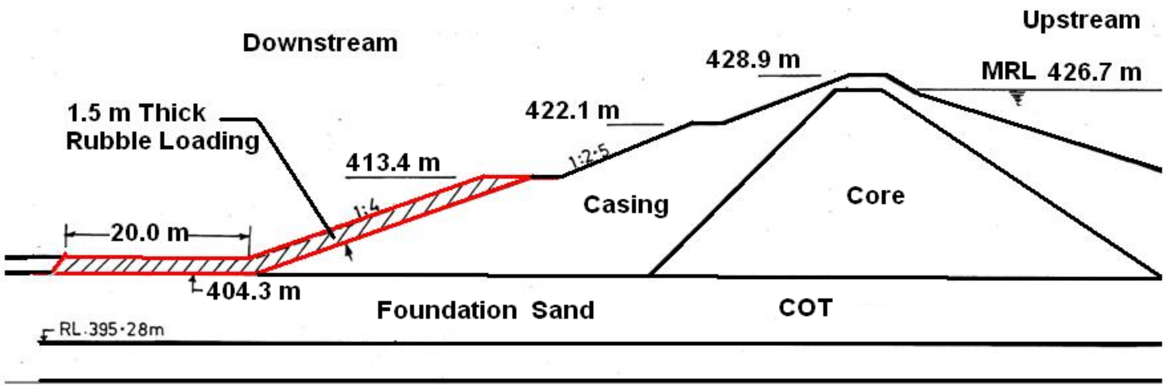

Figure 9.5 1.5 m thick rubble filled loading on downstream slope

9.4. UKAI, GUJARAT

The Ukai dam spans the Tapi River in Gujarat, approximately 110 km east of Surat. The earthen dam measures 1900 m in length and 70 m in height, and is situated about 40 km upstream of the Kakrapar Atomic Power Plant. As part of safety requirements for the power plant, stability analysis of the embankment dam under dynamic loading conditions was undertaken. Laboratory testing used Resonant Column and Dynamic Triaxial test equipment to determine strain-dependent shear modulus, damping properties, and dynamic strength properties of embankment and foundation soils. Response analysis of the dam employed Finite Element Model software using earthquake data provided by project authorities, with maximum acceleration of 1961 mm/s².

The analysis determined shear stresses induced in the dam due to earthquake loading. By comparing dynamic soil strength with earthquake-induced stresses, strain potentials within the dam body were calculated. Deformation analysis quantified potential permanent deformation resulting from seismic activity. Results indicated a maximum vertical settlement of 0.167 m on the downstream slope and maximum horizontal displacement of 0.74 m on the upstream slope. At the crest, vertical and horizontal displacements were calculated at 0.02 m and 0.31 m respectively.

The study concluded that the Ukai earthen dam could withstand the design earthquake with maximum ground acceleration of 1961 mm/s², although local and insignificant yielding might occur in small zones near the top of the dam.

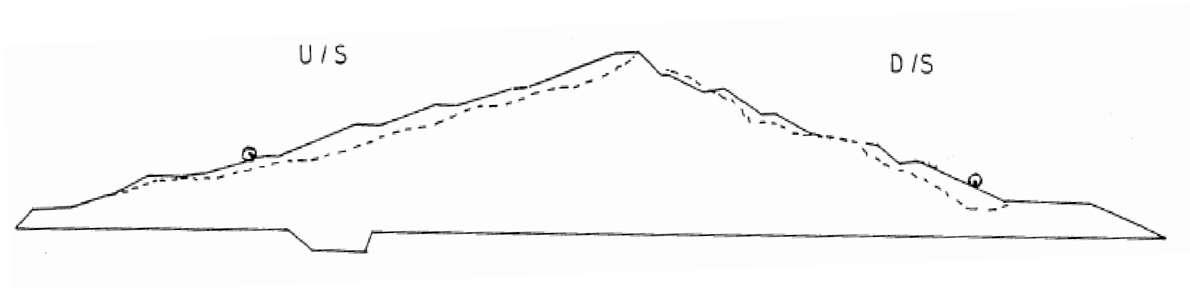

Figure 9.6 Original and deformed shape

9.5. ADLABS EARTHEN DAM FOR ENTERTAINMENT PARK, MAHARASHTRA

A 24.4 m high, 180.0 m long zoned earthen dam surrounded by hills was constructed near Khopoli in Raigad district, Maharashtra, to supply water to an entertainment park. Since its completion, the dam has experienced excessive seepage through its body and abutment.

Site inspection revealed continuous water seepage from the downstream slope at various elevations and from a point source near the right abutment. Several locations on the downstream slope showed subsidence of pitching and animal burrows. Three boreholes drilled from the dam crest through the hearting zone revealed boulders and relatively pervious soil (silty sand) to approximately 9.0 m depth. In-situ testing established an average permeability value for the hearting of 2.0×10^{-7} m/s.

Steady state seepage analysis using PLAXIS 2D software calculated seepage through the dam body at 38.4 m^3/day for reservoir level 147.0 m. However, reported total seepage (through dam body plus abutment) was approximately 2000 m^3/day, indicating that most seepage occurred through the right abutment. Slope stability analysis showed a factor of safety of 1.94 for a dry downstream slope. However, since field observations contradicted this condition, additional analysis with an elevated phreatic line was conducted. This analysis demonstrated reduced factor of safety values as the phreatic line elevation increased.

The study recommended remedial measures including grouting from the upstream top berm to foundation rock level and installing upstream geomembrane lining. Additional recommendations included installing monitoring instruments such as piezometers and V-notch weirs to track pore pressures and seepage quantities through the dam and foundation.

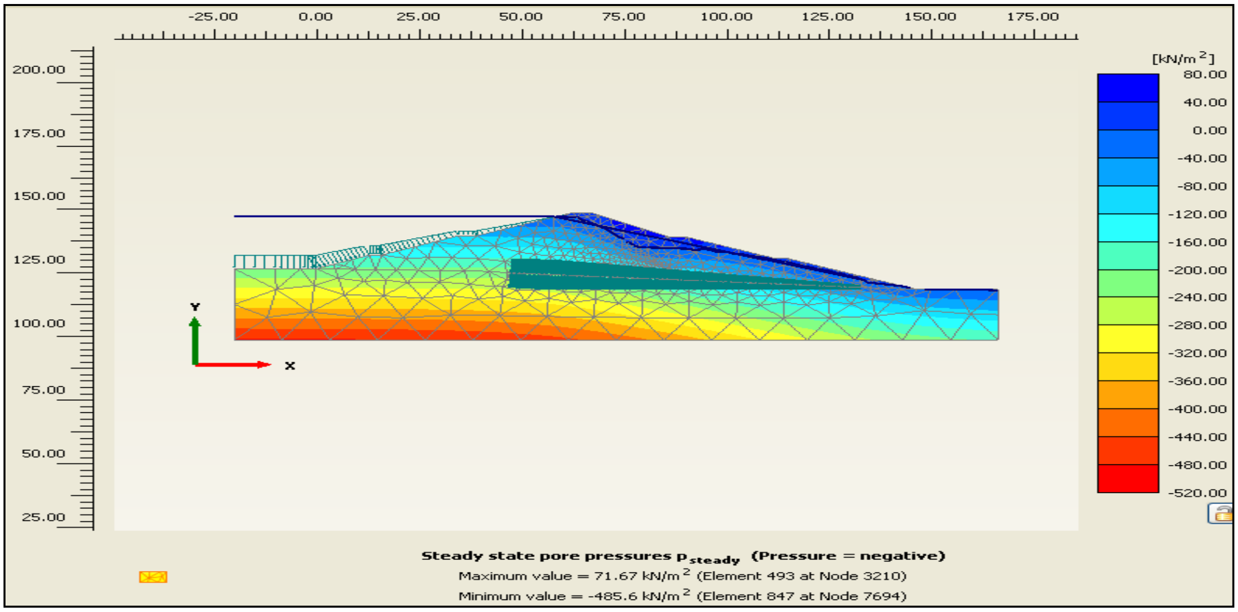

Figure 9.7 Steady seepage pore pressure contours

Figure 9.8 Installation of geomembrane on upstream face

9.6. KURUMURTHIRAYA RESERVOIR, TELANGANA

The Irrigation & CAD department, Government of Telangana, initiated the Palamuru-Rangareddy Lift Irrigation Scheme (PRLIS) to address irrigation and drinking water needs for 12.30 lakh acres in Telangana. The project involves lifting 90 TMC feet of water from the foreshore of Srisailam reservoir on Krishna River during flood season through construction of multiple storage reservoirs, pump houses, and canal networks. Kurumurthiraya is one of six planned reservoirs with a proposed capacity of 17.34 TMC feet. The reservoir is formed by a multi-zoned earth dam stretching 12.95 km with a maximum height of approximately 61.5 m. The Executive Engineer, I&CAD, Mahabubnagar, Telangana requested CWPRS to conduct geotechnical seepage and stability studies for this complex earth dam.

For construction efficiency, the dam's total length was divided into three packages: 13, 14, and 15. Four cross-sections were analyzed:

- Section I (Ht. 41.5 m at Ch. 0.4 km) in Package 13
- Section II (Ht. 61.5 m at Ch. 3.7 km) in Package 13
- Section III (Ht. 58.5 m at Ch. 6.7 km) in Package 14
- Section IV (Ht. 50.5 m at Ch. 10.2 km) in Package 15

Seepage analysis using PLAXIS 2D software examined steady state conditions to establish phreatic lines, seepage discharge, pore pressures, and hydraulic heads throughout the dam zones. Transient state analysis determined phreatic line behaviour during drawdown from Full Reservoir Level (FRL at 531.0 m) to Minimum Draw Down Level (MDDL at 491.1 m) at a rate of 3 m/month. Slope stability analyses were conducted using limit equilibrium software for steady state conditions, instantaneous drawdown from FRL to RL 512.0 m, and transient drawdown conditions.

Results showed total seepage discharge (dam body + foundation) of 1.088 m^3/day/m, 1.014 m^3/day/m, 0.116 m^3/day/m, and 0.595 m^3/day/m for cross-sections I, II, III, and IV respectively. A significant finding was that foundation seepage accounted for 90-98% of total discharge. For cross-sections I and II, discharge values exceeded the specified upper permissible limit of 0.8 m^3/day/m. The study recommended establishing permissible water loss criteria for the reservoir and implementing regular seepage discharge monitoring. It also advised adopting appropriate foundation seepage remediation if discharge exceeded the permissible values determined by project authorities.

Slope stability analysis confirmed that all four dam cross-sections were safe, with Factor of Safety (FS) values exceeding the required 1.5 for steady seepage and 1.3 for drawdown conditions as specified in IS 7894:1975. Pseudo-static analysis for earthquake conditions showed the dam remained safe with FS values above the required 1.0 for both steady seepage and full reservoir conditions. Additional recommendations included designing zone-I on upstream and downstream sides as transition zones and implementing regular dam maintenance and monitoring through piezometers, 'V' notch weirs, and appropriate drainage arrangements according to relevant IS codes

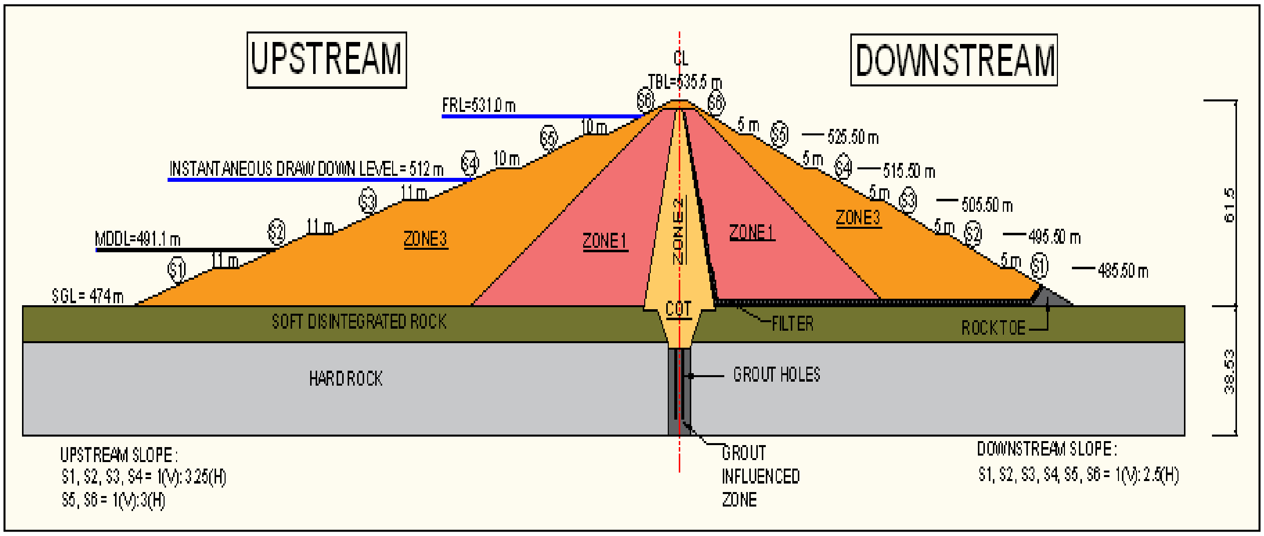

Figure 9.9 Design dam section ‘II’ of height 61.5 m at Ch. 3.7 km (maximum height)

9.7. SANKOSH ROCKFILL DAM, BHUTAN

The Tehri Hydro Development Corporation Ltd. (THDC) planned the construction of a 265 m high rockfill dam across the Sankosh River near Kerabari as part of the Sankosh Multipurpose project. To ensure the dam's safety, comprehensive studies were conducted including slope stability analysis of the downstream under steady seepage and upstream under sudden drawdown, pseudo-static slope stability analysis, and earthquake-induced deformation analysis.

The proposed main rockfill dam featured a central clay core flanked on both sides by rolled rockfill with maximum particle size of 450 mm. Slope stability analysis using Bishop's method demonstrated a minimum Factor of Safety (FoS) of 1.89 for the downstream slope under steady seepage conditions. For the upstream slope, analysis showed minimum FoS values of 2.29 for reservoir full conditions and 1.51 for sudden drawdown conditions.

Pseudo-static slope stability analysis, employing a seismic coefficient of 0.11g, indicated minimum FoS values of 1.48 and 1.43 for downstream and upstream slopes respectively. Dynamic deformation analysis employed Newmark's approach using two methodologies:

- Prof. F. Makdisi and Prof. B. Seed's empirical curves
- Dynamic response analysis using finite element method

This analysis utilized Design Basis Earthquake (DBE) parameters with Peak Ground Acceleration (PGA) of 0.19g (horizontal) and 0.12g (vertical). The yield acceleration (Ky) for the potential sliding mass was calculated at 0.22g, while the maximum acceleration (Kmax) of the vibrating potential sliding mass was 0. 0875g.With the ratio of Ky to Kmax exceeding 1.0, the analysis concluded that the potential sliding mass would not undergo any permanent displacement during a Design Basis Earthquake, confirming the dam's seismic stability.

CHAPTER - 10

OVERALL GUIDELINES AND RECOMMENDATIONS

Neeraj Mansingh Meena, Scientist 'B'

10. GENERAL

This chapter provides a concise summary of direct, actionable points for dam authorities, focusing on the strategic rehabilitation of embankment dams, followed by proactive maintenance and monitoring. These guidelines are framed based on the expertise and study-based approach of CWPRS, as detailed in the current memorandum. Furthermore, they are also based on the Bureau of Indian Standards (BIS) codes relevant to earthen dam design, construction, safety, and rehabilitation (such as IS 8826, IS 12169, IS 14954, IS 7894, IS 9429, IS 1893, IS 14690, and various parts of IS 2720 for testing of soil samples, amongst others). The recommendations also incorporate best practices and insights from various manuals and guidelines issued by the Central Water Commission (CWC) (including those on Safety Inspection, Assessing Structural Safety, Rehabilitation of Large Dams, Instrumentation, O&M Manuals) and those developed under the Dam Rehabilitation and Improvement Project (DRIP), which aim to enhance dam safety management in India. The Dam Safety Act, 2021, serves as the supreme regulatory framework, mandating comprehensive measures for all specified dams. Adherence to these collective guidelines, coupled with sound engineering judgment and expert consultation, is essential for sustainable operation of these vital structures.

10.1. ROLE OF CWPRS IN IMPLEMENTING DAM REHABILITATION

The Central Water and Power Research Station (CWPRS) is a key partner for dam authorities in ensuring the safety and longevity of embankment dams. CWPRS provides essential expert support, especially for the complex tasks of dam rehabilitation and in applying these guidelines effectively. Key activities undertaken by CWPRS that can be helpful to dam authorities are listed below:

- **Guideline implementation support (General):** For almost every guideline listed below, from initial site investigations and data analysis to complex modeling and choosing the right repair methodologies, CWPRS offers specialized services to help authorities apply them correctly and effectively.
- **Expert site inspections and distress diagnosis:** CWPRS can lead or support detailed site inspections, help accurately document distresses, identify critical signs, and correlate observations with operational data, ensuring no crucial detail is missed.
- **Comprehensive data review and investigation planning:** CWPRS can assist in thoroughly reviewing all existing dam data, identifying gaps, and planning targeted, cost-effective additional geotechnical investigations (field and lab tests) if needed.
- **Advanced analysis and parameter finalization:** CWPRS uses its expertise and advanced tools (like FEM/ FDM softwares) to conduct detailed seepage and stability analyses (including earthquake safety and liquefaction checks), ensuring compliance with IS code requirements for Factors of Safety.
- **Optimizing rehabilitation solutions:** Based on the rigorous study-based approach, CWPRS can help dam owners select the most technically sound and site-specific repair solutions for any diagnosed issues, whether it's controlling seepage, stabilizing slopes, or improving earthquake resistance.
- **Guidance on monitoring and maintenance strategies:** CWPRS can advise on setting up effective instrumentation plans, interpreting and monitoring data, developing robust O&M Manuals, and establishing best practices for vegetation, erosion, and animal control.

By partnering with CWPRS, dam authorities can ensure that their approach to dam safety is not just compliant, but also technically robust, utilizing the best available expertise and methodologies to safeguard these critical national assets.

10.2. GUIDELINES FOR DAM REHABILITATION AND MAINTENANCE

A systematic, study-based approach is critical when an embankment dam shows signs of distress or requires upgrading to meet current safety standards. The following guidelines outline a comprehensive process, presenting requirements as direct, actionable instructions with specific technical criteria where applicable:

a) **Systematic visual inspections:** Perform regular and thorough site inspections by qualified and experienced engineering personnel to systematically identify all types of distresses (*CWC 'Guidelines for Safety Inspection of Dams'*; *IS 9296:1979)*

b) **Identify and report critical observations:** Thoroughly review all historical data for information gaps, meticulously document the precise location and progression of distresses observed at site and immediately report any critical warning signs such as muddy seepage, sinkholes, or sudden changes in water level the moment they appear.

c) **Seek expert consultation for investigation planning:** If existing data is found to be insufficient, outdated, or inadequate for a thorough safety assessment; obtain expert advice to formulate a detailed and technically sound plan for additional investigations.

d) **Execute targeted geotechnical field investigations:** To address specific concerns identified during site inspections and data review, conduct targeted field investigations. These may include drilling new boreholes, excavating trial pits, and performing in-situ tests.

e) **Conduct comprehensive laboratory testing program:** Conduct comprehensive laboratory tests on the collected disturbed and undisturbed soil and rock samples to determine:

- **Index properties**: Natural moisture content, density, specific gravity, particle size distribution, Atterberg limits (as per IS 2720 Parts 2, 3, 4, 5).
- **Compaction characteristics**: Maximum Dry Density (MDD) and Optimum Moisture Content (OMC) using Standard or Modified Proctor tests (as per IS 2720 Parts 7, 8).
- **Shear strength parameters**: Cohesion (c) and angle of internal friction (ϕ) through direct shear tests (as per IS 2720 Part 13) or triaxial shear tests (UU, CU, CD as appropriate - as per IS 2720 Parts 11, 12).
- **Consolidation properties**: Coefficient of consolidation (C_v), compression index (C_c), etc., using oedometer tests (as per IS 2720 Part 15).
- **Hydraulic properties**: Coefficient of permeability (k) using constant head or falling head tests (as per IS 2720 Part 17).
- **Dynamic soil properties** (if seismic analysis is required): Shear modulus (G), damping ratio (D) from resonant column tests, cyclic triaxial tests, or bender element tests.

f) **Evaluate need for geophysical investigations:** Evaluate the need for non-invasive geophysical surveys (like ERT, GPR, or seismic refraction) to map internal features such as anomalies, seepage paths, voids, or subsurface variations. These methods are particularly useful where drilling is difficult or when more data is needed.

g) **Finalize design input parameters with representative soil properties:** Based on a complete review of all historical and new geotechnical data, finalize the design parameters for each distinct soil and rock zone of the dam and its foundation. This includes selecting representative values for density, shear strength (c′, ϕ ′), permeability (k), and stiffness (E, μ).

h) **Perform comprehensive seepage analysis:** Conduct detailed seepage analysis using appropriate numerical modeling techniques (ex. Finite Element/ Finite Difference Methods – FEM/ FDM) to simulate the seepage regime through dam body and foundation under various critical operating conditions, including steady-state seepage at Full Reservoir Level (FRL) and transient conditions during rapid drawdown.

i) **Conduct comprehensive slope stability analysis:** Perform detailed slope stability analyses for upstream and downstream slopes of all critical dam sections using accepted Limit Equilibrium Methods (LEM). Utilize appropriate shear strength parameters (c', ϕ ' for effective stress analysis) derived from laboratory tests.

j) **Perform detailed seismic stability assessment:** Conduct stability analyses for seismic conditions by pseudo-static or other detailed methods using appropriate horizontal and vertical seismic coefficients derived from IS 1893 based on the dam's seismic zone and importance.

k) **Assess liquefaction potential:** For dams with saturated, loose, cohesionless materials in the embankment or foundation, which are potentially liquefiable; assess liquefaction potential under design earthquake loading using empirical methods based on SPT N-values or CPT data, or through advanced effective stress dynamic analysis.

l) **Assess stability for critical loading conditions and ensure minimum Factors of Safety (FS):** The stability of earth dams must be ensured under various operational, loading, and environmental scenarios, including seismic events. The minimum required Factors of Safety (FS) for various critical loading conditions, primarily guided by IS 7894:1975 and IS 1893 (Part 1):2016 for seismic conditions, are summarized below:

Table 10.1 Factor of safety required for various conditions

Condition	Critical Slope	F.S.	Remarks
End-of-Construction	Upstream & Downstream	>1.3	This condition considers the stability of slopes immediately after completion of the dam construction, before filling of reservoir.
Steady Seepage	Downstream	>1.5	This is a long-term condition where the reservoir has been full for a sufficient

			duration to establish a steady seepage pattern through the dam.
Rapid Drawdown	Upstream	>1.3	This condition simulates the sudden lowering of reservoir water level, where pore water pressures within the upstream slope do not dissipate quickly.
Construction	Upstream & Downstream	>1.5	This applies to the stability of slopes during construction phase of the dam.
Seismic (Earthquake)	Upstream & Downstream	>1.0	The analysis is conducted using pseudo-static or dynamic methods as outlined in IS 1893.

m) **Structured decision-making framework for rehabilitation measures:** Use a structured framework to select the best rehabilitation measure evaluating all potential solutions against key criteria such as: technical feasibility, effectiveness, cost-benefit, constructability, operational impacts, environmental/ social effects, and long-term durability.

n) **Evaluate and implement appropriate seepage mitigation measures:**

- **Foundation seepage control**: Consider and evaluate techniques such as cut-off walls (ex. concrete diaphragm, soil-bentonite slurry, as per IS 14344), grouting (foundation or dam body, as per IS 11293-1, IS 6066, IS 4999, IS 11973), upstream impervious blankets (compacted clay or geomembranes), and relief wells (IS 5050, IS 8414).
- **Dam body seepage control**: Consider and evaluate upstream liners, internal core/ diaphragm wall repair or construction, grouting in the dam body, and enhancement/ installation of internal drains and filters (IS 9429).

o) **Implement appropriate structural stability enhancement measures:** Consider and evaluate a range of rehabilitation options.

- These include slope stability measures like slope flattening, berm construction, soil reinforcement, retaining walls, and foundation shear keys.
- Also assess seepage control methods such as filling cracks and restoring the bond at structural junctions (IS 17837), as well as capacity and drainage improvements like raising the dam crest and upgrading surface drainage.

p) **Evaluate and implement appropriate seismic restoration and liquefaction measures:** Consider and evaluate ground improvement techniques (ex. vibro-compaction, stone columns, dynamic compaction), reinforcement of critical zones, and improved drainage systems to dissipate seismic pore pressures.

q) **Ensure strict quality control for rehabilitation works:** Implement and enforce strict quality control and assurance measures during the execution of all rehabilitation works, ensuring adherence to design specifications and relevant IS codes for construction (IS 14690).

r) **Implement a standardized instrumentation and data management plan:**

- Equip the dam with essential instruments: piezometers (IS 7356-1), seepage measurement weirs (IS 14750), surface settlement monuments, internal settlement devices (IS 7500, IS 8226), and inclinometers.
- Plan instrument locations strategically at critical sections and to target suspected weak points or seepage paths (as per CWC Guidelines for Instrumentation of Large Dams and IS 7436 Part 1).
- Establish clear procedures for reading frequency, data recording (ex. DHARMA platform), plotting trends, and comparing with baseline data and pre-defined alert/ alarm thresholds.
- Monitor for internal erosion indicators such as increased or turbid seepage, anomalous piezometer readings, or localized settlements.
- Implement a regular maintenance schedule for all instruments to ensure their proper functioning and reliability.

s) **Enforce strict vegetation, erosion, and animal control:**

- Manage vegetation proactively by prohibiting and removing trees and large shrubs.
- Maintain a healthy, short grass cover through regular mowing (a key maintenance activity as highlighted in DRIP-related maintenance guidelines).
- Promptly repair any surface erosion (rills, gullies less than 6 inches deep should be filled with suitable compacted soil and seeded/ sodded).
- Ensure surface drainage systems are clear and functional (as per IS 8237 and DRIP-related maintenance guidelines).
- Promptly and properly excavate and backfill any animal burrows with compacted, low-permeability soil (ex. soil-bentonite mix).

t) **Adherence to Dam Safety Act, 2021 and National Standards:** Ensure full compliance with all provisions of the Dam Safety Act, 2021, and all relevant national standards (BIS codes) and guidelines (CWC, DRIP). Actively participate in the regular review and modernization of dam safety regulations.

A list of all relevant BIS codes, manuals and guidelines is presented at the end of this chapter for ready reference.

10.2.1. List of bis codes relevant to earthen dams

1. IS 8826:1978, Guidelines for Design of Large Earth and Rockfill Dams
2. IS 12169:1987, Criteria for Design of Small Embankment Dams
3. IS 1498:1970, Classification and identification of soils for general engineering purposes
4. IS 14690:1999, Quality Control During Construction of Earth and Rockfill Dams – Recommendations
5. IS 14954:2001, Distress and Remedial Measures in Earth and Rockfill Dams – Guidelines
6. IS 7894:1975, Code of Practice for Stability Analysis of Earth Dams
7. IS 9429:1999, Drainage System for Earth and Rockfill Dams - Code of Practice
8. IS 5050:1992, Code of practice for design, construction and maintenance of relief wells
9. IS 8414: 1977, Guidelines for design of under-seepage control measures for earth and rockfill dams
10. IS 8237:1976, Code of practice for protection of slope for reservoir embankment
11. IS 17837:2022, Junction of Earth/Earth Core Rockfill Dam with Spillway, Non-Overflow (NOF) Dam, Foundation and Outlets-Guidelines
12. IS 6066:1994, Pressure grouting of rock foundations in river valley projects - Recommendations
13. IS 11293-1: 1985, Guidelines for the design of grout curtains, Part 1: Earth and rockfill dams
14. IS 4999: 1991, Recommendations for grouting of pervious soils
15. IS 11973: 1986, Code of practice for treatment of rock foundations, core and abutment, contacts with rock, for embankment
16. IS 14344: 1996, Design and construction of diaphragms for under seepage control - Code of practice
17. IS 1893:2016, Criteria for earthquake resistant design of structures
18. IS 7436 (part 1):1993, Guide for types of measurements for structures in river valley projects and criteria for choice and location of measuring instruments
19. IS 1892:1992, Code of practice for subsurface investigation for foundations
20. IS 6955 (2008), Subsurface exploration for earth and rockfill dams - Code of practice
21. IS 2131:1981, Method for standard penetration test for soils

22. IS 7356-1:2002 Code of Practice for Installation, Maintenance and Observation of Instruments for Pore Pressure Measurements in Earth Dams and Rockfill Dams, Part 1: Porous Tube Piezometers
23. 9296:1979 Guidelines for inspection and maintenance of dam and appurtenant structures

10.2.2. List of guidelines

1. **Guidelines for Assessing and Managing Risks Associated with Dams**
   Introduces a risk-informed dam safety framework, including failure mode identification and risk assessment methodologies.
2. **Guidelines for Safety Inspection of Dams**
   Provides standardized procedures for periodic and post-event inspections, tailored for various dam types, including embankments.
3. **Manual for Assessing Structural Safety of Existing Dams**
   Offers methodologies for evaluating the structural integrity of dams, focusing on embankment-specific concerns such as seepage and slope stability.
4. **Manual for Rehabilitation of Large Dams**
   Details best practices for rehabilitating aging dams, addressing common issues like erosion and seepage in embankment structures.
5. **Guidelines for Preparing Operation and Maintenance Manuals for Dams**
   Assists dam owners in developing comprehensive O&M manuals, emphasizing routine maintenance and emergency procedures for embankment dams.
6. **Guidelines for Developing Emergency Action Plans (EAP) for Dams**
   Outlines the creation of EAPs to ensure preparedness and coordinated response in case of dam-related emergencies.
7. **Guidelines for Classifying the Hazard Potential of Dams**
   Provides a framework for categorizing dams based on potential risks, aiding in prioritizing safety measures.
8. **Guidelines for Mapping Flood Risks Associated with Dams**
   Focuses on identifying and mapping areas at risk of flooding due to dam operations or failures, crucial for embankment dams.
9. **Guidelines for Selecting and Accommodating the Inflow Design Floods for Dams**
   Assists in determining appropriate inflow design floods, ensuring dams can safely handle extreme hydrological events.
10. **Guidelines for Instrumentation of Large Dams**

Details the selection, installation, and maintenance of instruments to monitor dam behaviour, crucial for early detection of issues in embankment dams.

11. **Guidelines for Mapping Flood Risks Associated with Dams**
    Provides methodologies for assessing and mapping potential flood zones due to dam operations or failures, aiding in risk mitigation planning.
12. **Manual for Assessing Hydraulic Safety of Existing Dams – Volume I & II**
    Offers procedures for evaluating the hydraulic aspects of dam safety, including spillway capacity and flood handling capabilities.
13. **Guidelines for Assessing and Managing Environmental Impacts in Existing Dam Projects**
    Addresses the environmental considerations in dam operations, emphasizing sustainable practices and compliance with environmental regulations.
14. **Inspection Manual for Dam Field Engineers After Seismic Events**
    Provides a framework for post-earthquake inspections, ensuring structural integrity and safety of dams following seismic activities.
15. **Technical Specifications of Hydro-meteorological, Geodetic, Geotechnical, and Seismic Instruments**
    Outlines the technical requirements for instruments used in dam monitoring, ensuring standardized data collection and analysis.
16. **Basic Design Provisions for Existing Dams**
    Discusses design considerations for existing dams, including freeboard requirements and spillway capacities, to enhance safety and performance

10.3. CONCLUSION

Dams constitute perhaps the largest and the most complex of structures being built by mankind. They are designed to withstand all possible destabilizing forces with a certain factor of safety. There has been tremendous progress in design, construction, and maintenance of dams with evolving study methodologies, advanced construction techniques and upgraded safety standards. However, still failures are being experienced. Failures in old dams can be attributed to ageing to some extent, but those in new dams are alarming. Data suggests that most dam failures have occurred within first five years of construction. This calls for an introspection to examine and review our practices to safeguard these structures. With initiatives such as Dam Rehabilitation and Improvement Programme (DRIP) and the Dam Safety Act 2021, the Government of India has put forth a significant step in this direction.

Talking of earthen dams, they constitute the maximum number of dams built in India as well as worldwide. Being constructed with soil, which is inherently a complex and heterogeneous material; the design, construction, maintenance, and rehabilitation of earthen dams is relatively complex. It requires an elaborate study-based approach covering various aspects such as seepage, static stability and dynamic stability to implement remedial measures for rehabilitation.

Though software and numerical tools are commonly available today, undertaking these studies is a skilled task involving fair amount of experience and judgement in every stage such as carrying out geotechnical investigations, selection of input design parameters, using appropriate study methodology, drawing inferences of studies, correlating study results with site observations and recommending the most suitable site-specific remedial measures.

Through this Technical Memorandum an attempt is made to elaborate various aspects of the study-based approach being implemented at CWPRS. It is also emphasized that a comprehensive geotechnical investigation and effective instrumentation data serves as the backbone for designing effective rehabilitation strategies and mitigating potential failure mechanisms.

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