सत्यमेव जयते

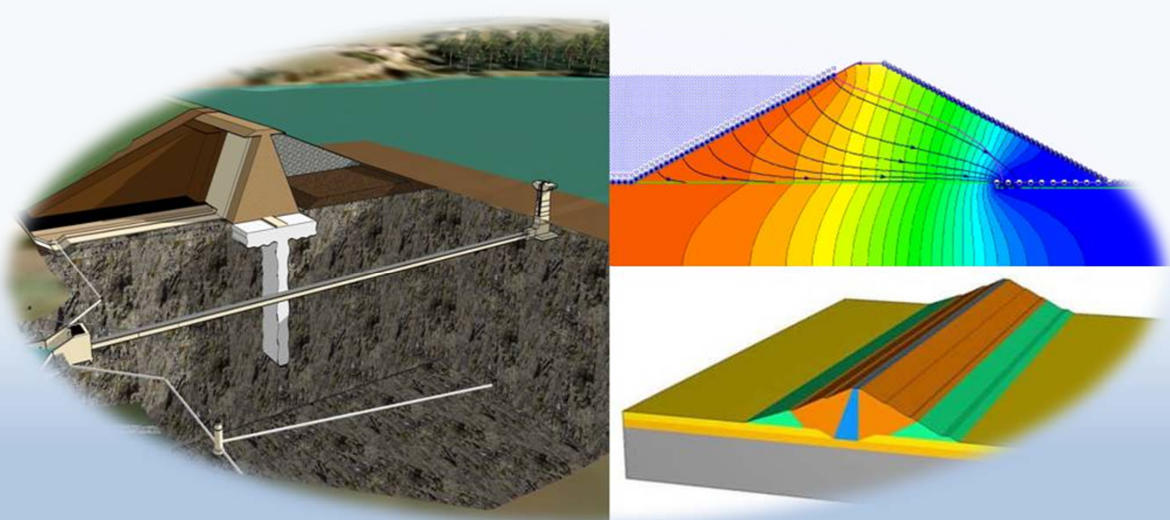
भारत सरकार
जल शक्ति मंत्रालय
जल संसाधन, नदी विकास और गंगा संरक्षण विभाग
केन्द्रीय जल आयोग
राष्ट्रीय जल अकादमी

केन्द्रीय जल अभियांत्रिकी सेवा के नव नियुक्त अधिकारियों का इकत्तीसवां प्रवेशन प्रशिक्षण कार्यक्रम

19 August 2019 – 07 February 2020

डिजाइन और अनुसंधान

Module II : Analysis and Design Aspects of Embankment Dams



पुणे
नवंबर 2019

Government of India
Central Water Commission
National Water Academy

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Module-II

ANALYSIS AND DESIGN ASPECTS OF EMBANKMENT DAMS

25-29 November 2019

Module Co-ordinator

Shri S N Pande, Director (Designs)

Pune
November 2019

Module – II

Analysis and Design Aspects of Embankment Dams

INDEX

Embankment Dams

1. Introduction

- Water conservation and development of water resources for irrigation have attracted human ingenuity since time immemorial.
- The oldest known works dates from 504 B.C. in Ceylon (now Sri Lanka).
- In India, the Grand Anicut across the Cauvery River, built more than 1600 years ago is still in service, providing irrigation to 0.4 million hectares of land in the Cauvery delta.
- In the past, design of the earthen embankments was mainly carried by the rule of thumb and judgment of the designer.
- The heights adopted were modest. Most of the embankments constructed earlier to 1900 AD were of homogeneous type using locally available soils compacted by animal force.
- Puddle filled trenches and lime concrete or masonry core walls were usually provided.
- In the post-independence period there has been a great stride in water conservation and development resulting in development of technology and design and construction of earthen embankments.
- Dams have been built across rivers by mankind right from the dawn of civilization for storing the water during rainy season and releasing it afterwards for other purposes: e.g. domestic, irrigation, flood control and generation of hydro power.
- With the growth of population all these functions of dams and storages have assumed greater significance.
- Dams constitute perhaps the largest and the most complex kind of structures being built by civil engineers.
- Are built to last from 100 to 300 years depending upon merits of each case.
- Designed to withstand all the possible destabilizing forces with a certain factor of safety which has been an indicator of a factor of ignorance or lack of knowledge of various response processes of materials used in construction, the stresses caused, the strains experienced and finally the failure mechanism.
- Large dams store very large volumes of water, therefore, has to be extra safe so that there is a minimum probability of their failure and consequent rapid or sudden release of storage which can cause havoc on the downstream.

2. Types of Embankment Dams

- Homogeneous Embankment
- Zoned Embankment
- Rock-fill Dams with clay cores
- Rock-fill Dams with u/s Face Membranes

a) HOMOGENEOUS EMBANKMENT:

- In this type of embankment, the dam section entirely consists of almost one type of material.
- It is adopted due to compulsions of material availability within a reasonable distance.

- A zoned section is always preferable, if materials in the two broad categories of ‘impervious or semi-pervious’ and ‘pervious’ are available.
- Usually this type of section is made of low permeability material and requires flatter slopes than a zoned section.

b) ZONED EMBANKMENT:

- This type of embankment uses two or more types of materials, depending on their availability, utility and costs.
- There is an impervious zone called the ‘core’ inside the dam section.
- The outer zones on both sides, called ‘shells’, should preferably be of pervious materials.
- If different grades of pervious material are available, the coarser or more pervious materials are placed on the outer faces.
- The different zones are separated by filters and even if the material in the shell is not pervious enough, it may still be necessary to provide internal drainage.

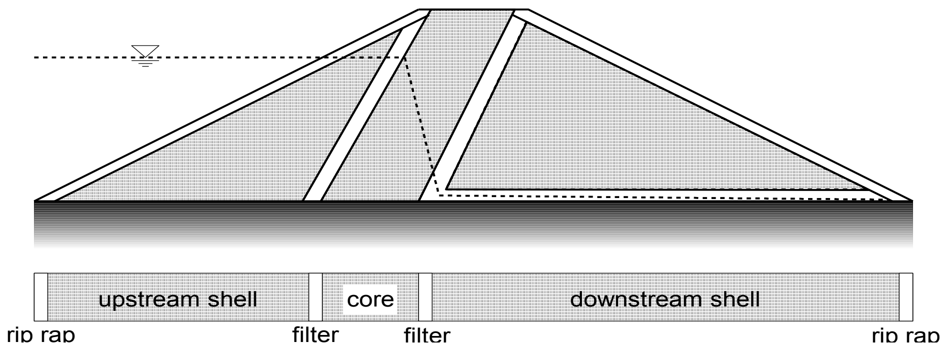
c) ROCK FILL DAMS WITH EARTH CORES

- Any dam which relies on fragmented rock material, either obtained by blasting or available as natural boulder deposits, as a major structural element is called a rock fill dam.
- Rock fill dams with earth cores usually have substantial rock fill zones on both sides, with an impervious zone in the middle, and transition zones and /or filters in-between.
- There may be further zoning by material type gradation or degrees of compaction within each category also.
- Good quality rock fill provides free drainage and high shear strength and most of the highest embankment dams are of this type.
- This type is basically similar to zoned earth embankment.

3. Embankment Dams:

- Embankment dams are defined as those constructed primarily of the natural materials of the earth, namely soil and rock.
- The principal vulnerability of an embankment dam is that it may be damaged or even destroyed if insufficient height or spillway capacity allows overtopping and erosion of the dam, or if uncontrolled seepage results in internal erosion of the embankment and its foundation.

Simple representation of earth dam system



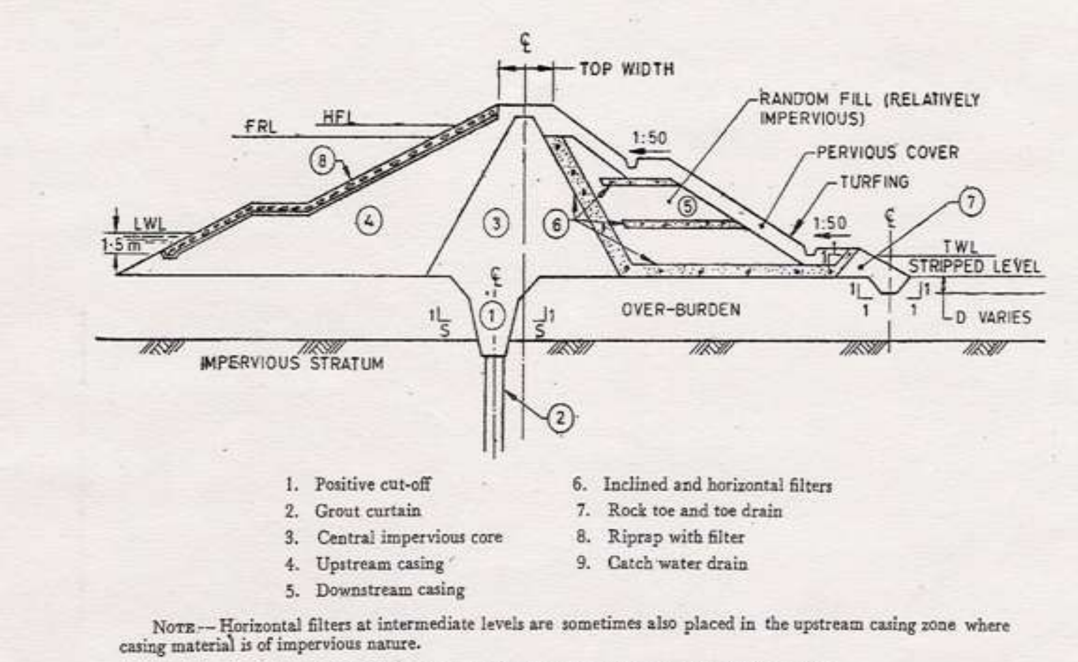
4. TERMINOLOGY :EMBANKMENT DAMS

- BORROW AREA – Source of construction materials for earth & rock fill dam.
- CASING – Zones except core in a zoned earthen dam, also called shell or shoulder.
- CORE – A zone of impervious earth within a zoned earth or rock fill dam.
- CUT-OFF – A barrier to reduce seepage of water through foundation and abutments.
- POSITIVE CUT-OFF – A cut-off taken to an impervious stratum. A full cut-off in the form of an open excavated trench and back filled with compacted impervious material, also provided in the form of sheet piles, plastic diaphragm, concrete diaphragm, grouted cut-off, cut-off wall, etc.
- PARTIAL CUT-OFF – A cut-off, which does not go down to impervious stratum.
- FULL RESERVOIR LEVEL (FRL) – The highest reservoir level that can be maintained without spillway discharge or without passing water through sluiceways. It dose not include any depth of surcharge.
- MINIMUM DRAW DOWN LEVEL (MDDL) – The lowest level to which a reservoir may be lowered keeping in view the requirements for hydro-power generation or irrigation and other requirements.
- FREEBOARD – The vertical distance between the crest of the embankment (without camber) and the maximum reservoir water level.
- HORIZONTAL FILTER – A layer of uniform or graded previous materials placed horizontally.
- INCLINED/VERTICAL FILTER – A layer of uniform or graded pervious materials, placed inclined or vertical.
- IMPERVIOUS BLANKET – An upstream impervious soil layer laid over a relatively pervious stratum and connected to the core.
- RELIEF WELL – Relief Wells are generally provided downstream of partial cut-off to relieve excess hydrostatic pressure.
- RIPRAP – It is the protection to the embankment material against erosion due to wave action, velocity of flow, rain-wash, wind action, etc. provided by placing a protection layer of rock fragments or manufactured material.
- TURFING – It is a cover of grass grown over an area to prevent erosion of soil particles by rain wash.
- TOE DRAIN – A trench filled with filter material along the down-stream toe of an earthen dam to collect seepage from horizontal filter and lead it to natural drain.
- PARAPET WALL– A wall provided along the edge of the embankment.
- PORE PRESSURE – The pressure developed in the fluid within the voids of the soil under external force when drainage is prevented.
- ROCK TOE – A zone of free draining material provided at the toe of the dam.

5. COMPONENTS OF EMBANKMENT DAM

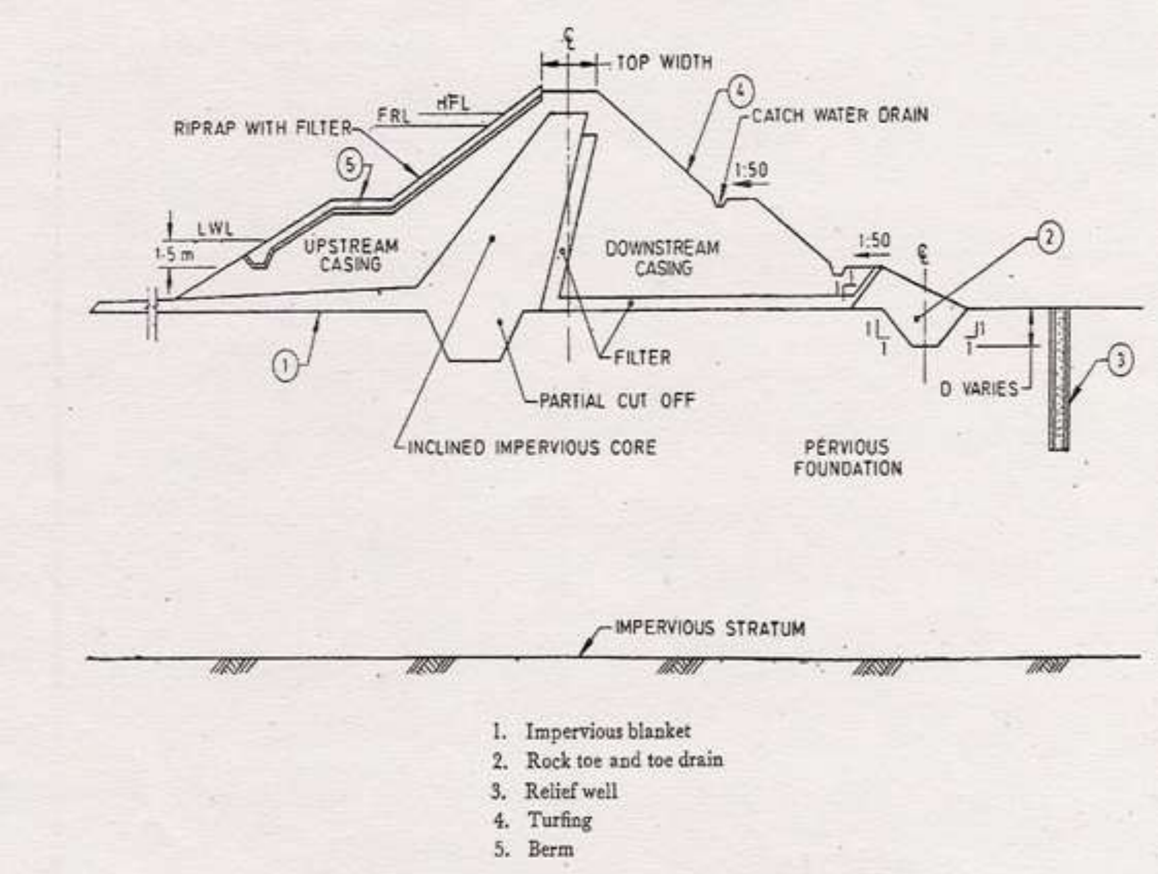
- An embankment dam generally consists of the following components:
- Cut-off
- Core
- Casing
- Internal Drainage system and foundations
- Slope protection
- Surface drainage
- The following components are provided in special cases:
- Impervious blanket
- Relief well.

Earthen Dam with Central Core & Positive Cutoff



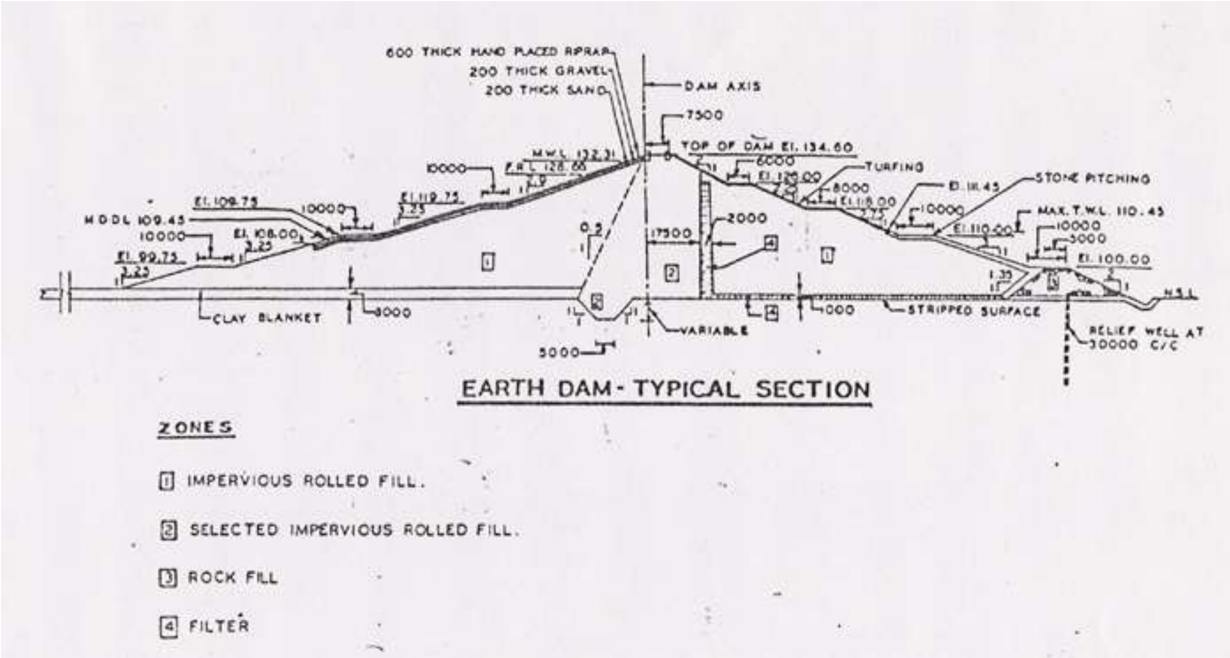
EARTH DAM WITH CENTRAL CORE AND POSITIVE CUT-OFF

Earthen Dam with Central Core & Partial Cutoff



EARTH DAM WITH INCLINED CORE AND PARTIAL CUT-OFF

Embankment Dam: Typical Section



6. FUNCTIONS AND DESIGN REQUIREMENTS OF COMPONENTS

CUT-OFF: is required for the following functions:

- To reduce loss of stored water through foundations and abutments.
- To prevent subsurface erosion by piping.
- The type of cut-off should be decided on the basis of detailed geological investigation.
- It is desirable to provide a positive cut-off. Where this is not possible, partial cut-off with or without upstream impervious blanket may be provided.
- Cut-off may be in the form of trench, sheet piling, cement bound curtain, diaphragm of bentonite, concrete or other impervious materials.
- The alignment of the cut-off should be fixed in such a way that it's central line should be within the base of the impervious core.
- In case of positive cut-off, it should be keyed at least to a depth of 400 mm into continuous impervious sub-stratum.
- The partial cut-off is specially suited for horizontally stratified foundations with relatively more pervious layer near top. The depth of the partial cut-off in deep pervious alluvium will be governed by:
  - ✓ Permeability of substrata
  - ✓ Relative economics of depth of excavation governed usually by cost of dewatering versus length of upstream impervious blanket

CORE: provides impermeable barrier within the body of the dam.

- Impervious soils are generally suitable for the core. However, if possible, soils having high compressibility and liquid limit and having organic content may be avoided, as they are prone to swelling and formation of cracks.
- Table-A gives recommendations for suitability of soils for construction of earth dam based on IS: 1498-1970.
- Recommendation regarding suitability of soil for construction of core for earth dam in earthquake zones are given in Table-B as per IS: 1498-1970.

- The core may be located either centrally or inclined upstream.
- The locations will depend mainly on the availability of materials, topography of site, foundation conditions, diversion consideration, etc.
- The main advantage of a central core is that it provides higher pressure at the contact between the core and foundation reducing the possibility of leakage and piping.
- Inclined core reduces the pore pressure in the downstream part of the dam and thereby increases the safety.
- Also permits the construction of down -stream casing ahead of the core.
- The following practical considerations governs the thickness of the core:
    - Availability of suitable impervious material
    - Resistance to piping
    - Permissible seepage through dam
    - Availability of other materials for casing, filter, etc.
    - Minimum width that will permit proper construction
- The minimum top width of core should be 3 m with thickness at any height not less than 30 percent (preferably not less than 50%) of maximum water head at that height.
- The top level of the core generally should be fixed at 0.5 m above MWL.

7. BASIC DESIGN REQUIREMENTS OF EMBANKMENT DAMS

The basic requirements for design of embankment dam are to ensure:

- Safety against overtopping
- Slope Stability
- Safety against internal erosion
- Phreatic line within down stream face
- Safety against wave action

8. SAFETY AGAINST OVERTOPPING

- Sufficient spillway and outlet capacity should be provided to prevent overtopping of earth embankment during and after construction.
- The freeboard should be sufficient to prevent overtopping by waves and should take into account the settlement of embankment and foundation.
- Analysis should be made for computing the settlement of the embankment and of the foundation in order to determine extra freeboard to be provided as settlement allowance.
- For compressible foundation, the settlement should be computed based on laboratory test results and should be provided for by increasing the height of dam correspondingly.
- Longitudinal camber should be provided on the top of dam along the dam axis to provide for settlement.
- The camber varies from zero height at the abutments to maximum at the central section in the valley where maximum settlement is anticipated.

SLOPE STABILITY

- The slopes of the embankment shall be stable under all loading conditions and flat enough so as not to impose excessive stress on foundation.
- Slopes shall be designed as per the provisions contained in IS: 7894-1975.
- The u/s slope shall be protected against erosion by wave action and the crest and d/s slope shall be protected against erosion due to wind and rain.

SAFETY AGAINST INTERNAL EROSION

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

PHREATIC LINE WITHIN D/S FACE:

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

SAFETY AGAINST WAVE ACTION:

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

9. WORLD'S HIGHEST EMBANKMENT DAMS

S.N.	NAME	COUNTRY	TYPE	HEIGHT(m)
1.	ROUGN	CIS	EARTH / ROCKFILL	335
2.	NUREK	CIS	EARTH	317
3.	CHICOASEN	MEXICO	ROCKFILL	261
4.	TEHRI	INDIA	EARTH / ROCKFILL	261
5.	GUAVIO	COLUMBIA	ROCKFILL	243
6.	MICA	CANADA	EARTH	242
7.	CHIVOR	COLUMBIA	ROCKFILL	237
8.	OROVILLE	USA	EARTH	230
9.	SAN ROQUE	PHILLIPINES	ROCKFILL	210
10.	KEBAN	TURKEY	ROCKFILL	207
11.	RAMGANGA	INDIA	EARTH	126

10. CAUSES OF FAILURE OF..EMBANKMENT DAM

- Overtopping 30%

Seepage effect, Piping and Sloughing	25%
Slope Slides	15%
Conduit Leakage	13%
Damage to slope Paving	5%
Miscellaneous	7%
Unknown	5%

11. LIST OF CODES OF BUREAU OF INDIAN STANDARDS FOR DESIGN OF "EMBANKMENTS DAMS"

- IS:8826-1978 Guidelines for designs of large earth and rockfill dams.
- IS:10635-1993 Guidelines for free board requirement in Embankment dams.
- IS:7894-1975 Code of practice for stability Analysis of earth dam.
- IS:1498-1970 Classification and identification of soil for general Engineering Purposes.(Amendment No.-1 in Aug.'1982 & Reaffirmed in 1987)
- IS:1893-1984 Criteria for earthquake resistant design of structure.
- IS:4999-1991 Recommendation for grouting of pervious soil.
- IS:5050-1992 Code of practice for design, construction and maintenance of relief wells.
- IS:5529-1985 Code of practice for insitu permeability test.(Part-1 Test in overburden)
- IS:5529-1985 Code of practice for in-situ permeability test.(Part-2 Tests in bed rocks)
- IS:6066-1994 Recommendations for pressure grouting of rock foundation in River Valley projects.
- IS:8237-1985 Code of practice for protection of slope for reservoir embankment.
- IS:8414-1977 Guidelines for design of Under-seepage control-measures for Earth and rockfill dams.
- IS:11293-1985 Guidelines for design of Grout curtain.(Part-1 Earth and Rockfill dam)
- IS:9429-1980 Code of practice for drainage system for earth and rockfill dam.
- IS:11973-1986 Code of practice for treatment of rock foundation, core abutment contacts with rock for embankment dams.
- IS:14343-1996 Choice of grouting material for alluvial grouting – Guidelines.
- IS:14344-1996 Design and construction of diaphragms for under seepage Control. – Code of practice.
- IS: 12584-1989 Bentonite for grouting in civil engineering works –Specification.

12. FREEBOARD IN EMBANKMENT DAMS

Especially in Embankment Dams the overtopping causes an instant failure of the dam. Therefore sufficient margin is provided between the crest of embankment and the still reservoir water surface to prevent this overtopping .

Free Board Provides safety against many contingencies such as :

1. Occurrence of a flood somewhat larger than the design flood.
2. Malfunctioning of the gates of the spillway & outlets.
3. Settlement of the Dam greater than the anticipated etc.

FREEBOARD

It is the vertical distance between the crest of embankment (excluding camber) and the still reservoir water surface. Freeboard mainly can be classified in two categories:

1. Normal Freeboard
2. Minimum Freeboard

- NORMAL FREEBOARD

It is the freeboard above the full reservoir level (FRL).

- MINIMUM FREEBOARD

It is the freeboard above the maximum water level (MWL) worked out for designed inflow flood (DIF).

13. Terminology used for Freeboard computation

- Design Wave Height

  It is that wave height which the structure is designed to withstand so that it does not undergo more than the accepted probability of damage , should the same wave height be exceeded. It is a suitable multiple of the significant wave height depending on the degree of risk to be accepted.
- Fetch length

  It is the straight line distance along the wind direction (along central radial of fetch) over open water on which the wind blows.
- Effective Fetch

  It is the weighted average fetch length of water spread , covered by 45 degree angle on either side of trail – fetch (assuming the wind to be completely non-effective beyond this area) and measured in a direction parallel to the central radial line of the trail fetch.
- Maximum Wave Height

  It is the average wave height of highest one percent of waves in a representative spectrum.
- Significant Wave Height

  It is the average wave height of the highest one third of the wave percent in each sampling interval.
- Wave Length

  It is the length in meters from crest to crest for significant wave.
- Wave Period

  It is the average interval in seconds between successive crests or troughs of significant waves.
- Wave Run -Up

  It is the difference (vertical height) between maximum elevation attained by wave run–up on a slope and the water elevation on the slope excluding wave action.
- Wave Set-Up

  When wind blows over a water surface it exerts a horizontal force on the water surface driving it in the direction of the wind. This effect results in piling up of the water on one shore of the lake or reservoir .The magnitude of rise above the still reservoir water surface is called “wind set-up” or “wind-tide”.

14. Factors Considered For Freeboard Estimation.

- Wave characteristics, particularly wave height and wave length.
- Height of wind set-up above the still water level adopted as freeboard reference elevation
- Slope of the dam and roughness of the pitching.

- <u>**Important Note**</u>

- Freeboard requirement does not account for effect of
- Earthquake
- Settlement of dam
- Dam foundation
- Earthquake Seiches

15. Method For Freeboard Computations

Out of the available methods for freeboard computations/ assistance has been derived from <u>"T. Saville's"</u> method, which is widely used for free-board computation of embankment dams. The details of the procedure to be followed for computation for freeboard are discussed later .

<u>The freeboard should be calculated for following conditions</u>:-

1. Normal freeboard that is at FRL.
2. Minimum freeboard that is at MWL.

- <u>Normal Freeboard</u>

  While calculating normal freeboard at FRL, full wind velocity should be adopted . The design wave height (Ho) be taken as 1.67 times the significant wave height (Hs). Normal freeboard should not be less 2.0m.

- <u>Minimum Freeboard</u>

  While calculating minimum freeboard at MWL half to two third wind velocity should be adopted . The lower values may be adopted in regions where maximum wind velocities occur during the period when water level in the reservoir is at or below FRL. This freeboard should be subject to a minimum of 1.5m . The design wave height (Ho) be taken as 1.27 times the significant wave height (Hs).

- The freeboard which gives the highest requirement of TBL (Top Bund Level) should finally be adopted.
- 1.0m high parapet wall may be provided in all embankment dams but the same is not to be considered as a part of freeboard. The freeboard which gives the highest requirement of TBL (Top Bund Level) should finally be adopted.

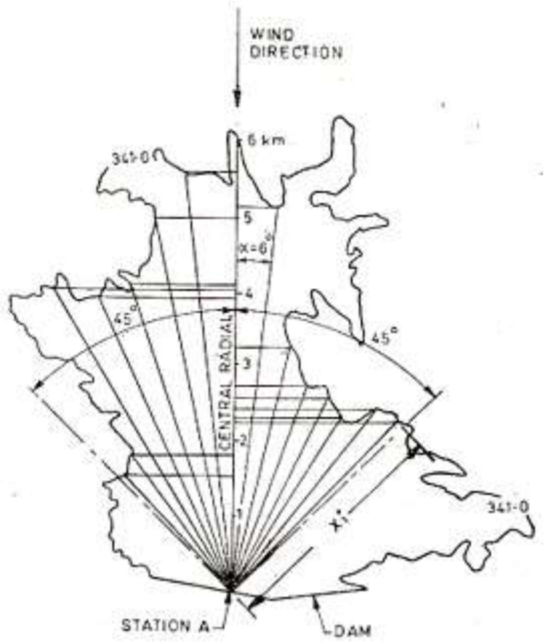
16. Procedure for Computation of Freeboard for Embankment Dams

1. <u>Normal Freeboard</u>

- Select a line "A-B" , with "A" on Dam axis and "B" on FRL contour.
- So as to cover the maximum reservoir water spread area within 45 degrees on either side of line AB (fetch length).
- Draw seven radials at six degrees interval on each side of line "A-B" and compute effective fetch (F_e) for FRL by the following formula :

$$F_e = \frac{\Sigma X_1 \cos\alpha\ (\cos\alpha)}{\Sigma \cos\alpha}$$

- Where X_1 denotes the length of any radial which is at an angle the central radial.



- If felt necessary more trials may be done so that maximum effective fetch may be computed. Enter effective fetch (F_e) as step (1) .
- From Fig. 1 of IS 875 (Part3):1987 read basic wind speed on land for 50years return period (U) for region in which proposed dam falls. Enter wind velocity on land (U) as step (2).

- **Compute wind velocity on water surface (V) , by multiplying coefficient Q from Table (Wind Velocity Relationship Land to Water)**

<u>Effective Fetch in "km"</u>	<u>Coefficient "Q"</u>

1	1.10
2	1.16
4	1.24
6	1.27
8	1.30
10 and above	1.31

- Corresponding to effective fetch to the wind velocity on land (U) . Enter Q and wind velocity on water surface as steps (3) and (4) respectively.

- Using relationship given below or graphical diagram shown in Fig.-2 of IS 875 (Part3):1987 graph between compute significant wave height (H_s):

  $g. H_s / v^2 = 0.0026 (g. f_e / v^2)^{0.47}$

Enter Significant wave height (H_s) as Step 5.

- Using relationship given below or graphical diagram shown in Fig.-3, of IS 875 (Part3):1987 Compute wave period (T_s):
  $g. T_s / v = 0.45 (g f_e / v^2)^{0.28}$

Enter T_s as Step 6.

- Compute wave length (L_s) with following relationship:
  $L_s = 1.56 T_s^2$
  Enter L_s as Step 7.

- Compute design wave height H_o, with the relationship:
  $H_o = 1.67 H_s$
  Enter H_o as Step 8.

- Work out steepness ratio H_o / L_s, With the help of curves given in Fig.-4 of IS 875 (Part3):1987 between different values of steepness ratio and the Embankment slopes read R/H_o ratio, and compute Wave run-up on smooth surface (R). The wave run–up on rough surface (Ra) may be computed by multiplying surface roughness coefficient, given in Table , to the wave run up on smooth surface (R).
  - Enter H_o / L_s as Step 9.
  - Enter R/H_o as Step 10.
  - Enter (R) as Step 11.
  - Enter (Ra) as Step 12.

S.No.	Type of Pitching	Recommended coefficient
1	Cement Concrete Surface	1.0

2	Flexible brick pitching	0.8
3	Hand placed rip rap (a) Laid flat (b) Laid with projection	0.75 0.60
4	Dumped rip rap	0.50

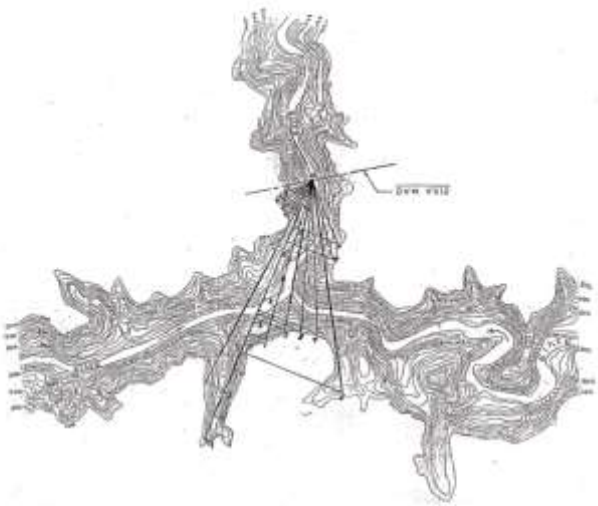
- Calculate average water depth (D) along fetch length (F). Enter average reservoir depth (D) as Step 13.
- Compute wind set–up (S) from the formula:

  $s = V^2 . F/ 62000 D.$

  If wind set–up as calculated above is higher than the average depth of water , the value of wind set-up should be limited to average depth of water. Enter wind set–up (S) as Step 14
- Compute Freeboard as step (12) + step (14). Enter as Step 15.
- If freeboard calculated in step (15) is less then 2.0 meters , if so provide at least 2.0 meters freeboard. Enter required freeboard as Step 16.

  Minimum Freeboard at MWL

  For obtaining minimum freeboard at MWL repeat above procedure by calculating fetch length (F) and effective fetch (Fe) at MWL. Half to two –third wind velocity on land and effective fetch at MWL may be adopted for different calculations using above steps.
- Check, if minimum freeboard is less than 1.5 m and if so, provide at least 1.5 m freeboard.
- Fixing of TBL:- Calculate the TBL required for the following conditions and enter as step (17).
- FRL+ Normal freeboard (not less than 2.0 m)
- MWL + Minimum freeboard (not less than 1.5 m).
- Adopt the highest of the above two values as TBL.



Suitability of soils for construction of Earthen Dams (IS: 1498-1970) : Table A

Relative Suitability	Homogeneous Dykes	Zoned Earth Dam		Impervious Blanket
		Imperious	Pervious	
Very suitable	GC	GC	SW, GW	GC
Suitable	CL, CI	CL, CI	GM	CL, CI
Fairly suitable	SP, SM, CH	GM, GC, SM, SC, CH	SP, GP	CH, SM, SC, GC
Poor	-	ML, MI, MH	-	-
Not suitable	-	OL, OI, OH, Pt	-	-

- **GW**- Well graded gravel, **GP**- Poorly graded gravel,
- **GM**- Silty gravel, **GC**- Clayey gravel,
- **SW**- Well graded sand, **SP**- Poorly graded sand,
- **SM**- Silty sand, **SC**- Clayey sand,
- **CL**- Clayey and silty of low compressibility, **CI**- Clayey and silt with medium compressibility,
- **CH**- Clayey and silt with high compressibility.

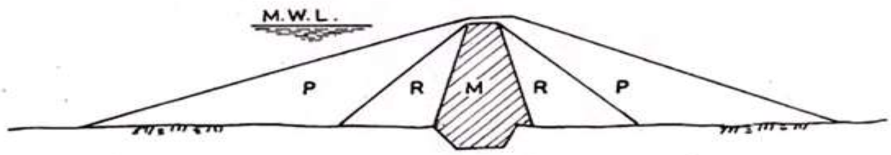
Soils for Core in Earthquake Zones: Table-B

S. No	Relative Suitability	Type of soil
1.	Very Good	Very well graded coarse mixtures of sand, gravel and clayey fines, D_{85} coarser than 50mm, D_{50} coarser than 6mm. If fines are cohesion less, not more than 20 % finer than 75 micron IS Sieve.
2.	Good	a) Well graded mixture of sand, gravel and clayey fines, D_{85} coarser than 25mm. Fines consisting of inorganic clay (CL with plasticity index greater than 12). b) Highly plastic tough clay (CH with plasticity index greater than 20).
3.	Fair	a) Fairly well graded, gravelly, medium to coarse sand with cohesion less fines. D_{85} coarser than 19mm, D_{50} between 0.5mm and 3.0mm. Not more than 25% finer than 75 micron IS Sieve. b) Clay of medium plasticity (CL with plasticity index greater than 12).
4.	Poor	a) Clay of low plasticity (CL and ML) with little coarse fraction. Plasticity index between 5 and 8. Liquid limit greater than 25. b) Silts of medium to high plasticity (ML or MH) with little coarse fraction. Plasticity index greater than 10 c) Medium sand with cohesion less fines.

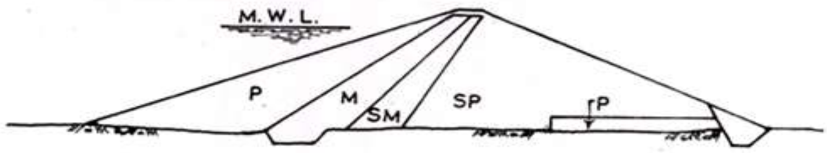
5.	Very Poor	a) Fine, uniform, cohesion less silty sand, D85 finer than 0.3mm. b) Silt from medium plasticity to cohesion less (ML). Plasticity index less than 10.

- **The expansive, dispersive soil shall not be used in the construction of the embankment dam.**
- D_{85} – The 85% size (85% is finer) in millimeters as determined by a grain size analysis.
- D_{50} – The 50% size (50% is finer) in millimeters as determined by a grain size analysis.

DAM ON IMPERVIOUS FOUNDATIONS

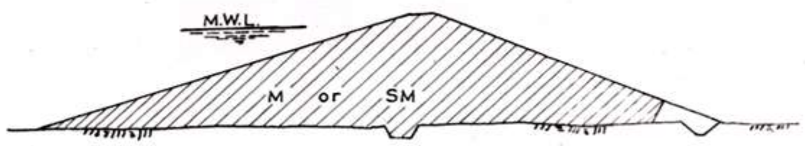


CENTRAL CORE TYPE-SUITABLE TO HIGH & MODERATELY HIGH DAMS

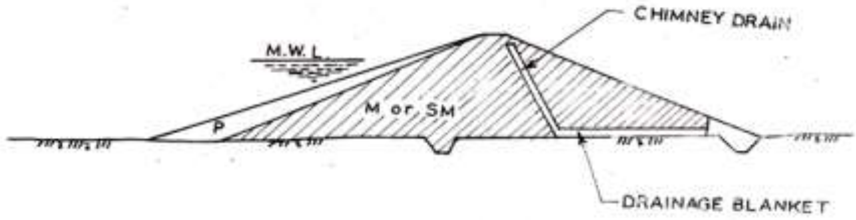


INCLINED CORE TYPE-PROVIDES WIDER WORKING ZONE IN CORE FOR LOW DAMS

M - IMPERVIOUS **P** - PERVIOUS
SP - SEMI-PERVIOUS **SM** - SEMI-IMPERVIOUS
R - RANDOM, INCLUDING SP OR SM



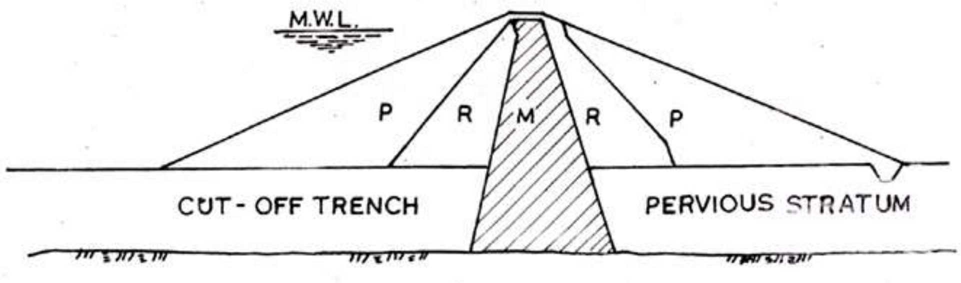
HOMOGENEOUS TYPE-PRINCIPALLY ONE TYPE OF MATERIAL AVAILABLE, REQUIRES RELATIVELY FLAT SLOPES



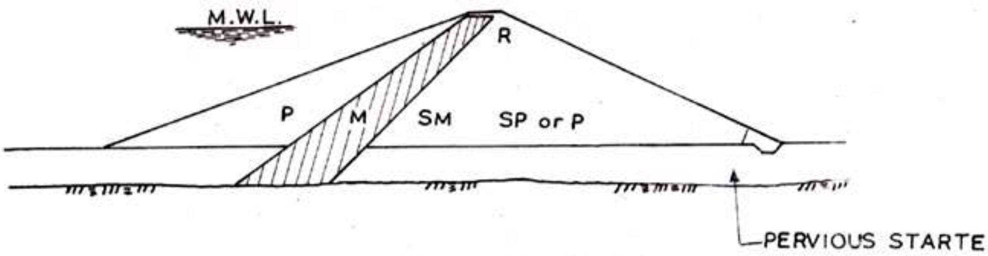
MODIFIED HOMOGENEOUS TYPE-USES LIMITED QUANTITY OF PERVIOUS TO CONTROL DRAWDOWN AS WELL AS STEADY SEEPAGE NEUTRAL PRESSURES

M - IMPERVIOUS **P** - PERVIOUS
SP - SEMI-PERVIOUS **SM** - SEMI-IMPERVIOUS
R - RANDOM, INCLUDING SP OR SM

DAMS ON SHALLOW PERVIOUS FOUNDATIONS

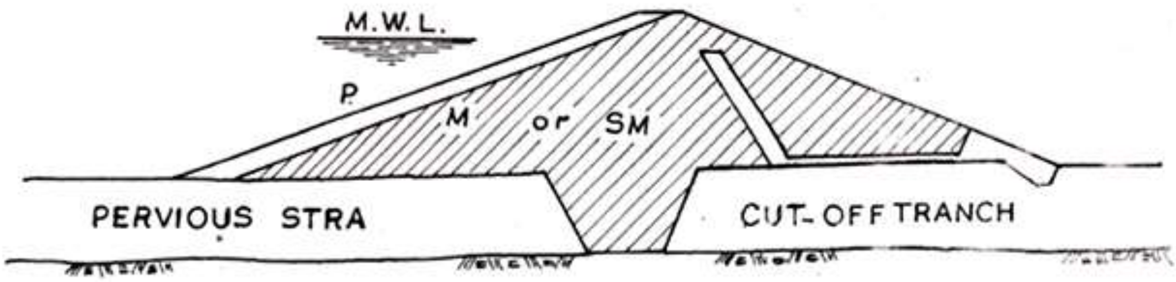


CENTRAL CORE TYPE-SUITABILITY AS FOR (I) CORE THICKNESS ADJUSTABLE ACCORDING TO AVAILABLE IMPERVIOUS



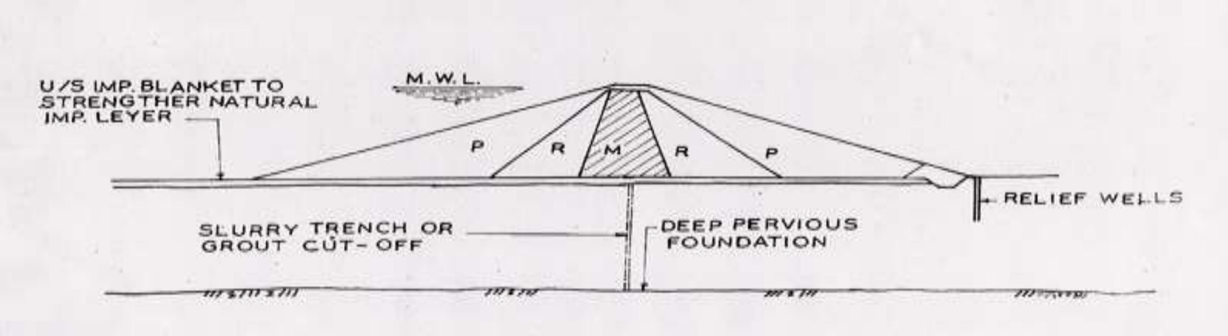
INCLINED CORE TYPE-SUITABILITY AS FOR (II)

M - IMPERVIOUS **P** - PERVIOUS
SP - SEMI-PERVIOUS **SM** - SEMI-IMPERVIOUS
R - RANDOM, INCLUDING SP OR SM



MODIFIED HOMOGENEOUS-SUITABILITY AS FOR (IV)

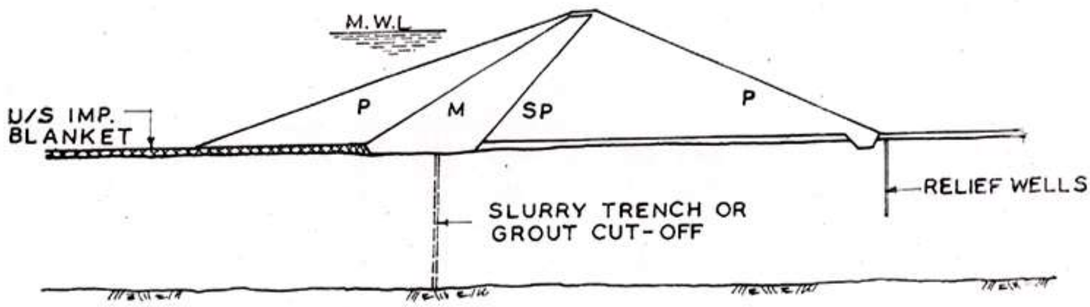
M - IMPERVIOUS **P** - PERVIOUS
SP - SEMI-PERVIOUS **SM** - SEMI-IMPERVIOUS
R - RANDOM, INCLUDING SP OR SM



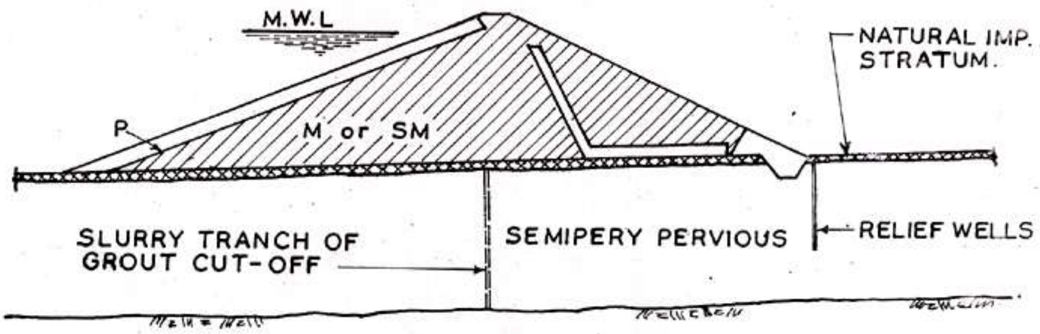
CENTRAL CORE TYPE-SUITABILITY AS FOR (I) POSITIVE CUT-OFF MAY REPLACE IMPERVIOUS BLANKET

M - IMPERVIOUS **P** - PERVIOUS
SP - SEMI-PERVIOUS **SM** - SEMI-IMPERVIOUS
R - RANDOM, INCLUDING SP OR SM

DEEP PERVIOUS FOUNDATIONS WITH IMPERVIOUS TOP STRTUM



INCLINED CORE TYPE-SUITABILITY AS FOR (II)



MODIFIED HOMOGENEOUS-SUITABILITY AS FOR (IV)

M - IMPERVIOUS **P** - PERVIOUS
SP - SEMI-PERVIOUS **SM** - SEMI-IMPERVIOUS
R - RANDOM, INCLUDING SP OR SM

- **Advantages of Central Core**

  ✓ Provides higher pressure on the contact surface between the core and the foundation, thus reducing the possibility of hydraulic fracturing.
  ✓ For a given quantity of soil, the central core provides slightly greater thickness.
  ✓ Provides better facility for grouting of foundation or contact zone or any cracks in the core if required afterwards, as this can be done through vertical rather than inclined holes.
  ✓ Foundation area is independent of depth of foundation and hence can be marked and treated in advance.

- **Disadvantages of Central Core**

  ✓ The advantages listed for a vertical core are not obtainable. Also, a moderately thick central core with pervious shells will result in a slightly flatter downstream slope of the dam.
  ✓ The problem of differential settlement between the core and the shell zone may result in cracking parallel to the dam axis.

- **Advantages of Slanting Core**

  ✓ D/S rockfill can be placed in advance and laying of filter, core and U/S zone can be taken up later. Ensures rapid progress, especially in conditions wherein core placement is possible only during part of the year.
  ✓ Foundation grouting of the core can be carried out while the downstream shell is being placed and thus better progress achieved.
  ✓ Since a very small part of the slip surface intersects the slanting core, the section is practically free from the steady seepage pore pressures and is thus more stable under a steady-state condition. This results in a steeper slope of the downstream shell and corresponding economy.
  ✓ Since the flow lines are essentially vertical and equipotential lines are almost horizontal under sudden drawdown, the drawdown pore pressures are very much reduced. However, a larger part of the slip surface for the upstream slope passes through the core material than would be the case with a central core.
  ✓ In the case of cracking of the core, the inclined core will leave a large mass of stable rockfill on the downstream side and is likely to be safer.
  ✓ Filter layers can be made thinner and placed more conveniently.

- **Disadvantages of Slanting Core**

  ✓ Contact area treatment becomes difficult : The depth of excavation of the foundation at the contact surface of the core is determined by the nature of the formations and cannot be predetermined in advance. Thus advance treatment of the contact area may present a problem in the case of a slanting core because if the depth of excavation increases, the contact area moves upstream.
  ✓ Upstream slope becomes flat: By slanting the core upstream, although the downstream slope can be made steeper, nevertheless, the upstream slope will generally become

flatter as the shear strength of the core material will be less than that of the pervious shell material; the advantage of reduced drawdown pore pressures may not compensate this factor. Thus any economy in total quantity of materials by adjustment of core position would depend on the relative strength of the two materials.

EXPANSIVE SOIL

- Expansive soils exhibit heaving with the increase in moisture content, exert swelling pressures on the structures restraining the heave.
- During summer, wide, deep and map type cracking is normally observed.
- Structure constructed using conventional methods exhibit heaving of floors, cracking of walls and jamming of doors during rainy season. Restraining structures get tilted and roads get rutted.
- Bed heaving and side slips and sloughing are noticed in canals.

DISPERSIVE SOIL

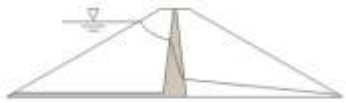
- Dispersive clays refer to clays in which physico-chemical state of the clay fraction of the soil is such as to cause individual clay particles to disperse and repel each other in the presence of relatively pure water.
- Highly erodible by water flow with low hydraulic gradients and tractive stresses.
- Non-cohesive silt, rock flour, and very fine sands also disperse in water and may be highly erodible.

CASING : The function of casing is to impart stability and protect the core. Relatively pervious materials, which does not crack on direct exposure to the atmosphere, are suitable for casing.

INTERNAL DRAINAGE SYSTEM: Comprises of inclined or vertical filter, a horizontal filter, a rock toe, a toe drain, etc.

- The design of filter consists of applying the conventional filter criteria, which take into account the grain size distribution and the shape of the grains.
- However, in addition to the grain size, the stability of the base soil adjacent to a given filter depends on its resistance to drag forces.
- Inclined or vertical filter together with the base filter, if required, is desirable to be provided especially to protect the core material from migration.
- Also filters help in controlling the leak through cracks in the core and subsequent repair by it self of the crack.
- However, the inclined or vertical filter may be avoided in zoned sections having pervious d/s shell and clayey cores but a transition filter between the core and the d/s shell would be necessary in case of dams where rock fill is used as shell material. Adequate toe protection shall, however, be provided.

Internal Drainage System

Toe drain	
Blanket drain	
Chimney and blanket drain	
Impermeable core and blanket	

17. Design of Filter

1. As per the Indian Standard Code (IS: 9429-1980) a properly designed filter should satisfy the following requirements:

- **It should be much more pervious than the protected base material.**
- **It should be of such gradation that particles of the base material do not migrate through or clog the filter voids.**
- **It should be sufficiently thick to provide a good distribution of all particle sizes throughout the filter.**
- **To satisfy the above requirements, the following filter criterion is recommended:**

1. $\frac{D15F}{D15B} > 4 \text{ and } < 20$

2. $\frac{D15F}{D85B} < 5$

3. $\frac{D50F}{D50B} < 25$

4. The gradation curve of the filter material should be nearly parallel to the gradation curve of the base material.

2. Other criteria for design of filter are as follows:

(i) Terzaghi Criteria:

$\frac{(D15) \text{ filter}}{(D85) \text{ protected soil}} \leq 4 \text{ to } 5$ (Piping criteria)

$\frac{(D15) \text{ filter}}{(D15) \text{ protected soil}} \geq 4 \text{to } 5$ (Permeability criteria)

(ii) USBR Criteria:

(i) For graded filters of angular particles :

$R\,50 = \frac{(D50) \text{ filter}}{(D\,50) \text{ base material}} = 9 \text{ to } 30$

$R15 = \frac{(D15) \text{ filter}}{(D15) \text{ base material}} = 6 \text{ to } 18$

(ii) For graded filters of sub rounded particles :

$R50 = \frac{(D50) \text{ filter}}{(D50) \text{ base material}} = 12 \text{ to } 58$

$R15 = \frac{(D15) \text{ filter}}{(D15) \text{ base material}} = 12 \text{ to } 40$

(iii) For uniform grain size filters :

$R50 = \frac{(D50) \text{ filter}}{(D50) \text{ base material}} = 5 \text{ to } 10$

The above criteria take into account only the grain size of base material, and is based on studies made with non-cohesive soils.

Even though the filters are provided generally to take care of the seepage through the pores of the embankment soils, they should also be capable of preventing erosion of soils through concentrated leaks that may occur in the dam body or at the foundation contact.

SLOPE PROTECTION:

U/S Slope

- The upstream slope protection is ensured by providing riprap.
- The riprap can be placed on the slope either by hand or it may be simply dumped.
- For design of the riprap, IS: 8237-1985 may be referred.
- The thickness of riprap in no case is less than 300mm.

Downstream Slope:

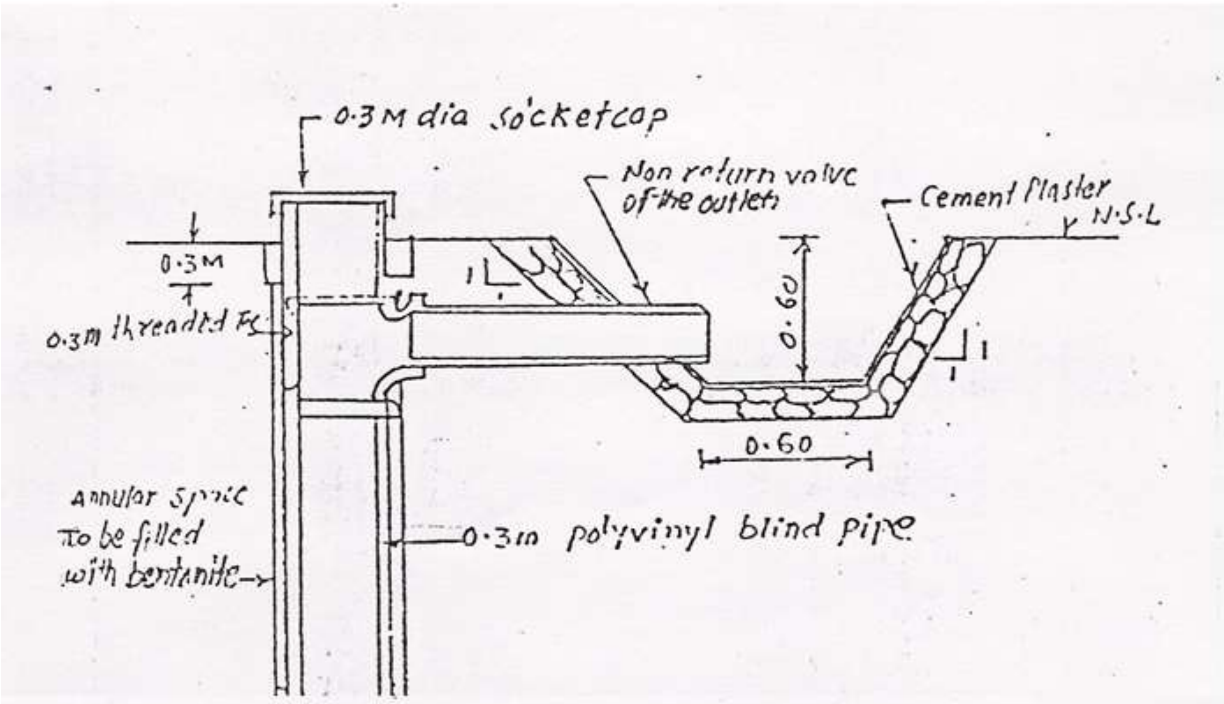
- The downstream slope protection is ensured by providing riprap or turfing.
- If the average annual rainfall (AAF) is less than 200cm, it is usual practice to protect the downstream slope from rain cuts by providing suitable turfing on the entire slope.
- In case if AAF is more than 200cm, 300mm thick riprap is provided.
- For details of d/s slope protection such as prevention from erosion by Rain-wash, prevention from erosion by tail water IS: 8237-1985 may be referred.

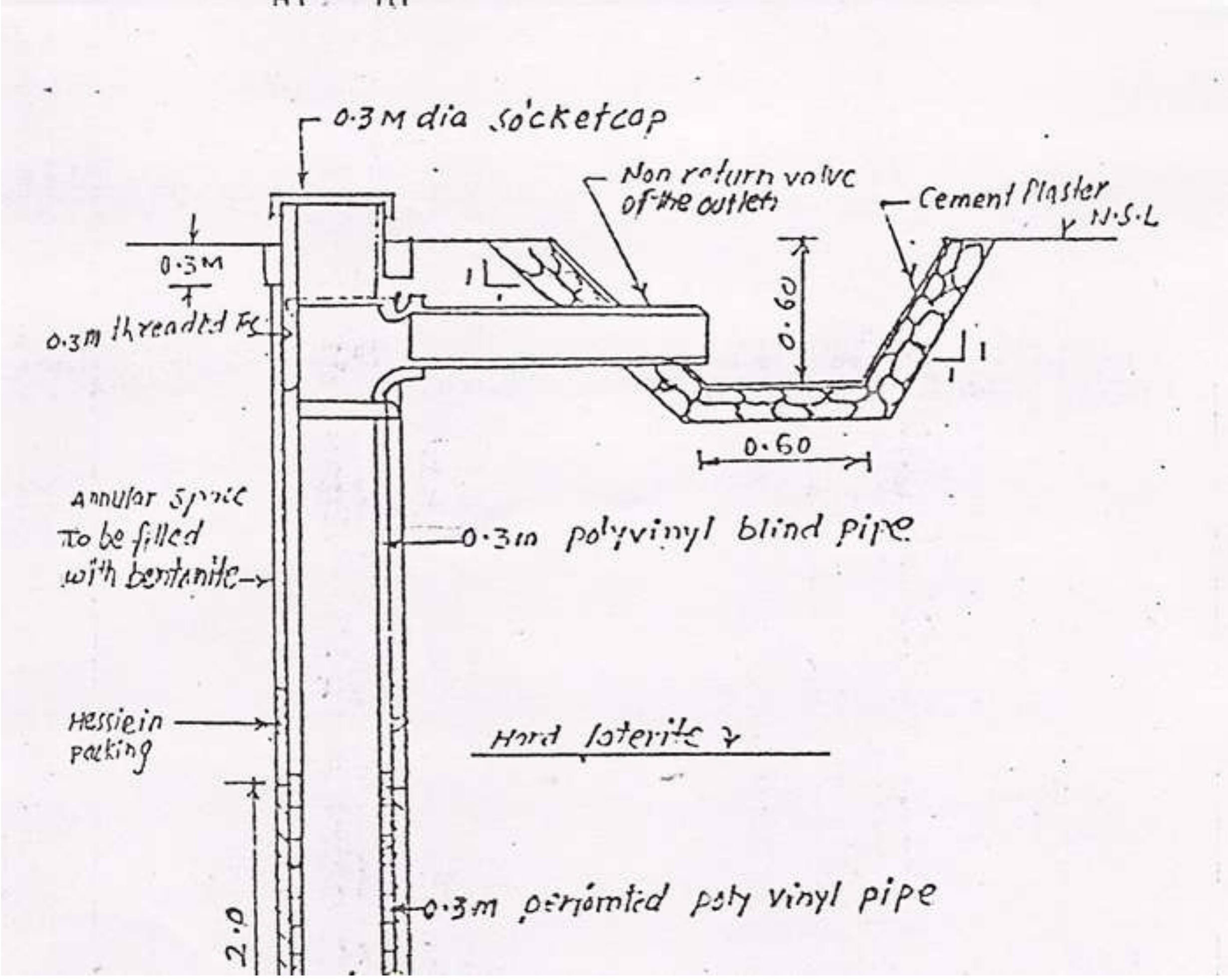
IMPERVIOUS BLANKET

- The horizontal u/s impervious blanket is provided to increase the path of seepage when full cut-off is not practicable on pervious foundations.
- The impervious blanket may be provided either with or without partial cut-off.
- Impervious blanket shall be connected to core of the dam.
- The material used for impervious blanket should have far less permeability than the foundation soil.
- To avoid formation of cracks, the material should not be highly plastic.
- 300 mm thick layer of random material over blanket is recommended to prevent cracking due to exposure to atmosphere.
- As a general guideline, impervious blanket with a minimum thickness of 1.0 m and a minimum length of 5 times the maximum water head measured from upstream toe of the core may be provided.
- The impervious blanket may be designed in accordance with IS: 8414.
- Reference may be made to IS: 1498 for suitability of soils for blanket.

RELIEF WELLS

- Consists of a pipe (10 to 15 cm dia) having narrow slots placed in the centre, surrounded by graded filter media (45 to 90 cm dia) sunk near d/s toe of earth dam.
- Permit the ingress of seepage water into the well allowing it to rise to the outfall (relief) level where the pressure gets relieved.
- Ensures safety of the earth dam when the cut-off is partial or reliance is placed on upstream blanket for controlling seepage,
- Controls the pressure developed below the d/s toe of the dam, especially when impervious layer of the soil at the top overlays a pervious layer.
- A system of relief wells suitably spaced reduces the intensity of the under seepage pressure and render it practically harmless.


0.3M dia socketcap
Non return valve of the outlet
Cement Plaster
N.S.L
0.3M
0.3m threaded Fs
0.60
0.60
Annular space To be filled with bentonite
0.3m polyvinyl blind pipe


0.3M dia socketcap
Non return valve of the outlet
Cement Plaster
N.S.L
0.3M
0.3m threaded Fs
0.60
0.60
Annular space To be filled with bentonite
0.3m polyvinyl blind pipe
Hessiein packing
Hard laterite
2.0
0.3m perforated poly vinyl pipe

18. ROCK FILL DAMS WITH EARTH CORES

- Any dam which relies on fragmented rock material, either obtained by blasting or available as natural boulder deposits, as a major structural element is called a rock fill dam.
- Rock fill dams with earth cores usually have substantial rock fill zones on both sides, with an impervious zone in the middle, and transition zones and /or filters in-between.
- There may be further zoning by material type gradation or degrees of compaction within each category also.
- Good quality rock fill provides free drainage and high shear strength and most of the highest embankment dams are of this type.
- This type is basically similar to zoned earth embankment.
- The rock for main rock fill should be hard, sound and durable so as to resist excessive breakdown during handling and placing operations.
- In general, un-weathered igneous and metamorphic rocks are suitable for rock fill, while sedimentary rocks are not desirable.
- Shale's (a type of rock) which slake in the presence of air and rocks which shatter into very small pieces or have high percentage of chips or dust are not suitable. The chips and dust should not be more than 10%.
- The angular bulky rocks are preferred as against flat elongated rocks or rounded boulders. If rounded cobbles or boulders are used, they should be scattered through out the rock fill and not concentrated in pockets.

19. ROCK FILLED EARTHEN DAMS

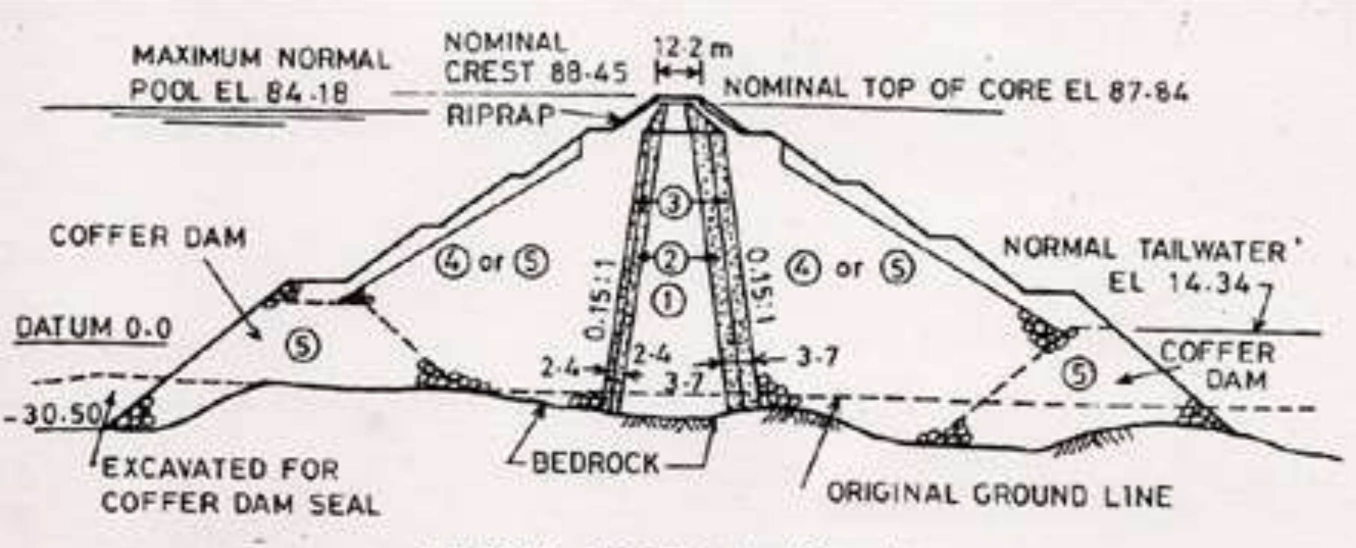


Fig. 12.22. Akosombo dam (Ghana).
1—core; 2—filter; 3—transition; 4—compacted rockfill; 5—dumped rockfill.

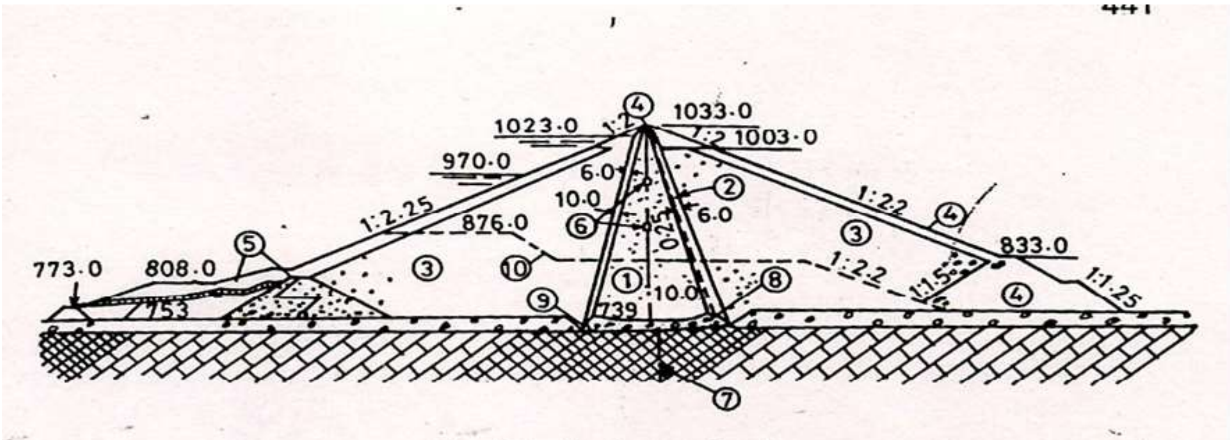


Fig. 12.4. Nurek dam, Russia.
1—rocky clay; 2—filter; 3—coarse gravel shells; 4—oversize rock surcharge on slopes; 5—upstream coffer dam; 6—inspection galleries for instrumentation; 7—grout curtain; 8—concrete block; 9—surface grouting; 10—first-stage construction.

20. CHOICE OF TYPE OF DAM

- <u>TYPE OF DAM</u>
    - Rigid
    - Flexible

- <u>CRITERION FOR CHOICE</u>
    - Foundation Geology
    - Construction Materials aspect
    - Valley shape
    - Construction Equipment
    - Cost
    - Safety

21. SELECTION OF DAM SECTION

No single type of cross-section of embankment dam is suited for all site conditions. The adoption of the particular type of embankment section depends upon the following factors:

- The designs of all dams in general and embankment dams in particular are site specific and cannot be generalized.
- The design has to be suited to valley shape and foundation geology, available materials and methodology of construction.

- As an important dam project involves a heavy investment, the designer is bound to study several feasible alternatives of dam height and reservoir capacity, layout of dam and ancillary works, and the dam section to meet the objective of optimum benefits.

- In the context of benefits, it is necessary to optimize the benefits for the river basin as a whole rather than for an individual project.

The adoption of the particular type of embankment section depends upon the following factors:

- Availability of the suitable local material in sufficient quantity within reasonable range.
- Foundation conditions and cut-off requirements.
- Types of construction, earth moving compaction machinery.
- Diversion consideration and construction schedule.
- Climatic conditions in relation to placement, moisture content control etc.
- Safety with respect to stability and seepage.

22. SPECIAL DESIGN REQUIREMENTS OF EMBANKMENT DAMS

In addition to basic design requirements, the following special design requirements, should also be satisfied for both earth and rock fill dams:

- Control of cracking
- Stability in earthquake regions
- Stability at junctions

23. CONTROL OF CRACKING

- Cracking of impervious zone results into a failure of an earth dam by erosion, piping, breaching, etc. Due consideration to cracking phenomenon shall, therefore, be given in the design of earth dam.
- Cracking in the core of earth or rockfill dam occurs due to foundation settlement and/or differential movements within the embankment.
- Differential movements in the embankment take place due to the following reasons:
    - Unsuitable and/or poorly compacted fill materials
    - Different compressibility and stress-strain characteristics of the various fill material
    - Variation in thickness of fill over irregularly shaped or steeply inclined abutments

24. MEASURES ARE RECOMMENDED FOR CONTROL OF CRACKING :

- Use of plastic clay core and rolling the core material at slightly more than optimum moisture content. In case of less plastic clay, 2 to 5 percent bentonite of 200 to 300 liquid limits may be mixed to increase the plasticity.
- Use of wider core to reduce the possibility of transverse or horizontal cracks extending through it.
- Careful selection of fill materials to reduce the differential movement. To restrict the rockfill in lightly loaded outer casings and to use well graded materials in the inner casings on either side of the core.
- Wide transition zones of properly graded filters of adequate width for handling drainage, if cracks develop.

- Special treatment, such as preloading, pre-saturation, removal of weak material, etc. to the foundation and abutment, if warranted.
- Delaying placement of core material in the crack region till most of the settlement takes place.
- Arching the dam horizontally between steep abutments.
- Flattening the downstream slope to increase slope stability in the event of saturation from crack leakage.
- Cutting back of steep abutment slopes

25. STABILITY IN EARTHQUAKE ZONES

- Dams situated in earthquake zones are likely to be subjected to additional stresses and deformation on account of earth acceleration. This needs a special treatment.
- Following are the additional factors to be considered while designing an earth dam in earthquake zones:
  - The stability of the slopes of the embankment under the extra forces set up by the lateral and vertical accelerations.
  - The settlement of loose or poorly compacted fill or foundation material leading to loss of freeboard and thereby possible overtopping.
  - The cracking of the impervious fill leading to possible failure by piping.
  - Liquefaction of deposits of loose sand in the foundation of the dam, causing cracking, sliding or actual horizontal movement of the dam.

26. MEASURES RECOMMENDED AGAINST THE EARTHQUAKE

- The stability analysis of slopes with earthquake consideration shall be carried out in accordance with the provisions contained in IS: 7894-1975.
- Additional freeboard shall be provided to avoid possible overtopping due to settlement of embankment or foundation or both during an earthquake.
- The provisions shall be made for discharging the maximum anticipated leakage rapidly. For this purpose, downstream zones of large quarried rock or screened gravels and cobbles are recommended.
- The impervious core shall be made thicker for resisting the piping action.
- The top of the dam should be made thicker by increasing the crest width or by using flatter slopes at the top than would be required in non-seismic regions, so as to increase the path of seepage through cracks.
- The foundation should be as compact as possible. All loose and soft material should be excavated and removed, if possible, or re-compacted.

27. STABILITY AT JUNCTIONS

Junctions or earthwork with foundation, abutments, masonry structures like overflow and non overflow dams and outlets need special attention with reference to one or all of the following criteria:

- Good bond between earthwork and foundation
- Adequate creep length at the contact plane
- Protection of earth dam slope against scouring action
- Easy movement of traffic

- Foundations on soils or non-rocky strata, vegetation like bushes, grass roots, trees, etc. should be completely removed.
- The soil containing organic material or dissoluble salt should also be completely removed. After removal of these materials, the foundation surface should be moistened to the required extent and adequately rolled before placing embankment material.
- For rocky foundation, the surface should be cleaned of all loose fragments including semi-detached and over-hanging surface blocks of rock. Proper bond should be established between the embankment and the rock surface so prepared.
- The rocky abutments should be suitably shaped and prepared in order to get good contact between the impervious core of the embankment and the rock. Overhangs, if any, should be removed.
- Vertical surfaces should be excavated to form moderate slopes, not less than 1 in 4 to 1 in 5. A wider impervious zone and thicker transitions are also provided sometimes at the abutment contacts to increase the length of path of seepage and to protect against erosion.
- Sufficient creep length should be provided between impervious section of the dam and the abutment, so as to provide safety against piping. The creep length should be not less than 4 times the hydraulic head.
- Junction of non-overflow masonry/concrete dam with earth dam shall be established by proper compaction of contact layers.
- Proper bond should be provided between the earthwork and the outlet walls.

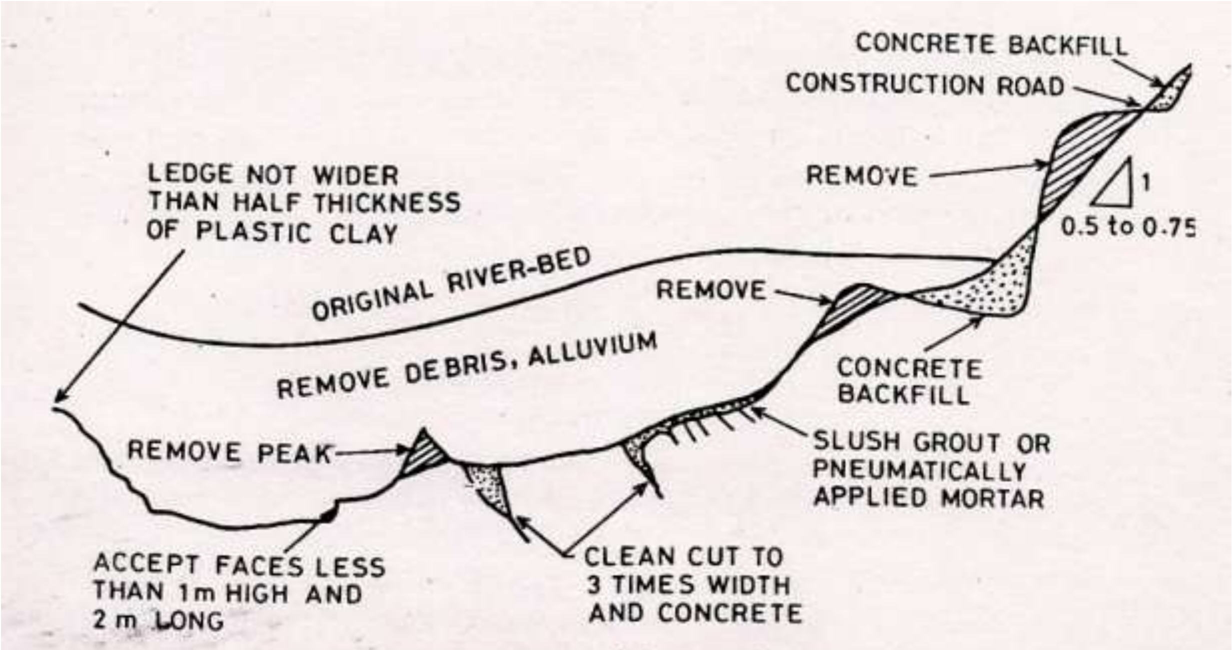


Fig. 13.8. Typical treatment of abutments in core contact zone.

28. CHOICE OF CONSTRUCTION MATERIALS FOR EARTHEN DAMS

- Because of huge quantities of material involved in construction of earth dam, the material must come from borrow areas and quarries close to the site.
- The earth dam may be designed as a homogeneous one or zoned type depending upon the qualities and quantities of the various materials available from the borrow areas and foundations.
- To economize the design, even erratic material that cannot be relied upon to have the consistent minimum properties needed for any zone can be utilized in random zones.
- It is general practice to utilize the materials available in their natural state rather than to improve the properties of the materials by blending, mixing, screening, washing, etc.
- The designer should aim at maximum utilization of the material available from compulsory excavation.
- The soils available from the borrow areas and excavation shell be identified and classified in accordance with IS: 1498 – 1970.

29. DESIGN CONDITIONS OF SLOPE STABILITY ANALYSIS

- An earth dam shall be safe and stable during all phases of construction and operation of the reservoir.
- Hence, the analysis shall be done for the most critical combination of external forces which are likely to occur in practice.

The following conditions are usually critical for the stability of an earth dam:

Case I – Construction condition with or without partial pool (for upstream and downstream slopes).
Case II – Reservoir partial pool (for upstream slope),
Case III – Sudden drawdown (for upstream slope),
Case IV – Steady seepage (for downstream slope),
Case V – Steady seepage with sustained rainfall (for down stream slope) where annual rainfall is 200 cm or more,
Case VI – Earthquake condition (for upstream and down-stream slopes)

30. METHOD OF SLOPE STABILITY ANALYSIS

The methods of analyzing the slope stability depending upon the profile of failure surface are:

- Circular arc method
- Sliding wedge method

CIRCULAR ARC METHOD

- In this method of analysis the surface of rupture is assumed as cylindrical or in the cross section by an arc of the circle.
- This method, also known as Swedish or Slip Circle method, is generally applicable for analyzing slope of earth dams and dams resting on thick deposits of fine grained materials.

SLIDING WEDGE METHOD

- The sliding wedge method of analysis is generally applicable in the circumstances where it appears that the failure surface may be best approximated by series of planes rather than a smooth continuous curve as for the method of circular arc.
- This method is predominantly not much used.
- For details refer BIS code IS: 7894-1975 code of practice for stability analysis of Earth Dam.

31. MINIMUM DESIRED VALUE OF FACTOR OF SAFETY AS PER B.I.S. FOR THE VARIOUS LOADING CONDITIONS

S.N	CASE NO.	LOADING CONDITION	F.O.S
1	Case I	Construction condition with or without partial pool (for upstream and downstream slopes)	1.0
2	Case II	Reservoir partial pool (for upstream slope)	1.3
3	Case III	Sudden drawdown (for upstream and down stream slopes)	1.3
4	Case IV	Steady seepage (for downstream slope)	1.5
5	Case V	Steady seepage with sustained rainfall (for down stream Slope) where annual rainfall is 200 cm or more	1.3
6	Case VI	Earthquake condition (for upstream and down-stream slopes)	1.0

32. ANALYSIS FOR EARTHQUAKE FORCES

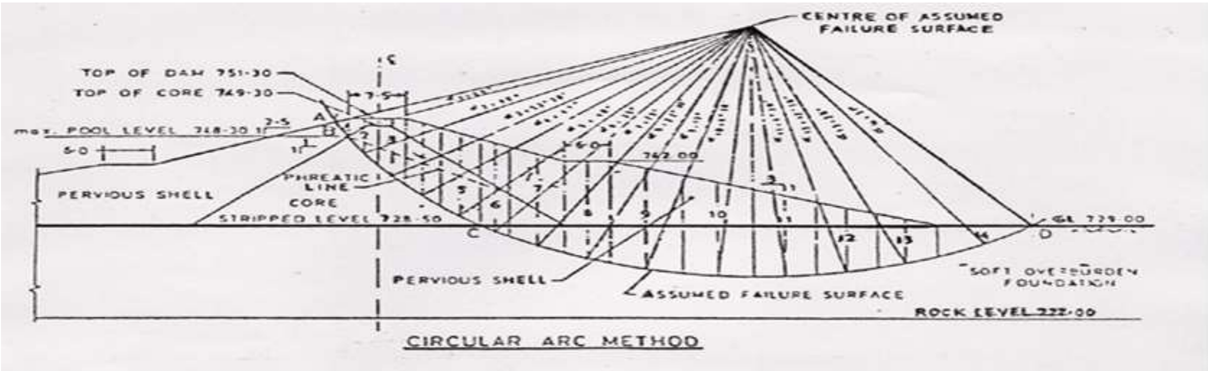
- In regions of seismic activity stability calculation of the slope of a dam should also include seismic forces because they reduce the margin of safety or may even bring about the collapse of the structure.
- General design approach for earthquake forces is given in IS: 1893-1984. Where the analysis is carried out by the circular arc or sliding wedge method, total weight of the sliding mass considered for working out horizontal seismic force shall be based on saturated unit weight of the zones below the phreatic line and moist weight above it.
- If the zone above the phreatic line is freely draining, drained weight shall be considered for that zone.
- The critical condition of analysis of upstream slope for operating condition is the sudden drawdown.
- When the reservoir is full the seepage pressure acting toward the downstream side increases the resistance of upstream slope toward the sliding and as such, such a condition is not considered most critical. However this condition combined with earthquake force is considered critical as the stability of upstream slope get reduced and may lead to the failure of dam when the reservoir is full.
- Similarly, the down stream slope of the dam shall be analyzed for a condition of steady seepage combined with earthquake forces.

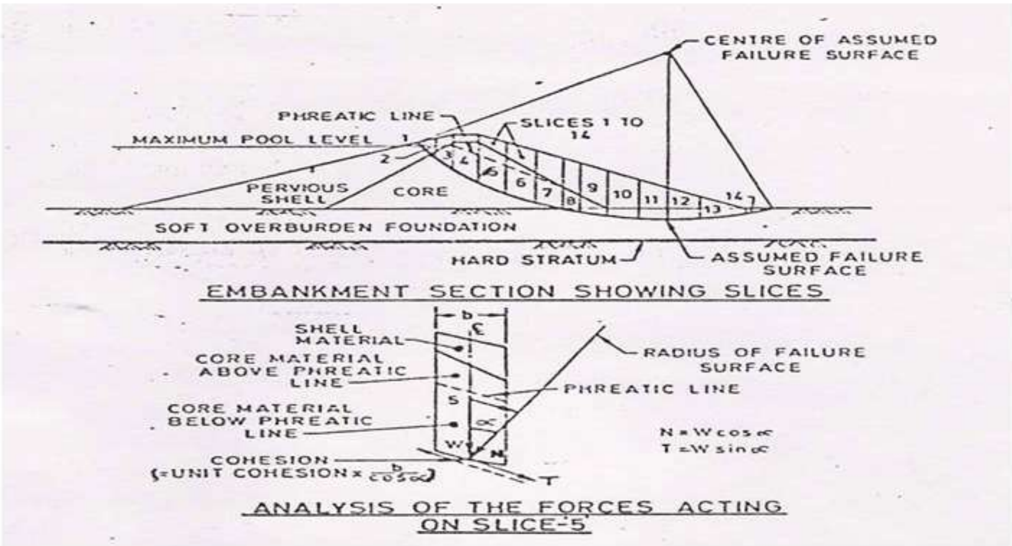
33. METHOD OF STABILITY ANALYSIS

- After deciding upon the tentative cross section of the proposed earth dam a possible circular failure surface through the dam and foundation is assumed.
- Theoretically it is necessary to try an infinite number of possible failure circles with different centers and radii before the most critical one giving lowest accepted factor of safety may be located.
- In practice, however, a limited number of slip circles, about 12 to 15, selected on the basis of past experience are considered sufficient for each condition of analysis.
- After trying about 12 to 15 failure circles, if the lowest value of factor of safety is acceptable the profile of the section needs no changes and the assumed profile shall be considered adequate from consideration of stability.
- The circle which yields the minimum value of factor of safety is the most critical.
- The circle which yields the minimum value of factor of safety is the most critical.
- However, if the value of the factor of safety obtained for the critical failure arc is more than required, the section shall be modified by reducing the berm widths and steeping the slopes.

- The above process shall be repeated till the profile of the section gives the required factor of safety. If, on the other hand, the value of the factor of safety obtained during the process of calculation for the failure surface is less than the minimum acceptable, the same shall be increased to the required value by trials after carrying out necessary changes in the profile.
- For the sake of simplicity and reducing the calculations, the various materials, namely, riprap, internal filters, rock-toe, etc. falling within the sliding mass shall be considered to have the same properties as those of the respective zones within which they are located.
- This will not materially affect the value of factor of safety as these materials usually cover only a small area as compared to the area of the zones in which they are located.
- However, if such materials cover appreciable cross- sectional area, they shall be considered separately.
- Stability analysis of the slope shall be done for sections of dam for different heights, the entire length being divided into suitable reaches.
- In deciding the reaches, variations in foundations conditions shall also be taken into account.
- The trial sliding mass is divided into a number of vertical slices. The number of slices depends upon the width and mass of the sliding mass, number of various zones included in the sliding mass and the accuracy desired.
- Usually 10 to 15 slices are desirable.
- For zoned embankment and stratified foundations with different properties, where an arc of the potential failure surface passes through more than one type of material, the vertical ordinates of the slices for each zone or part of the foundation shall be obtained by locating the slice at each such dividing point.
- The slices (for convenience) may be of equal width though it is not rigidly necessary to do so. The failure surface is shown in Figure.
- The driving forces and resisting forces are computed to calculate the factor of safety.
- The total weight W of the slice of width b is equal to the areas of the various zones included in the slice multiplied by their respective appropriate units weights (soil plus water).
- This acts vertically downwards through the center of the gravity of the slice.
- The two components of this weight W, namely the force normal to the arc of the slice, N= W cos α and the force tangential to the arc of the slice, T= W sinα are determined after resolving weight W in the radial and tangential direction, α being the angle made by the radius of failure surface with the vertical at the center of the slice.
- In the total stress method of analysis the test results include the influence of pore water pressure and hence they need not be accounted for separately in the analysis.
- However in the analysis by the effective stress method allowance for pore pressure shall be made separately. The pore water pressure U acting on the arc of the slice results in an uplift force which reduces the normal component of the weight of the slice.
- The net or effective downward force acting on the curved bottom boundary of the slice is the total weight minus the upward force due to pore water pressure.

- The effect of the pore pressure on resisting forces is accounted for by assuming buoyant weight of the material lying below the phreatic line.
- Component of shearing resistance due to internal friction is therefore (N-U) tanϕ, where ϕ is the angle of shearing resistance of the material at the base of the slice and (N-U) is the effective normal load N.
- Another force acting at the bottom of the slice and which opposes the movement of sliding mass is the shearing resistance offered by the material due to its cohesion, C and is equal to the unit cohesion, C multiplied by the length of the bottom of the slice and is approximately equal to c x b / cosα.
- In practice the length of the arc may be measured accurately as the expression b / cosα shall not give the length of arc of the slice when b is infinitely small.
- The total resisting or stabilizing force S developed at the bottom of the slice is equal to (c x b/cosα) + (N-U) tanϕ.
- The driving or the actuating force T due to the weight of the slice is equal to W sinα.
- Similar forces are worked out for all the slices considered for a potential failure surface.
- The results of these components shall be tabulated and sums of the resisting and driving forces shall be obtained.





The factor of safety against sliding for the assumed failure surface is computed by the equation:

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

Where,

FS = Factor of safety

S = Resisting or stabilizing Force.

T = Driving or actuating force.

$$C = c_1 \times \frac{b}{\cos\alpha}$$

N = Force normal to the arc or slice.

U = Pore water pressure.

ϕ = Angle of shearing resistance.

W = Weight of the slice.

α = Angle made by the radius of the failure surface with the vertical at the centre of slice.

c_1 = Unit cohesion, and

b = Width of the slice.

34. ADVANTAGES OF EMBANKMENT DAMS

Embankment dams have many advantages compared to gravity dams. Some of the main advantages are:

- Embankment dams can be constructed on any given foundation condition and the excavation for foundation need not be up to rock level, where the bed rock is deep seated.
- Soil/rock materials locally available are used with negligible processing.
- Use of costly manufactured items like cement and steel is eliminated and thus saving in cost.
- Embankment dam is more resistant to seismic forces and are preferred in areas of high seismicity.
- Embankment dam can be constructed in stages and the dam height can be increased later on easily, if needed.
- With modern earth moving machineries, the dam can be completed in less time compared to a rigid dam.
- Embankment dams are generally much cheaper.

35. Main Causes of Embankment Dam Failure

- Overtopping and external erosion during flood discharge because of inadequate spillway capacity or non-functioning flood gate
- Internal erosion along the dam-foundation interface or along embankment with adjoining or embedded appurtenants structures or concentrated piping in the embankment itself because of inadequate or non-existent filter zones.
- Non homogeneity in the foundation or dam (leading to foundation failure or erosion)
- Large settlement in the foundation
- Crack following the settlement, with resulting piping effect
- liquefaction

36. OVERTOPPING FAILURE OR EXTERNAL EROSION

Occurs when the reservoir water level exceeds the height of dam and flows over the crest.

- Overtopping failure takes place because of the external erosion of the dam and may result from
- Uncontrolled flow of water over the dam, around the dam, and adjacent to the dam plus the erosive action of water on the dam.

It may be caused by a number of factors or a combination of factors including:

- Inadequate inflow flood calculations,
- Inadequate spillway design, and / or,
- Poor spillway maintenance.

OVERTOPPING FAILURE OR EXTERNAL EROSION

37. Structural Failure

Occur in the abutments, foundation and embankment slopes e.g. failure due to overturning, sliding, build up of pore pressure.

Reasons:

- Poor foundation conditions
- Poor construction practices

- Poor fill materials
- Inadequate slopes
- Poor design of the dam and its appurtenances

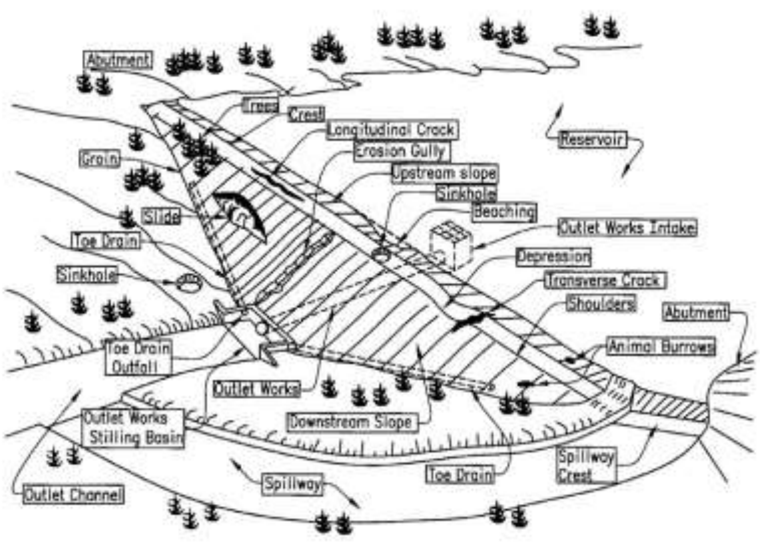
Can be observed in early stages by:

- Presence of longitudinal or transverse cracks on the crest or slopes
- Excessive settlement
- Misalignment of the crest.
- Development of a slope failure from transverse cracking
- Also occurs in appurtenant works, spillway, outlets and gates.

38. Seepage Failures

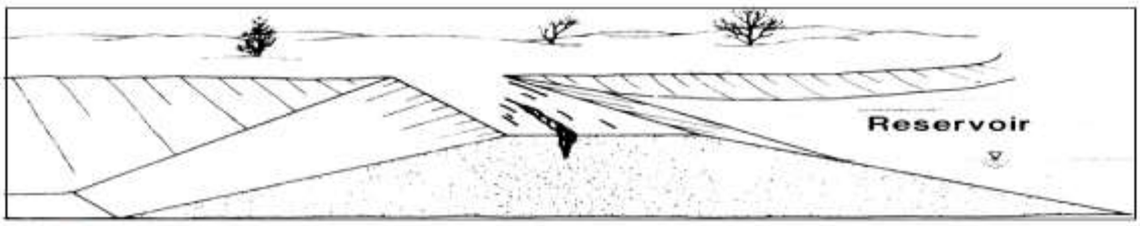
- Occurs both through the dam and under and around the dam in the foundation and abutment materials and allowed in controlled manner.
- If uncontrolled, can erode material starting at d/s slope or foundation back toward u/s slope to form a "pipe" which often leads to a complete failure. This phenomenon is known as "Piping".
- Another type of seepage problem is sloughing i.e. miniature slide on u/s or d/s slope of the embankment.
- Clear seepage is not a serious problem if adequate drains and filters are provided to prevent the transport of fill material and if the seepage water is not allowed to pond at the downstream toe.
- Seepage can emerge on the d/s slopes, below the toe of the dam or on the d/s abutments.
- The presence of seepage may be identified by a change in vegetation seepage areas on d/s slope.
- Dirty seepage is indicative of erosion of the fill material and may lead to failure by progressive erosion (piping) if remedial action is not taken.

Seepage Failure & Deficiencies in Dam

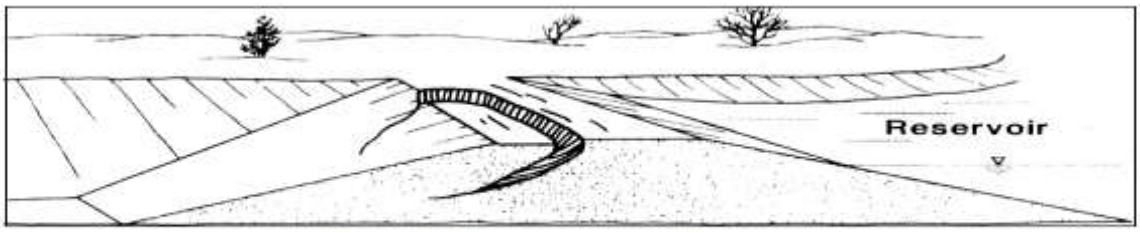


Longitudinal Crack in dam

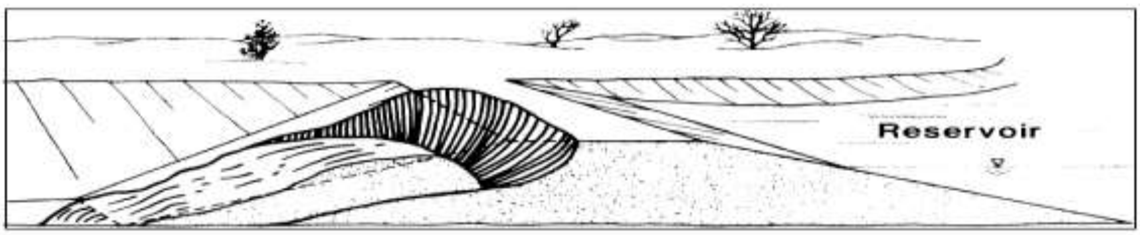
Longitudinal Cracks in Dam



A - Longitudinal cracks form and runoff water enters

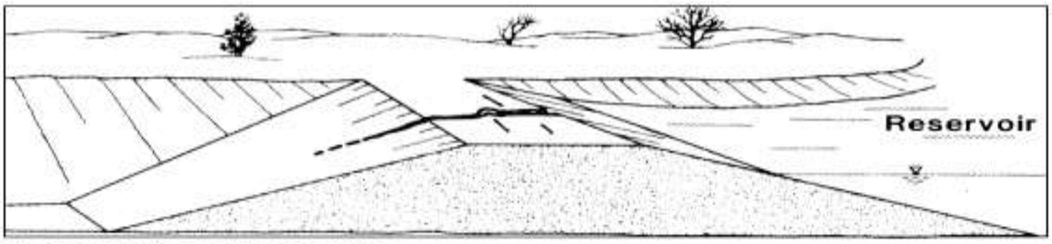


B - Cracks widen and the ground settles on one side of the crack

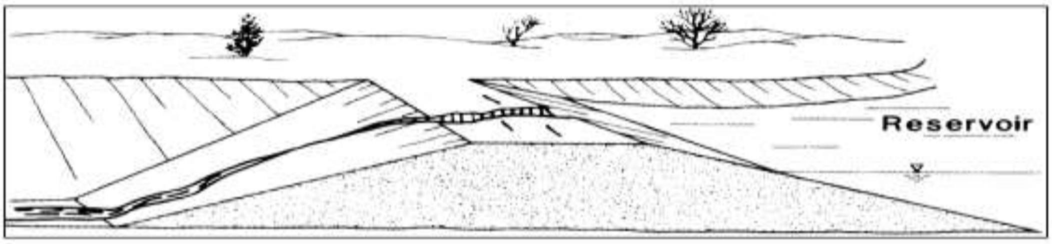


C - The slope fails

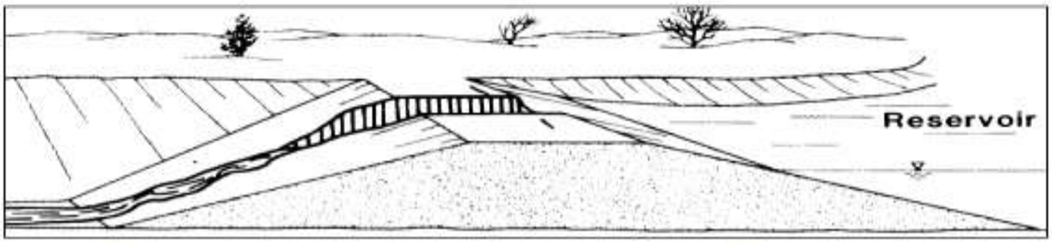
Transverse cracks in Dams



A - Initial Transverse Cracking
Often caused by settlement, foundation problems or placement of fill over steep abutments.



B - Progression of Transverse Cracking to a point below the waterline
Water from the reservoir begins to flow through the crack.



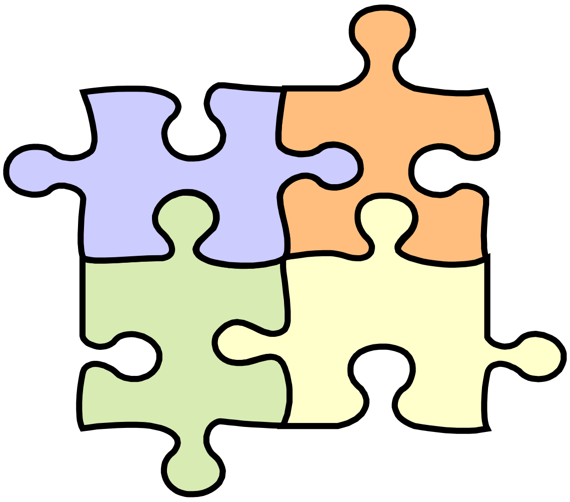
C - Transverse Cracking progressed to an overtopping situation
Condition has progressed to a point of imminent failure.

39. Statistics of Failures

a. OVERTOPPING ~ 35% of all failures
b. FOUNDATION DEFECTS ~ 30% of all failures
c. PIPING AND SEEPAGE ~ 20% of all failures
d. CONDUITS AND VALVES ~ 10% of all failures
e. OTHER ~ 5% of all failures

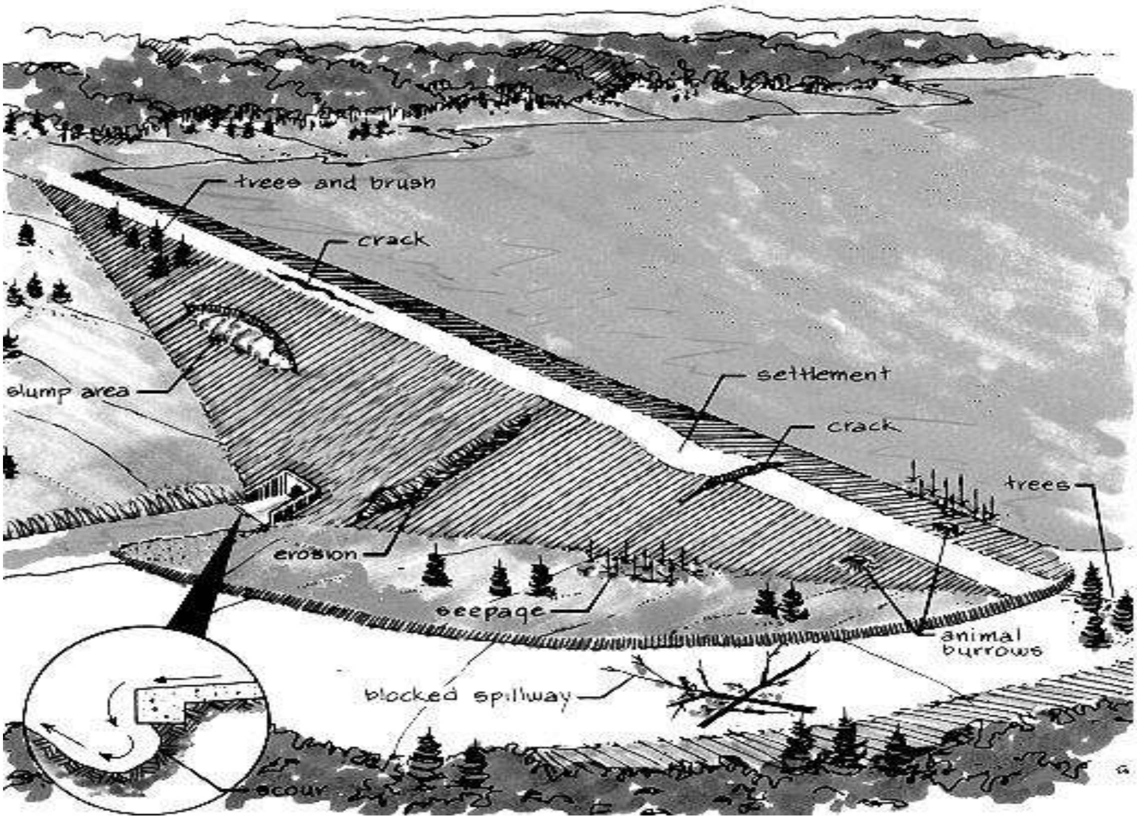
40. Failure Prevention

- Failure could have been prevented if some of these points had been observed
- Failure is a complex process and generally happens because of combinations of various reasons.
  - Begins with some abnormality in behavior.
  - Consequent deteriorations.
  - Further damage or disaster



Inspection, Monitoring & Maintenance of dams as well as data analysis and interpretation has a critical role in the field of dam safety

41. Potential Problem Indicator And Maintenance of Dam



42. TEHRI ROCKFILL DAM

- 260.5m high earth and rock fill dam.
- Consists of a moderately inclined central core flanked on both sides by filters and shell material.
- U/S slopes of the dam are 2.5: 1 and D/S 2: 1.
- The Plasticity index of the clay available for use in the core is of the order of 10 (+/-) which is on the lower side.
- The available clay was blended with gravelly sand to increase its resistance against cracking.

LIQUEFACTION

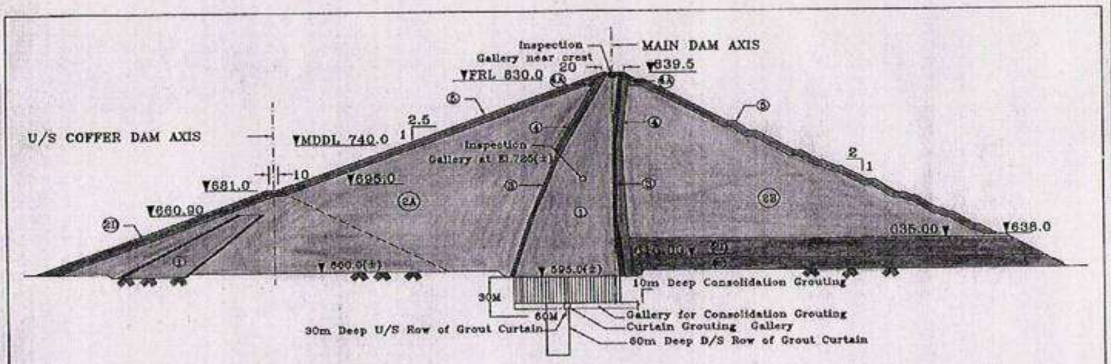
- The material for clay core and the shell were extensively tested for possibility of liquefaction under shaking and were found to be safe.
- However, the shell is being compacted to concrete like density of 2.36 to achieve maximum seismic resistance against liquefaction.
- Such high density could be achieved because of gradation of fill material available at Tehri, which was a mixture of sand, gravels and cobbles.

FILTERS

- Two tiers of filters (5m thick each), are provided both on u/s and d/s sides.
- The d/s filter can prevent failure of the dam even if there are concentrated leaks after a seismic event, by preventing migration of particles and can help in eventually plugging the leaks.
- The u/s filter has been designed so that in the event of cracking of core it would get washed into cracks and seal them by choking the water passages.

INSPECTION GALLERIES

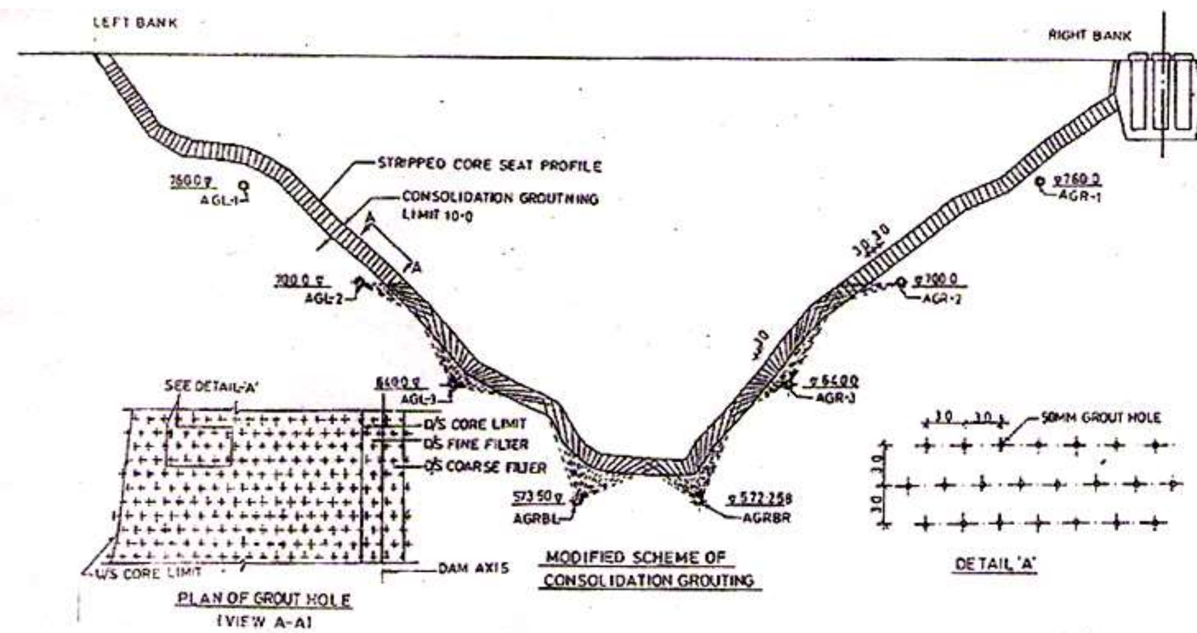
- An Inspection galley about 100 m below the FRL is provided for the first time in India.
- The gallery has been designed with special deformation joints so that it can easily undergo the settlement pattern of the clay over which it is seated.
- Through this, it is possible to observe the behavior of dam through its settlements, gaps in the joints etc., at various stages of construction, operation and seismic events.
- One more Inspection Gallery on the top of the core has also been proposed to observe the cracks in the core, if any due to differential settlements.

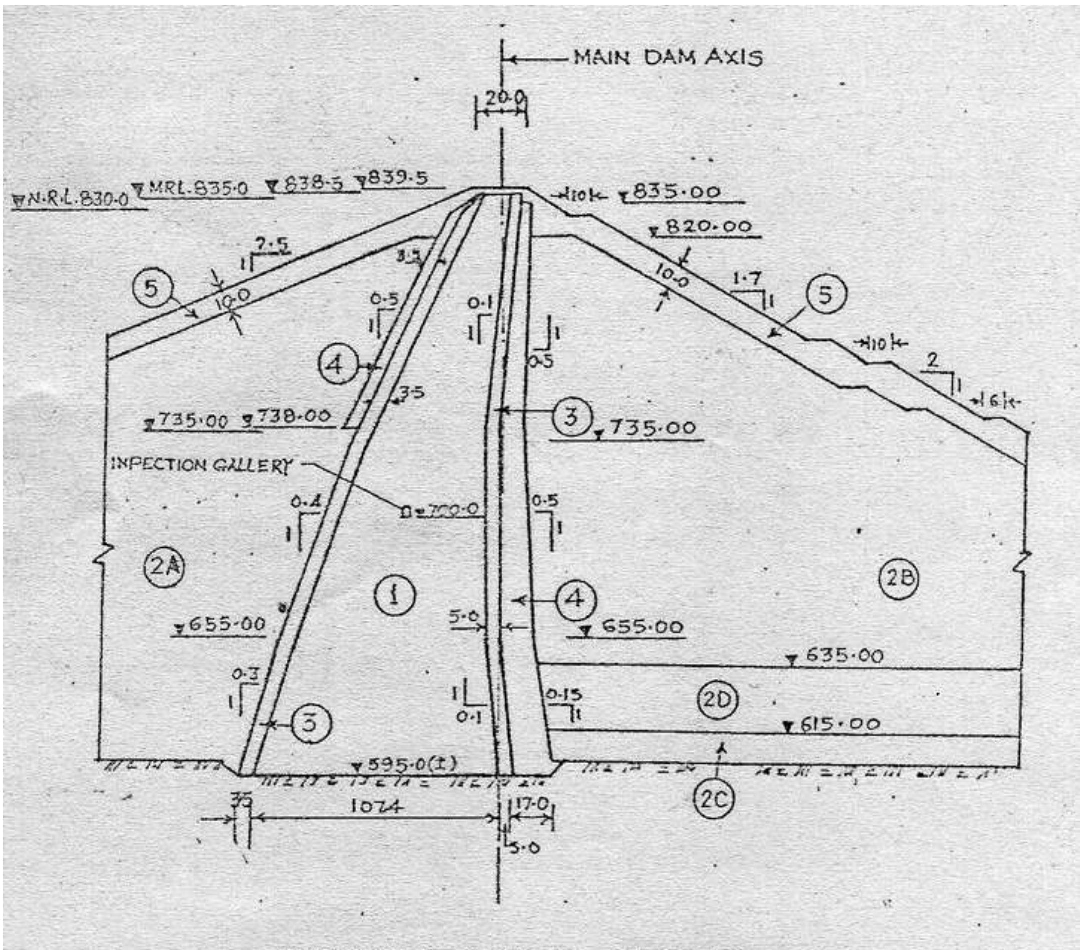


TEHRI DAM – TYPICAL SECTION

DAM ZONES

1. Well graded impervious blended core material with maximum particle size up to 200 mm.
2A. Well graded terrace gravelly material maximum size 600 mm, fines (< 4.75 mm) less than 35%, silt content(< 0.075 mm) not more than 5% provided further that 80% of the material should not have aleurite content exceeding 30%.
2B. Well graded terrace gravelly material maximum size 600 mm, silt content (<0.075 mm) not more than 5%.
- Well graded terrace gravelly material maximum size 600 mm, fines (<4.75 mm) between 10 to 22%.
- Well graded terrace gravelly material maximum size 600 mm, fines (<4.75 mm) between 10 to 18%.
3. First layer of transition zone (Fine filter), maximum particle size < 20 mm.
4. Second layer of transition zone (Coase filter), sand and gravel mixture, maximum size < 60mm.
4A. Third layer of transition zone, hard blasted rock with max. size upto 600mm
5. Well graded hard blasted rock with maximum size up to 1200 mm.





TYPICAL CROSS-SECTION

(2A) Well graded terrace gravelly material, maximum size 600 mm. fines (≤ 4.75 mm) less than 35%, silt content (≤ 0.075mm.) not more than 5%, provided further that 80% of the material should not have aleurite content exceeding 30%.

(2B) Well graded terrace gravelly material, maximum size 600mm. fines (≤475mm.) less than 35%, silt content (≤0.075 mm) not more than 5%.

(3) Fine filter, maximum size upto 20 mm, silt content less than 3%.

(4) Coarse filter, sand and gravel mixture, maximum size ≤60 mm, silt content less than 3%.

(5) Well graded hard blasted rock with maximum size upto 1200 mm

(2C) Well graded terrace gravelly material, maximum size 600mm., fines (≤475mm) between 10 –22% and silt content not more than 3%.

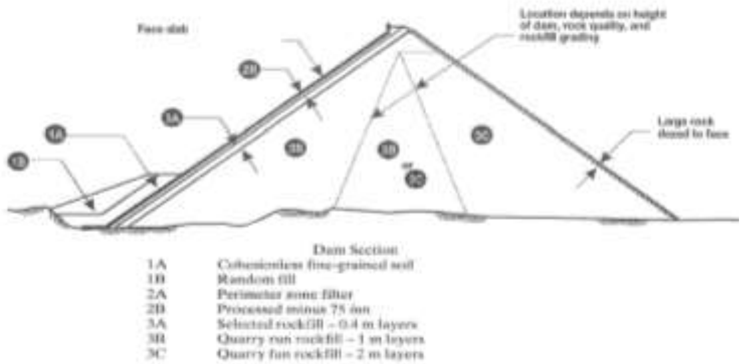
(2D) Well graded terrace gravelly material, maximum size 600mm, fines (≤4.75mm.) between 10 - 18% and silt content not more than 3%.

43. Rock-fill Dams with u/s Face Membranes

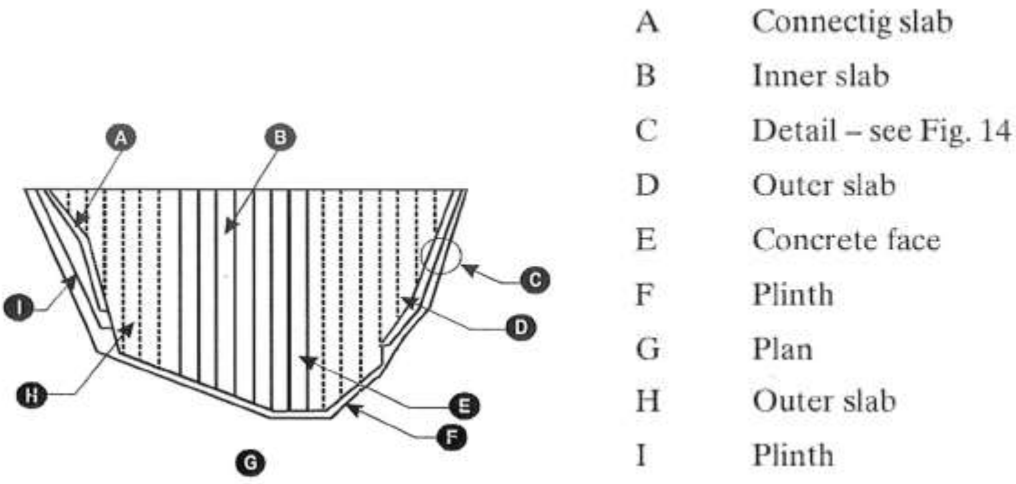
Type of facings or membranes

- Cement Concrete Membranes
- Asphaltic Concrete Membrane
- Steel or Timber Membranes

44. Dam with Cement Concrete membranes



45. Dam with Cement Concrete membranes



46. Advantages of Dam with U/S Membranes

- Greater Stability
- Greater tolerance for leakge
- Accessibility of membrane for inspection and repairs
- Speed of construction
- Stage construction facility

Design of Core and Filter in Earth and Rockfill Dams

1.0 DAMS AND THEIR DESIGN PHILOSOPHY

1.1 Role Played By Dams & Reservoirs

Dams have been built across rivers by mankind right from the dawn of civilization for storing the river flow during rainy season and releasing it during the remaining part of year for either domestic use or for irrigation. Flood control has been another important function of these dams. While releasing water from the storages, hydroelectric energy is also generated. With the growth of population all these functions of dams and storages have assumed great significance and hence every civilization has tried to keep pace with the needs of the society for food, energy, fibre and well being through this activity of water resources development.

1.2 Inputs For Safe Design

Dams constitute perhaps the largest and the most complex of structures being built by civil engineers. Basic input of water is dependent on nature, so also the river course, its history, its underlying strata and its stability. Assessment of the variability of these natural phenomenon and providing for it in the design of a dam, has been an important challenge for the dam builders. The dams are built to last from 100 to 300 years depending upon merits of each case. During their service life, they are designed to withstand all the possible destabilizing forces with a certain factor of safety which has been an indicator of a factor of ignorance or lack of knowledge of various response processes of materials used in construction, the stresses caused, the stains experienced and finally the failure mechanism.

1.3 Design Constants

The destabilizing forces themselves are associated with a significant natural variability. Assessment of the range of these forces likely to affect a dam stability during its lifetime and then ascribing a design value for such forces has been and will continue to be a matter of study and concern for the designers. Every design or construction engineer cannot study these processes for every dam and hence standards or codes of design and construction practice are laid down and updated as information and knowledge grows Assistance of scientists working in fields such as Hydrometeorology, Geology, Geophysics, Geomorphology, Seismology in assessing the likely parameters of these forces is taken, the information collected is processed as per standards and design constants worked out.

Large dams store very large volumes of water. Design of such dams, therefore, has to be extra safe so that there is a minimum probability of their failure and consequent rapid or sudden release of storage which can cause disproportionate flooding and losses to the human habitats in the downstream. Very stringent codes are laid down for this purpose. In case of inflow into a reservoir, for instance, a conceptual Probable Maximum Flood (PMF) is determined by following special analytical procedures. If the reservoir and the spillway caters to a properly determined outflow on the basis of such inflow, the dam would be hydrologically safe. In similar manner, geotechnical properties of foundation material or construction material can be determined and design constants worked out so that structural design based on them yields a safe structural construction. Statistically speaking, the design constants should cover the probability of occurrence of forces expected during the lifetime of the structure under design.

1.4 Design Philosophy

The codes of practice invariably lag the strata or knowledge or state of Research & Development (R&D). In fact codification follows verification of generated knowledge and its global acceptance. Codes, therefore, tend to remain conservative and normally incorporate a higher factor of safety and hence perhaps yield structures with larger dimension and/or with higher costs. There is yet another aspect of design philosophy which is not very explicitly understood nor adequately explained. It pertain to the various stages of design for complex structures like dams viz. conceptualization, pre-feasibility, feasibility, detailed project report (DPR), pre-construction, early construction and advanced construction stages.

1.5 Refine The Design As You Build

A designer starts with broad concept of design parameters in the beginning and goes on refining his data base and hence the designs, as he proceeds through the various stages. He assumes for the sake of his inadequate data base, simplifications or generalizations which obviously incorporates a large factor safety in initial stages. As the passes through successive stages, his data base proves, better and more accurate data base emerges; the range of design constants narrows down and factor of safety reduces.

Generally, the outer dimensions of a structure do not necessarily get modified; but components, zones or internal arrangements of a structure do undergo modification. The structure's response to the destabilizing forces is worked out with greater detail and is refined while moving from one stage to the next stage. Engineers call this a process which is loosely described as 'Design as you build' or 'Refine the design as you build' mode. It certainly does not mean inadequacy of design or does not reflect on ignorance or incompetence of project or design engineers. However, an inadequate understanding of this very philosophy is one major factor responsible for much public criticism of many of our water resources projects.

2.0 Defensive Measures

International practice recommends deployment of various defensive measures to provide extra safety in design of high risk rockfill dams.

- Allow ample freeboard to allow for settlement, slumping or faul movements.

- Use wide transition zones of material not vulnerable to cracking.

- Use chimney near the central portion of embankment.

- Provide ample drainage zones to allow for possible flow of water through cracks.

- Use wide core zones of plastic materials not vulnerable to cracking.

- Use a well-graded filter zone upstream of the core to serve as a crack-stopper.

- Provide crest details which will prevent erosion in the event of overtopping.

- Flare the embankment core at abutment contacts.

- Locate the core to minimize the degree of saturation of materials.

- Stabilize slopes around the reservoir rim to prevent slides into the reservoir.

- Provide special details if danger of fault movement in foundation exists.

This list should not by any means be considered as all-inclusive. However, defensive measures, specially the use of wide filters and transition zones, provide a major contribution to earthquake-resistant design and should be the first consideration by the prudent engineer in arriving at a solution to problems posed by the possibility of earthquake effects.

3.0 Criteria for Safe Design of Earth/Rock fill Dam

(i) There should lie no possibility of dam being overtopped by flood water.

(ii) The seepage line should be well within the downstream face.

(iii) The u/s and d/s slopes should be stable under worst condition.

(iv) The foundation shear stresses should be under safe limits.

(v) There should be no opportunity of free flow of water from u/s to d/s face.

(vi) The dam and foundation should be safe against piping.

(vii) The U/s face should be properly protected against wave action and the d/s face against the action of rain

4.0 DESIGN OF CORE FOR ROCK FILL DAMS

4.1 Core

4.2 The core provides impermeable barrier within the body of the dam. Impervious soils are generally suitable for core. However, soils having high compressibility and liquid limit are not suitable as they are prone to swelling and formation of cracks. Soils having organic content are also not suitable. IS:1498-1970 may be referred for suitability of soils for core. Appendix A gives recommendations based on IS:1498-1970. Recommendations regarding suitability of soils for construction of core for earth dams in earthquake zones are given in Appendix B.

4.3 Core may be located either centrally or inclined upstream. The location will depend mainly on the availability of materials, topography of site, foundation conditions, diversions considerations, etc. The main advantage of a central core is that it provides higher pressures at the contact between the core and the foundation educing the possibility of leakage and piping. On the other hand inclined core reduced the pore pressures in the downstream part of the dam and thereby increases its safety. It also permits construction of downstream casing ahead of the core. The section with inclined core allows the use of relatively large volume of random material on the downstream.

4.4. The following practical considerations govern the thickness of the core:

a) Availability of suitable impervious material;
b) Resistance to piping;
c) Permissible seepage through the dam; and
d) Availability of other materials for casing, filter, etc.

However, the minimum top width of the core should be 3.0 m.

4.5 The top level of the core should be fixed at least 1 metre above the maximum water level to prevent seepage by capillary siphoning.

5.0 Casing

5.1 The function of casing is to impart stability and protect the core. The relatively pervious materials, which are not subject to cracking on direct exposure to atmosphere are suitable for casing. IS:1498-1970 may be referred for suitability of soils for casing. Appendix A gives recommendations based on IS:1498-1970.

6.0 Special Design Requirements

6.1 **I**n addition to basic design requirements given at 5, the following special design requirements, should also be satisfied for both earth and rock fill dams:

a) Control of cracking.
b) Stability in earthquake regions, and
c) Stability at junctions.

6.2 Control of Cracking - Cracking of impervious zone results into a failure of an earth dam by erosion, breaching, etc. Due consideration to cracking phenomenon shall, therefore, be given in the design of earth dam.

6.3 Reasons of Cracking - Cracking in the core of earth or rockfill dam occurs due to foundation settlement and/or differential movements within the embankment. Differential movements in the embankment take place due to the following reasons:

a) Unsuitable and/or poorly compacted fill materials,
b) Different compressibility and stress-strain characteristics of the various fill materials, and
c) Variation in thickness of fill over irregularly shaped or steeply inclined abutments.

6.4 .Cracking also develops by tensile strains caused by various loads, such as dead load of the structure, filling of the reservoir and seismic forces. Hydraulic fracturing of the core may also occur when the hydrostatic pressure at a section in the core exceeds the total minor principal stress at that section.

6.5 Types of Cracks - Cracks may be classified based on the following factors:

a) Mechanism by which cracks are developed, such as tensile, compressive, shrinkage or shearing.
b) Types of surface with which the cracking is associated, such as flat or steep.
c) Physical process involved, such as moisture or temperature changes, loading or unloading action and dynamic activity, such as blasting or earthquakes.

6.6 Tensile stresses produce cracks on flat surface by capillary action in the moisture range just below saturation. Tensile stress steep slope category cracks are associated with slumping in poorly consolidated materials.

6.7 Shrinkage cracks are produced by wetting and drying action in the moisture range of plasticity index.

6.8 Compression cracks on flat surface are produced by an abrupt change in moisture followed by substantial consolidation and cracking around the periphery of the affected area.

6.9 Cracking associated with shearing is commonly associated with steep slopes. There are two conditions in this category. One is differential settlement which involves a limited range of motion and the other is a slide failure which may involve any amount of motion. The differential settlement condition commonly involves a structure extending over two or more kinds of foundation with differing compressive characteristics or a differential loading condition on a single kind of foundation material.

6.10 Slide failures may be associated with loading ,unloading or moisture change, the distinguishing characteristics is the potential for continued movement.

6.11 Importance of Cracks - Relative importance of each type of crack category or group is given at 3.1.3.1 to 3.1.3.3.

6.11.1 Where permeability and possible erosion are of primary concern, the tension group is potentially the most serous. In this group, the cracks are open and although usually only superficial, those associated with steep slopes may extend to depths comparable to the size of structure involved. Though the development of this type of cracking is from the surface, it may persist, although deeply buried, where eventually it may contribute to unsatisfactory seepage action.

6.11.2 Where maintenance of position is a prime structural requirement the compression type of cracking is the most important because it is probable that when this type of cracking appears the settlement has already completed.

6.11.3 Shearing cracks are identified primarily by displacement between the two sides and a tearing configuration. Unlike tension or compression cracking, shearing cracks commonly occur early in the failure action and further movement can be expected after the first cracking shows up.

6.12 Measures for Control of Cracking - Following measures are recommended for control of cracking:

a) Use of plastic clay core and rolling the core material at slightly more than optimum moisture content. In case of less plastic clay, 2 to 5 percent bentonite of 200 to 300 liquid limit may be mixed to increase the plasticity.
b) Use of wider core to reduce the possibility of transverse or horizontal cracks extending through it.
c) Careful selection of fill materials to reduce the differential movement. To restrict the rockfill in lightly loaded outer casings and to use well graded materials in the inner casings on either side of the core.
d) Wide transition zones of properly graded filters of adequate width for handling drainage, if cracks develop.
e) Special treatment, such as preloading, pre-saturation, removal of weak material etc., to the foundation and abutment, if warranted.
f) Delaying placement of core material in the crack region till most of the settlement takes place.
g) Arching the dam horizontally between steep abutments.
h) Flattening the downstream slope or increase slope stability in the event of saturation from crack leakage.
i) Cutting back of steep abutment slopes.

7.0 Foundation Treatment Below Core :

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

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

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

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

8.0 Suitability of Soils for Construction of Earth Dam

Relative Suitability	Zoned Earth Dam	
	Impervious Core	Pervious Casting
Very suitable	GC	SW, GW
Suitable	CL, CI	GM
Fairly suitable	GM, GC, SM, SC, CH	SP, GP
Poor	ML. MI, MH	-
Not suitable	OL, OI, OH Pt.	-

9.0 Suitability of Soils for Construction of Core of Earth Dam in Earthquake Zones

S.No.	**Relative Suitability**	**Type of Soil**
1.	Very good	Very well graded coarse mixtures of sand, gravel and fines., D_{85} coarser than 50mm, D_{50} coarser than 6 mm. If fines are cohesionless, not more than 20 percent finer than 75 micron IS Sieve.
2.	Good	a) Well graded mixture of sand, gravel and clayey fines, D_{85} coarser than 25 mm Fines consisting of inorganic clay (CL with plasticity index greater than 12). b) Highly plastic tough clay (CH with plasticity index greater than 20).
3.	Fair	a) Fairly well graded, gravelly, medium to coarse sand with cohesionless fines, D_{85} coarser than 19 mm, D_{50} between 0.5 mm and 3.0 mm. Not more than 25 percent finer than 75 micron IS sieve. b) Clay of medium plasticity (CL with plasticity

		index greater than 12).
4.	Poor	a) Clay of low plasticity (CL and CL-ML) with little coarse fraction. Plasticity index between 5 and 8. Liquid limit greater than 25. Liquid limit greater than 25. b) Silts of medium to high plasticity (ML or MH) with little coarse fraction. Plasticity index greater than 10. c) Medium sand with cohesion less fines.
5.	Very poor	a) Fine, uniform, cohesion less silty sand, D_{85} finer than 0.3 mm. b) Silt from medium plasticity to cohesionless (ML) Plasticity index less than 10.

10.0 Location of Core in Dam Section and Type of Core

The core can be located in one of the following three positions: (1) central, (2) moderately slanting or (3) slanting. The central location need not be exactly symmetrical: cores with a steeper downstream slope and flatter upstream slope, or even with a slight slant in the upstream direction would still have characteristics of central cores. When the downstream face of the core has an upstream slant of 0.5 H : 1 V or more, the core may be considered as moderately slanting. A truly slanting core would be such that the downstream zone has a self-supporting slope, i.e., 1.25 H:1 V or more, the core may be considered as moderately slanting. A truly slanting core would be such that the downstream zone has a self-supporting slope, i.e. 1.25 H:1 V or so; such a core is almost always associated with a rockfill dam in which the main mass of rockfill downstream of the core can be placed independently by dumping or in thick layers and the placement of filter zones, core and upstream pervious zone taken up later. Even with a moderately slanting core, if the downstream rockfill zone is substantial, it is possible to carry out a portion of the work ahead of core placement.

The relative advantages and disadvantages of vertical and sloping cores are discussed below:

10.1 Slanting Core

Advantages

j) Downstream rockfill can be placed in advance and laying of filter, core and upstream zone can be taken up later. This ensures rapid progress as placement of bulk rockfill in the downstream portion is accelerated, especially in conditions wherein core placement is possible only during part of the year.

ii) Foundation grouting of the core can be carried out while the downstream shell is being placed and thus better progress achieved.

iii) Since a very small part of the slip surface intersects the slanting core, the section is practically free from the steady seepage pore pressures and is thus more stable under a steady-state condition. This results in a steeper slope of the downstream shell and corresponding economy.

iv) Since the flow lines are essentially vertical and equipotential lines are almost horizontal under sudden drawdown, the drawdown pore pressures are very much reduced. However, a larger part of the slip surface for the upstream slope passes through the core material than would be the case with a central core.
v) In the case of cracking of the core, the inclined core will leave a large mass of stable rockfill on the downstream side and is likely to be safer.
vi) Filter layers can be made thinner and placed more conveniently.

Disadvantages

i) The depth of excavation of the foundation at the contact surface of the core is determined by the nature of the formations and cannot be predetermined in advance. Thus advance treatment of the contact area may present a problem in the case of a slanting core because if the depth of excavation increases, the contact area moves upstream.
ii) By slanting the core upstream, although the downstream slope can be made steeper, nevertheless, the upstream slope will generally become flatter as the shear strength of the core material will be less than that of the pervious shell material; the advantage of reduced drawdown pore pressures may not compensate this factor. Thus any economy in total quantity of materials by adjustment of core position would depend on the relative strength of the two materials.

10.2 Central Core

Advantages

i) Provides higher pressure on the contact surface between the core and the foundation, thus reducing the possibility of hydraulic fracturing.
ii) For a given quantity of soil, the central core provides slightly greater thickness.
iii) Provides better facility for grouting of foundation or contact zone or any cracks in the core if required afterwards, as this can be done through vertical rather than inclined holes.
iv) Foundation area is independent of depth of foundation and hence can be marked and treated in advance.

Disadvantages

i) The advantages listed for a slanting are not obtainable. Also, a moderately thick central core with pervious shells will result in a slightly flatter downstream slope of the dam.
ii) The problem of differential settlement between the core and the shell zone may result in cracking parallel to the dam axis.

11.0 Design of Filters for Earth/Rockfill Dams

11.1 Introduction

Water conservation and development of water resources for irrigation have attracted human ingenuity since time immemorial. A number of ancient tanks and earthen embankments stand testimony to the skill of our ancestors. The Grand Anicut across Cauvery River, built more than 1600 years ago and providing irrigation to 0.4 million hectares of land, is a typical example of the ancient earth dams in the country, still in service today. In the past, design of earthen dams was mainly carried by the rule of thumb and judgment of the designer, and the heights adopted were moderate. Advances in the field of soils mechanics and construction equipment over the years have made it possible to design and construct earth/rockfill dams to heights which would have been considered impossible in the past The Beas Dam (115m) Ram Ganga Dam (125m) and Salal Dam (128m) are examples of such high dams successfully completed. Considering these achievements within the country and the satisfactory performance of embankment dams of greater heights elsewhere, a number of high embankment dams in the Himalayan region have been taken up and are in different stages of development. The Tehri Dam (260m) now under completion on Bhagirathi river in Uttaranchal is the first of the kind, and several other dams of similar height are in the pipeline. The future mega dams like Tipaimukh, Subansiri, Dehang, Bursar, Kishau, Kotli Behl and Utyasee within the country, as well as the Karnali, Pancheswar, Wangchu and Kurichu dams across the border will need many engineering skills.

Design and construction practice for embankment dams have undergone a number of changes over the years. One of the important features that could be noticed is recognition of the useful role of 'protective filters'. Analysis of the performance of embankment dams in the world showed that there are almost no cases of damage or failure by piping, when filters had been provided as per accepted design practices, and most of the failures had occurred in dams without chimney filter or which had excessively coarse filters. Well planned filter drainage has become obligatory in the design of modern dams. Filters are provided to safely carry the seepage water which may pass through the body of dam, through the foundations, or along their contact, thus protecting the structure against the undesirable and harmful effects of seepage. Generally seepage is expected to occur through the pores of the base soil. But there could be a more severe condition of water leaking through cracks which may develop in the dam body foundation system. Enough evidence already exists from the observed behaviour of dams, supported by theoretical calculations, that such concentrated leaks can develop due to various reasons. Fortunately, recent studies have shown that the filters, if adequately designed can also be effective in controlling erosion through such concentrated leaks. The embankment dam designer should therefore pay adequate attention in arriving at a proper design of these filters.

The filter criteria contained in the IS code is based on the criteria recommended by Terzaghi and studies carried out with non-cohesive soils. There is scope to improve the provisions in the code to cater all soil types. Recent studies, which included controlled laboratory tests performed by various agencies and individuals ha e brought out some new findings on the evolution of criteria for conservative/critical filters, capable of preventing erosion and sealing off concentrated leaks. Particulars of this modified criteria and details of its adoption in rehabilitating a dam are briefly described. Some other situations where protective filters could be advantageously used, are also discussed.

12.0 Conservative Filter Criteria

As per the Indian Standard Code (IS: 9429-1980) a properly designed filter should satisfy the following requirements:

a) It should be much more pervious than the protected base material.
b) It should be of such gradation that particles of the base material do not migrate through or clog the filter voids.
c) It should be sufficiently thick to provide a good distribution of all particle sizes throughout the filter.

To satisfy the above requirements, the following filter criteria is recommended:

i) $\frac{D15F}{D15B} > 4$ and < 20

ii) $\frac{D15F}{D85B} < 5$

iii) $\frac{D50F}{D50B} < 25$

iv) The gradation curve of the filter material should be nearly parallel to the gradation curve of the base material.

13.0 Other criteria for design of filter are as follows:

(1) Terzaghi Criteria :

$$\frac{(D15)\ \text{filter}}{(D85)\ \text{protected soil}} \leq 4 \text{ to } 5 \text{ (Piping criteria)}$$

$$\frac{(D15)\ \text{filter}}{(D15)\ \text{protected soil}} \geq 4\text{to } 5 \text{ (Permeability criteria)}$$

(2) USBR Criteria :

(i) For graded
(ii) filters of angular particles :

$$R\ 50 = \frac{(D50)\ \text{filter}}{(D\ 50)\ \text{base material}} = 9 \text{ to } 30$$

(D15) filter

$$R_{15} = \frac{\phantom{(D_{15})\ filter}}{(D_{15})\ \text{base material}} = 6 \text{ to } 18$$

(iii) For graded filters of subrounded particles :

$$R_{50} = \frac{(D_{50})\ \text{filter}}{(D_{50})\ \text{base material}} = 12 \text{ to } 58$$

$$R_{15} = \frac{(D_{15})\ \text{filter}}{(D_{15})\ \text{base material}} = 12 \text{ to } 40$$

(iv) For uniform grain size filters :

$$R_{50} = \frac{(D_{50})\ \text{filter}}{(D_{50})\ \text{base material}} = 5 \text{ to } 10$$

The above criteria takes into account only the grain size of base material, and is based on studies made with non-cohesive soils.

Even though the filters are provided generally to take care of the seepage through the pores of the embankment soils, they should also be capable of preventing erosion of soils through concentrated leaks that may occur in the dam body or at the foundation contact.

Certain improvements and modifications to the above criteria have been brought recently on the basis of controlled laboratory tests performed by various organizations and individuals. Contributions by the US Department of Agriculture, Soil Conservation Service is worth making a special mention. Filter tests have been conducted using compacted impervious soil specimens with an artificial slot or hole and subjected o water flow discharging into the filters of varying coarseness. These studies confirmed that a conservative filter would be remarkably effective in preventing erosion and sealing off concentrated leaks, even with relatively high water pressures, velocities and gradients. Such filters are required on the downstream face of impervious core of a zoned embankment dam, and in the chimney filter of a homogeneous dam section. Because of the important role of these filters they are also known as 'critical filters'. Some of the useful conclusions drawn from the studies are :

i) The gradation curve of a filter need not have to be parallel or similar in shape to the gradation curve of the base material.
ii) A filter should be uniformly graded to provide permeability and prevent segregation. Particles finer than 0.075 mm in the filter should not exceed 5 per cent to ensure adequate permeability. The permeability of a filter should be at least 25 times that of the base material (D15F should be more than 5xD15B).
iii) Coarse broadly graded soils need finer filters than believed to be necessary. The filter should be designed to protect the fine matrix of the base material

rather than the total range of particle sizes. Filters designed based on minus 4.75mm are found to be satisfactory.

iv) Sands and gravelly sands with average D15 size of 0.5 mm or smaller are conservative filters for most of the fine-grained clays (including dispersive clays) in nature with D85 size of 0.03 mm or larger.

v) Sand filters with average D15 size of 0.1 mm or smaller are conservative for the finest dispersive clays.

Based on the above findings, the US Interior Bureau of Reclamation (USBR) has developed a new set of filter criteria (2). The filter gradation limits are determined through steps A to B as described below:

A. Select the gradation curve of the base soil that requires the smallest D15F size.

B. Proceed to step D if the base soil does not contain gravel (4.75 mm and above).

C. Prepare adjusted gradation curves for soils with particles larger than 4.75 mm. Use the adjusted curve in working step D.

D. Determine the category of the soil from Table-1.

E. Determine the maximum D15F size in accordance with Table-2.

F. To ensure sufficient permeability set the minimum D15F size greater than or equal to 5xD15B, but not less than 0.1 mm.

G. Set the maximum particle size at 75 mm and the maximum passing 0.074 mm must have a plasticity index of zero.

H. Design the filter limits within the maximum and minimum values determined in steps E, F and G. Plot the limit values and connect all the maximum and minimum points by straight lines.

Typical filter gradation limits arrived for a category 2 type of base soil is shown below:

TABLE - 1

Category	Percentage finer than 0.074 mm
1.	$>$ 85
2.	40 - 85
3.	15 - 39
4.	$<$ 15

TABLE - 2

Category of Soil	Filter Criteria	Remarks
1.	D15F $<$ 9 x D85B	Minimum 0.2 mm
2.	D15F $<$ 0.7 mm	
3.	D15F $<$ 0.7 mm $+ \frac{(40-A)(4xD85B-0.7)}{25}$	A = percentage passing 0.074mm. Minimum value of 4xD85B is 0.7.
4.	D15F $<$ 4xD85B	D85B obtained from unadjusted gradation curve.

Foundation Treatment of Dams

1. Foundation Treatment ?

To Foundation treatment is the controlled alteration of the state, nature or mass behaviour of ground materials in order to achieve an intended satisfactory response to existing or projected environmental and engineering actions**.**

Mitchell, J M and Jardine, F M (2002) 'A Guide to ground treatment' CIRIA

2. Why Foundation Treatment ?

- To increase bearing capacity and stability (avoid failure)
- To reduce post construction settlements foundation and thus the overall settlement of the top of the dam
- To reduce liquefaction risk(seismic area)
- To reduce leakage through the dam foundation
- To reduce seepage erosion potential
- To reduce uplift pressure under concrete gravity dams in conjunction with drain holes

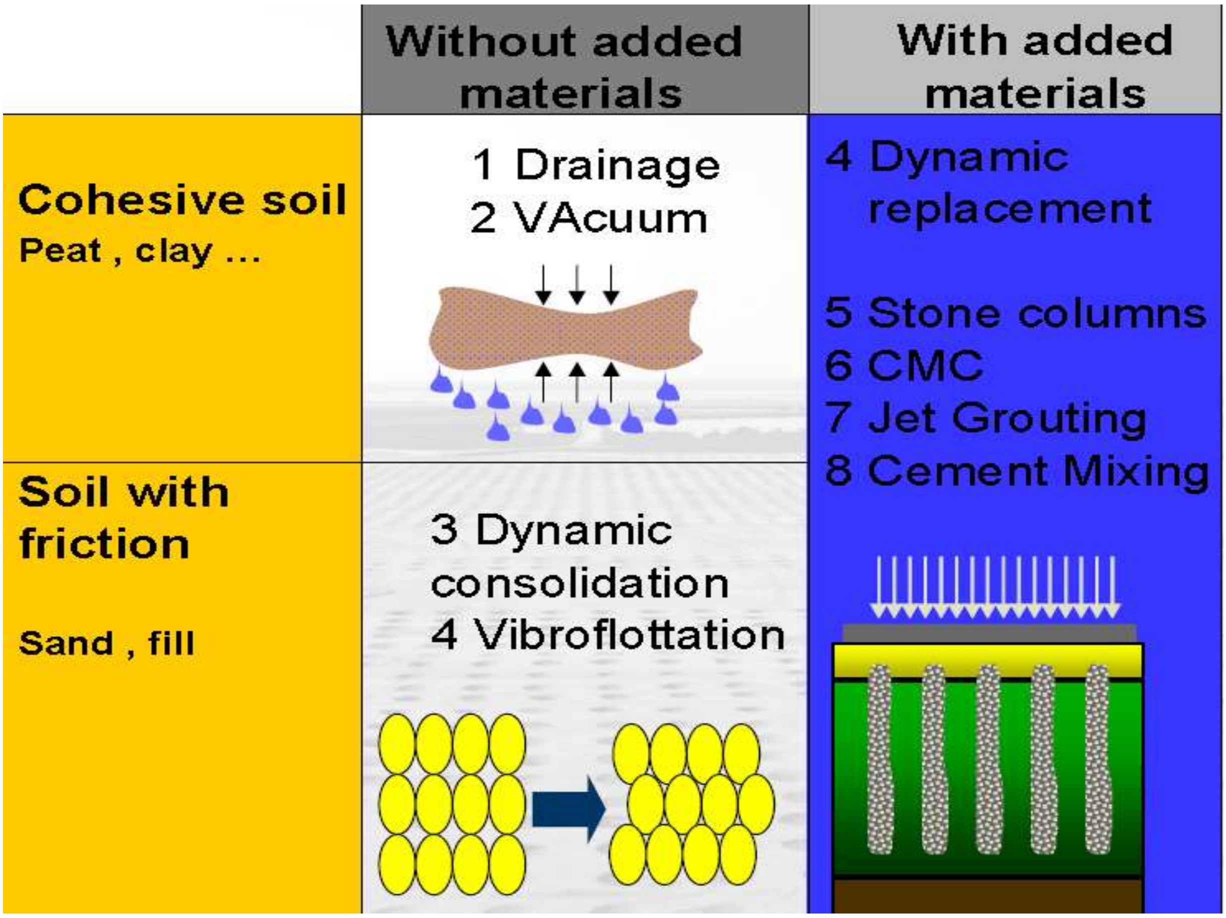
3. Main Improvement Techniques

- Temporary
  - e.g. dewatering or ground freezing, where the improvement is only during the application.
- Short-term
  - e.g. some forms of grouting, or use of diaphragm walls for ease of construction with longer term benefits.
- Long-term
  - e.g. soil nailing, vibro-replacement, curtain grouting of a dam, where the treatment is integral to the permanent works.

4. Effect on the Ground

- Change of state;
  - i.e. the same ground but made stronger, stiffer, denser, more durable.
- Change of nature;
  - i.e. the ground becomes a different material by inclusion of other materials.
- Change of response;
  - i.e. through the incorporation of other materials, the ground becomes a composite material with enhanced load-carrying or deformation characteristics.

5. **Foundation Treatment Techniques**



6. **Parameters for Concept**

- **Soil Characteristics**
  - Cohesive or non-cohesive
  - Blocks
  - Water content
  - water table position
- **Organic materials**
- **Soil thickness**
- **Structure to support**
  - Isolated or uniform load
  - Deformability
- **Site Environment**
  - Close to existing structure
  - Height constraints
- **Time available to build**

7. **Main Improvement Techniques**
   - **by vibration**
   - **by adding load**

- **by structural reinforcement**
- **by structural fill**
- **by admixtures**
- **by grouting**
- **by specialist dewatering**

8. CRITERIA & GROUTING TECHNIQUES

9. Purpose of Grouting

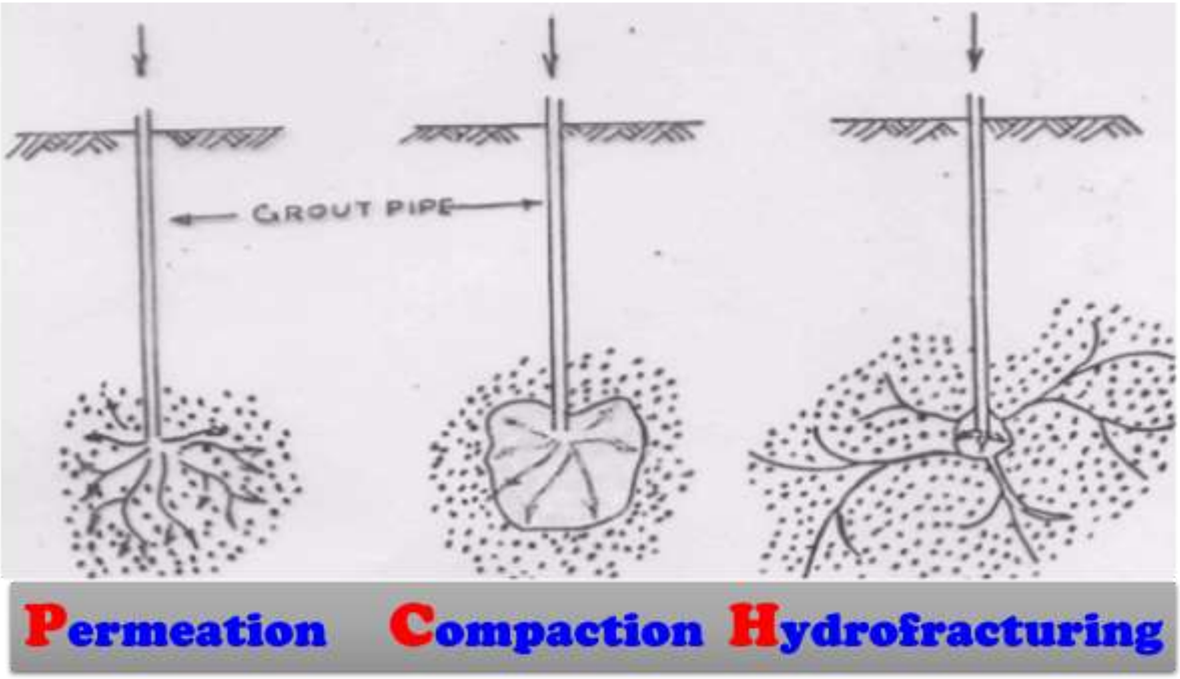
- Reduce leakage through the dam foundation
- Reduce seepage erosion potential
- Reduce uplift pressure under concrete gravity dams in conjunction with drain holes
- Strengthen the Dam foundation
- Reduce settlements in the foundation and thus the overall settlement of the top of the dam

The limiting Lugeon value given in the table, are recommended for deciding the necessity or otherwise of grouting. Lugeon values in excess of those given in the table would indicate that grouting is desirable.

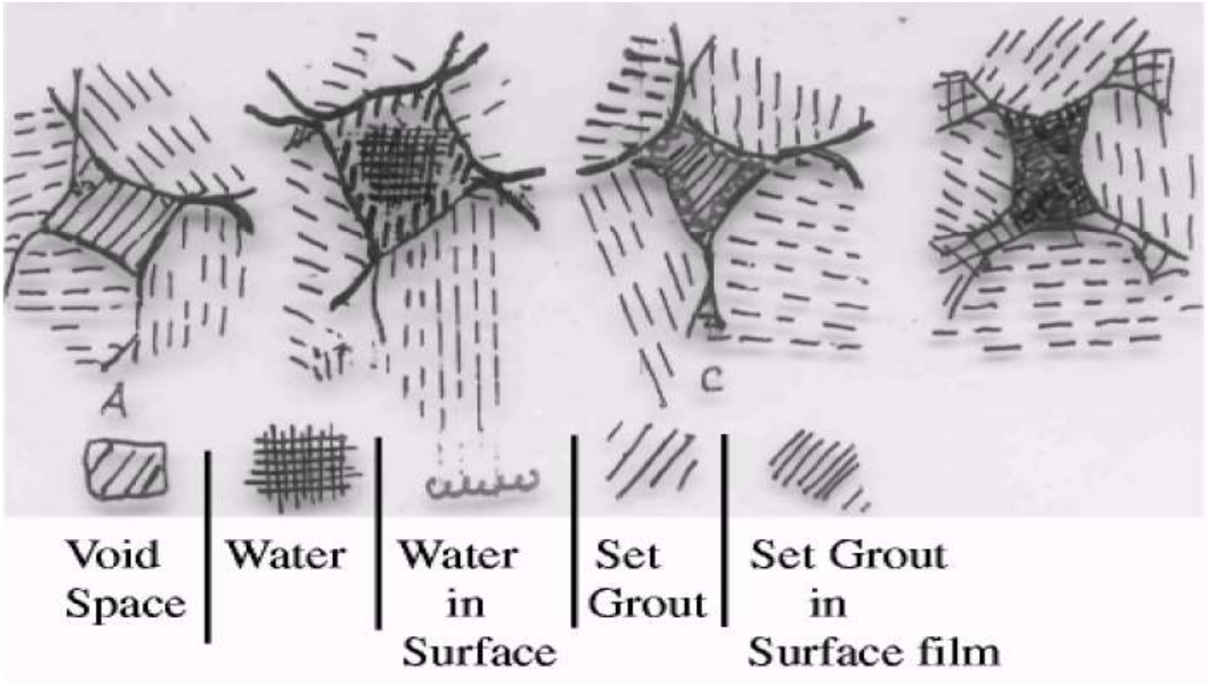
Item	Rock below Cut-off trench	Rock below Masonry dam
Group A		
Laminar flow	5 to 10	5 to 7
Group B		
Turbulent flow	3 to 5	3 to 5
Group C		
Dilation	1 to 3	1 to 3
Group D		
Washout and Hydraulic fracturing	1 to 3	1 to 3
Group E		
Void fills	3 to 5	3 to 5

(Adopted from clause 3.3.1 of IS 6066:1994)

10. Function of Grouting



11. **Possibilities of infilling of grout in Rock joints / Void space**



12. **Classification of Grouting**

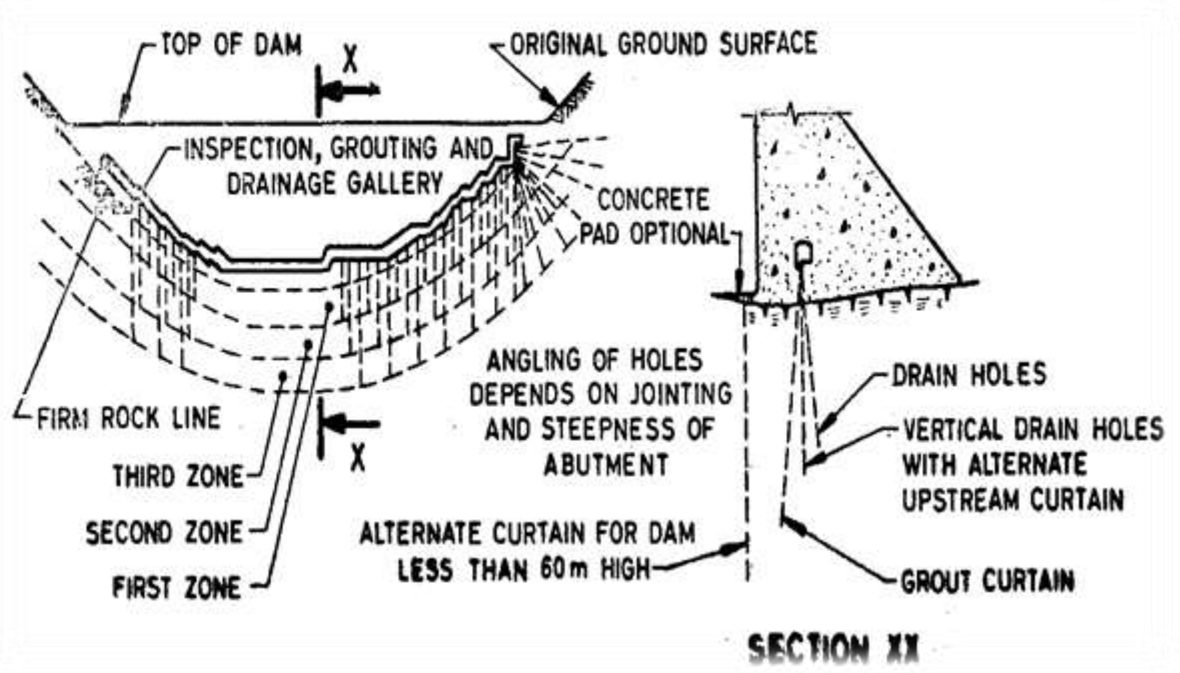
Foundation grouting can be classified into two types:

- **Curtain grouting**
- **Consolidation grouting**

13. **Curtain grouting**

- **Curtain grouting is designed to create a thin barrier (or curtain) through an area of high permeability.**
- **It consists of a single row of holes (3 to 5 rows in very permeable foundations) drilled and grouted to the base of permeable rock.**
- **It safeguards the foundation against erodability hazard.**

14. **Typical Profile of Curtain grouting**



15. Consolidation grouting

- **Consolidation grouting is designed to give intensive grouting of the upper layer of more fractured rock, in the vicinity of the dam core, or in regions of 'high' hydraulic seepage gradient.**

- **Under the plinth for a concrete face rock fill dam It is usually restricted to the upper 5 to 15 m While carried out in sequence, consolidation grouting is commonly applied to a predetermined hole spacing.**

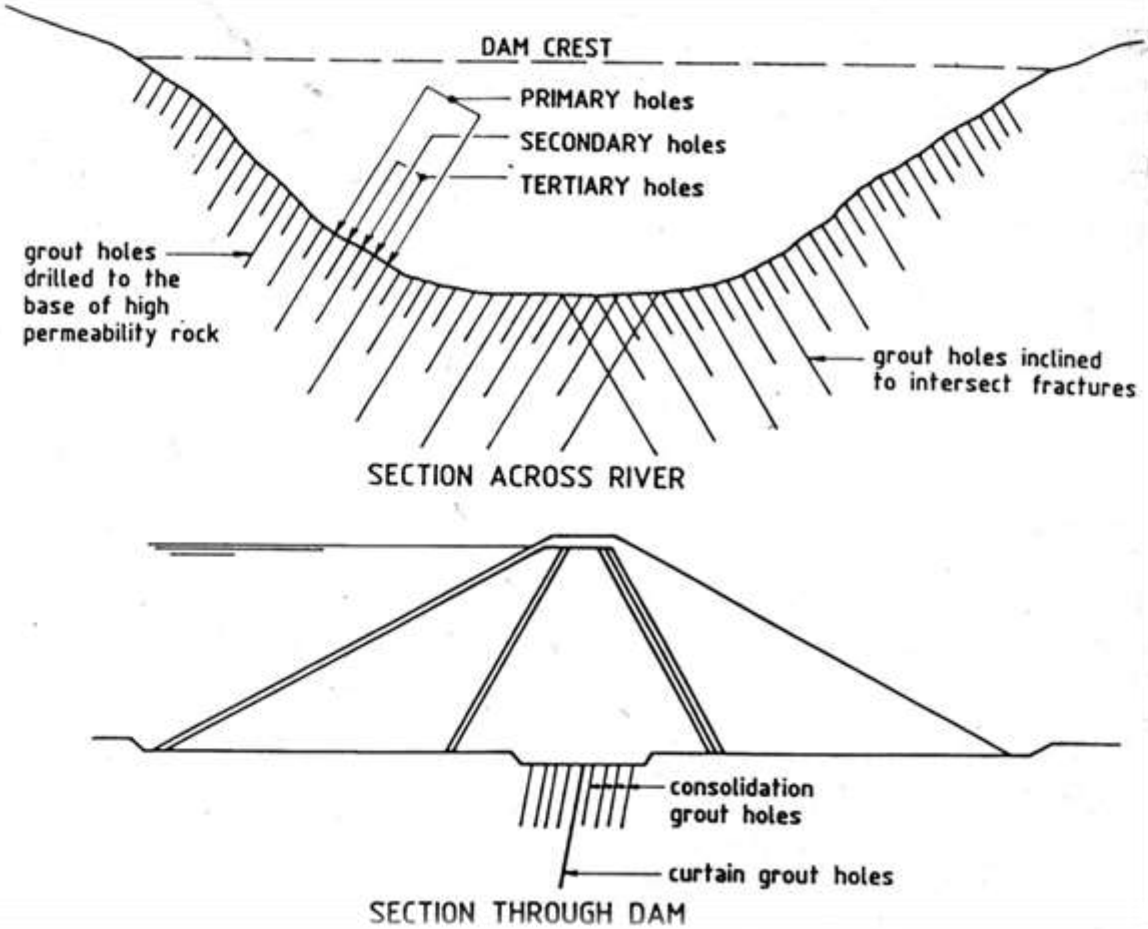
16. Depth of Grout holes

According to IS: 11293 (Part 1)-1985 “Guidelines for the design of grout curtains”, the following empirical criteria may be used as a guide:

D= H/3 to H

- **Where D is the depth of the grout curtain in meters and H is the height of the reservoir water in meters.**

- **The grout holes may be either vertical or inclined.**



17. Orientation of Grout holes

- **The orientation, plan and inclination of grout holes depend upon the type of joints and the other discontinuities in the foundation rock.**
- **The most common practice is to drill holes inclined towards the upstream at 5 to 10 degrees to the vertical.**
- **Apart from the gallery at the foundation level, there could be other galleries also.**

18. Sequence of Grouting Operations

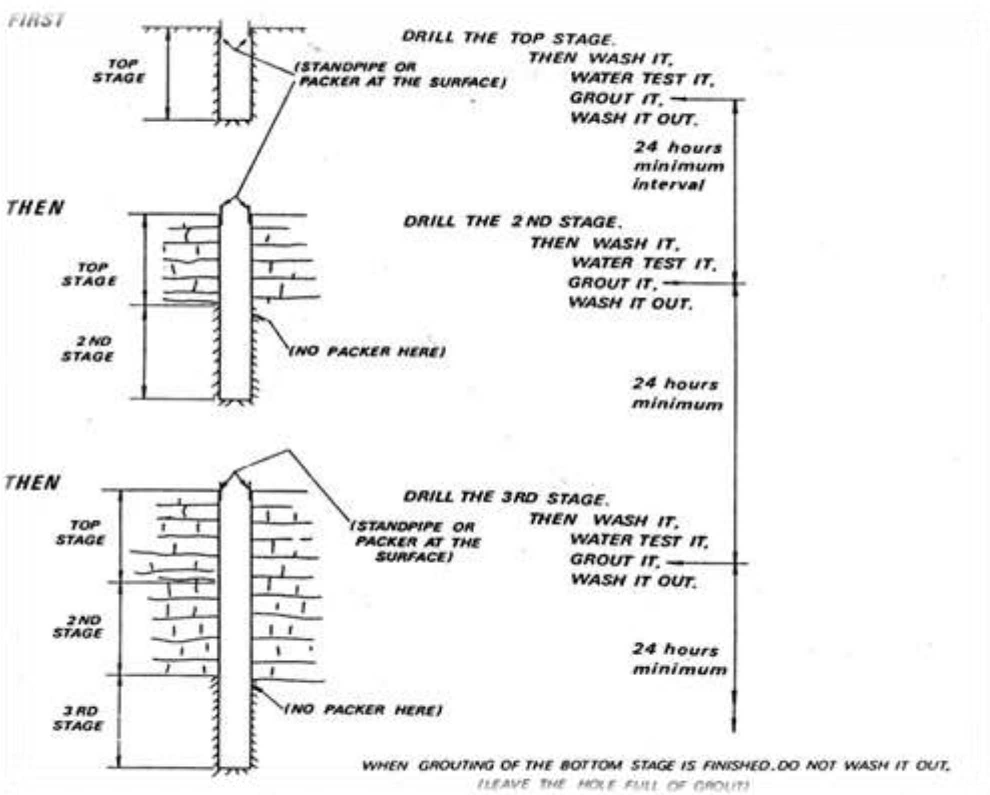
- **The holes are drilled and grouted in sequence to allow testing of the permeability before grouting and allow a check on effectiveness of the grout take by the foundation.**
- **Primary holes are drilled first, followed by secondary and then tertiary.**
- **The final hole spacing will commonly be 1.5 or 3, but may be as close as 0.5 m. This staged approach allows control over grouting operations.**

19. **Staging of grouting**

- **Downstage without Packer**
- **Downstage with Packer**
- **Upstage**
- **Full Depth**

20. **Downstage without Packer**

- **One of the preferred methods.**
- **Reduces the risk of leakage of grout to the top stage, allowing progressive assessment as to whether the hole has reached the desired closure requirement.**
- **Allows higher pressures to be used for lower stages, giving better penetration.**
- **Expensive, as drilling required every time.**

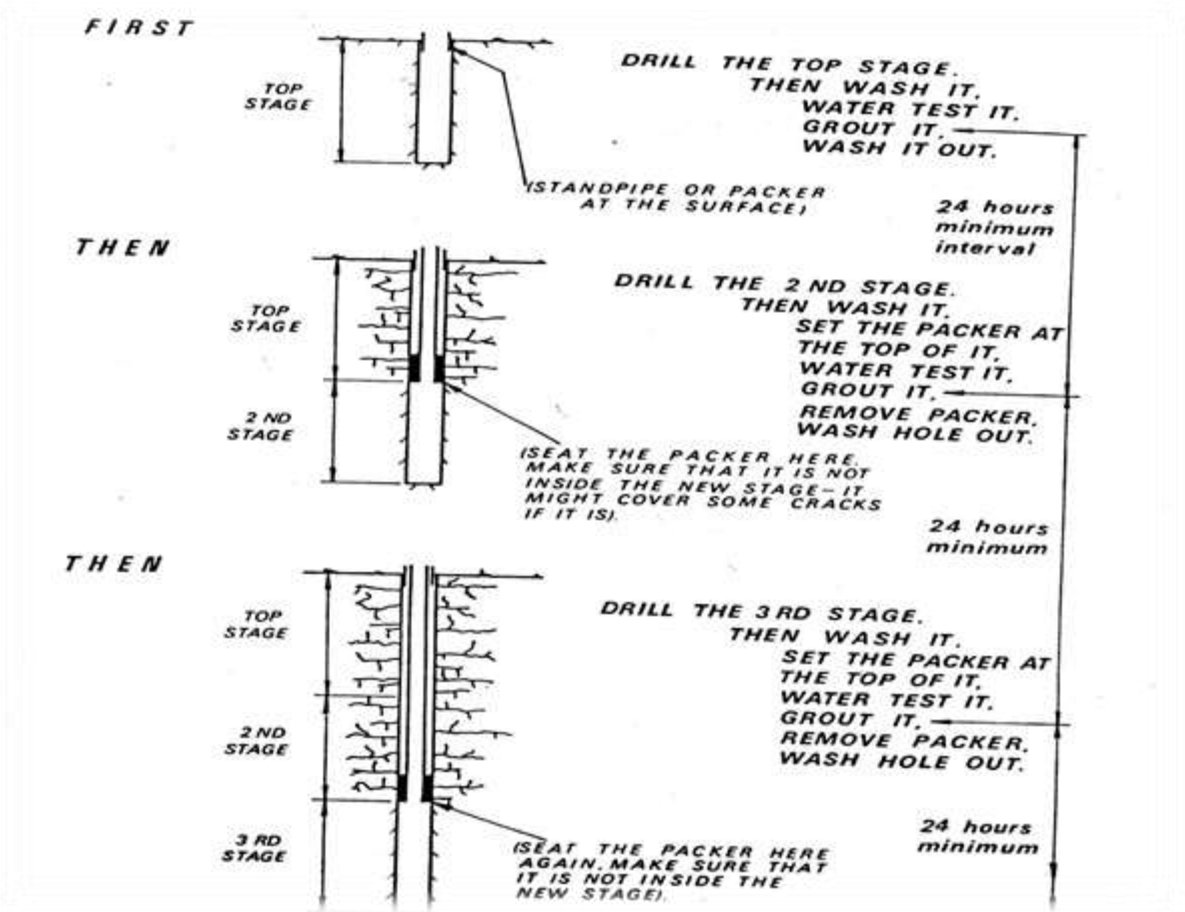


21. **Downstage with Packer**

- ◆ **Problem of seating, leakage past the packer.**
- ◆ **Bleeding water cannot be removed as in the first case.**

◆ Ewert prefers this method as the potential to fracture the rock in the upper levels if packers are not used.

◆ Expensive because each time drilling is required.



22. Upstage

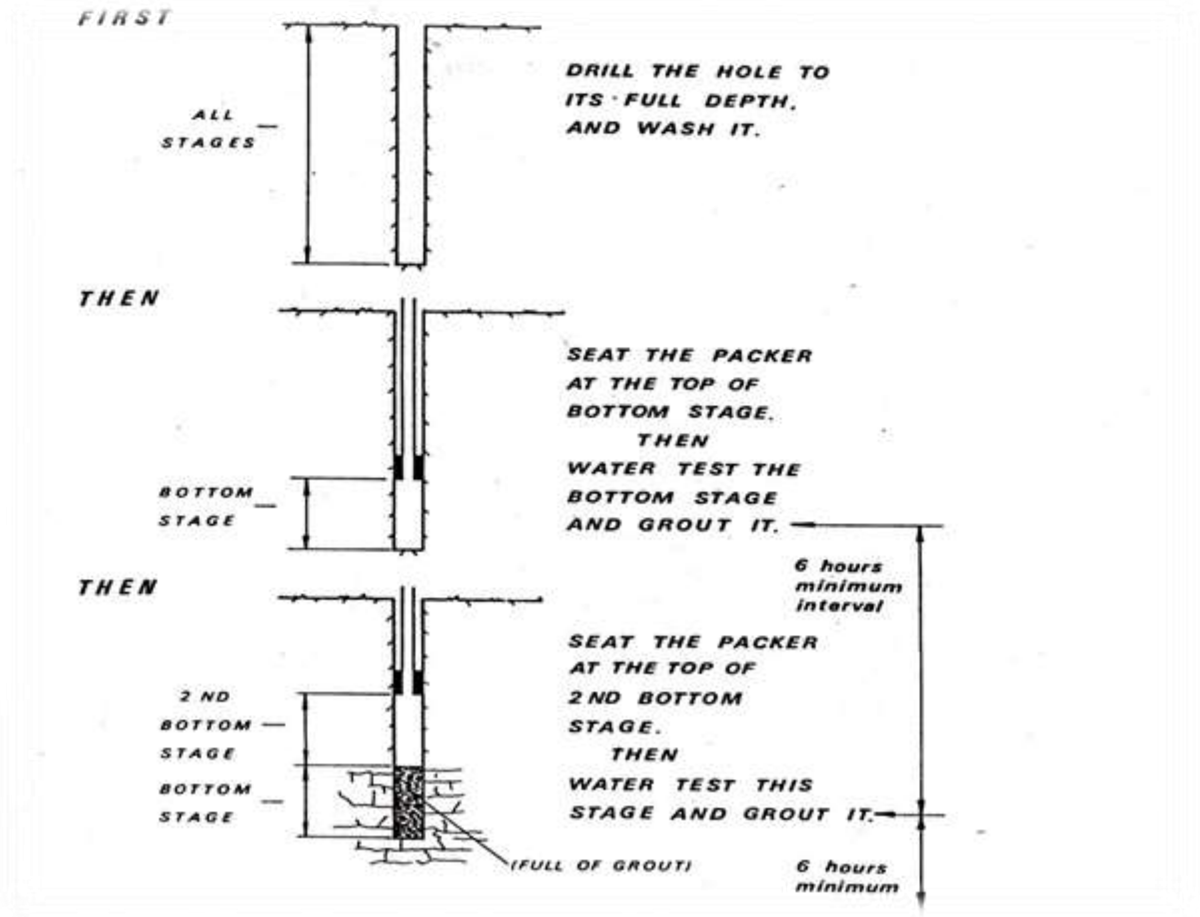
◆ Full depth is drilled in one go and grouting is done in stages using packers.

◆ Does not allow progressive assessment of the depth of grout hole needed to reach closure.

◆ Cheaper, as the drilling rig is set only once, but savings may be offset by the need for more conservative total depths.

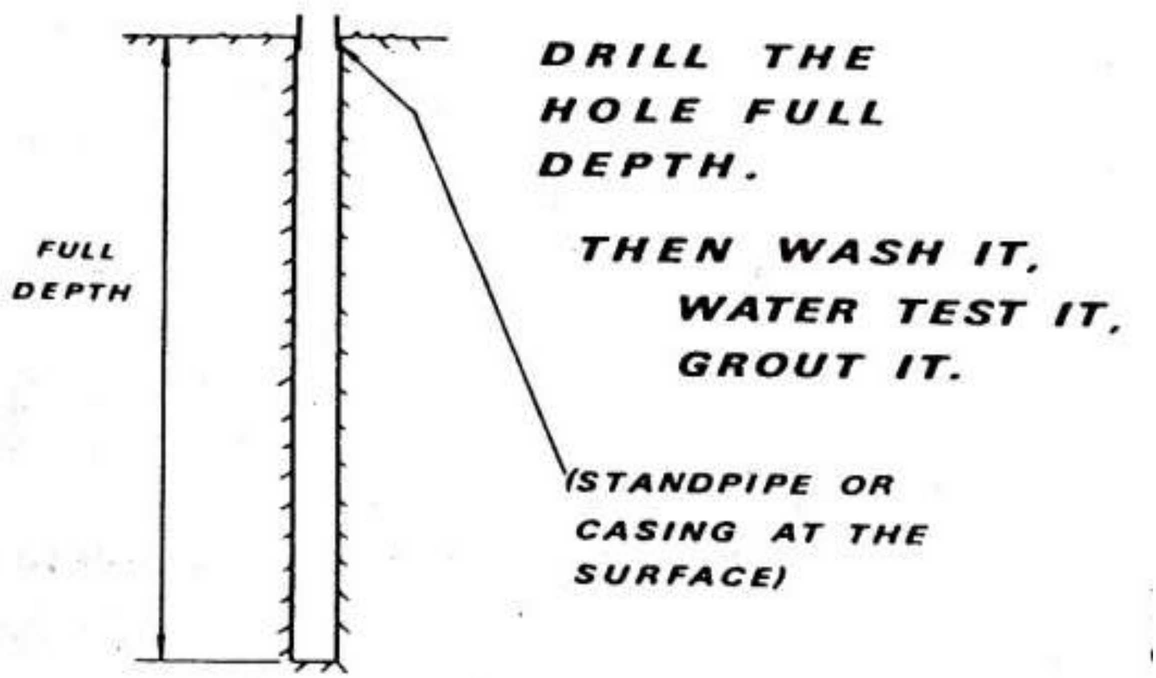
◆ Appropriate for secondary and tertiary holes.

◆ Bleeding water cannot be removed.



23. Full Depth

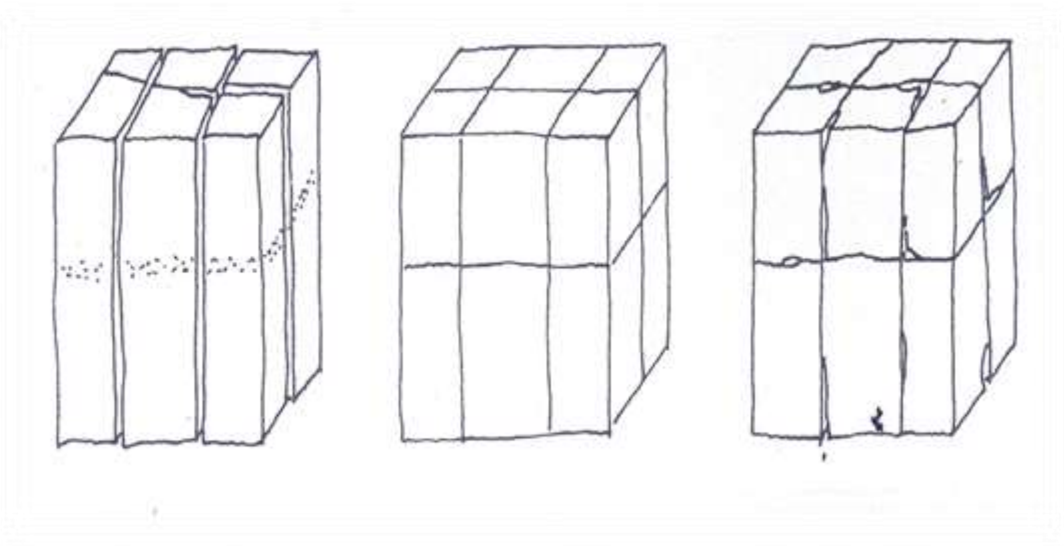
- **Full depth of the hole is drilled in one go.**
- **The hole is washed and grouted.**
- **Does not allow proper assessment of where grout take is occurring or reduction in Lugeon value is taking place.**
- **Grouting pressures are limited.**
- **It is not an acceptable method except for consolidation grout holes**



24. **Lugeon Test**

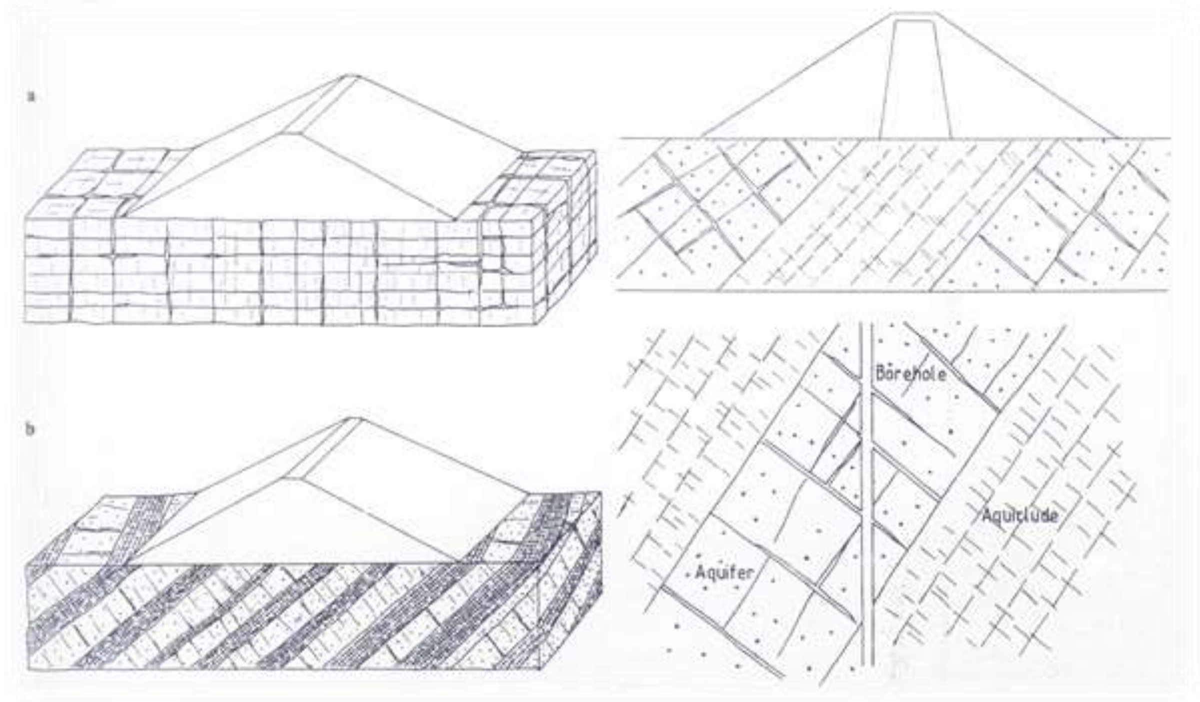
- **Decision to install grout curtain depends largely on the results of WPT**
- **Tests not in harmony with grout takes, small quantity of water, large cement take**
- **Impossible to reduce permeability even though originally permeable & large takes**
- **Very little seepage though originally high permeability but grout takes were low**
- **No proportional head reduction despite high take**

25. **Permeability of joints depends on the Orientation of joints**



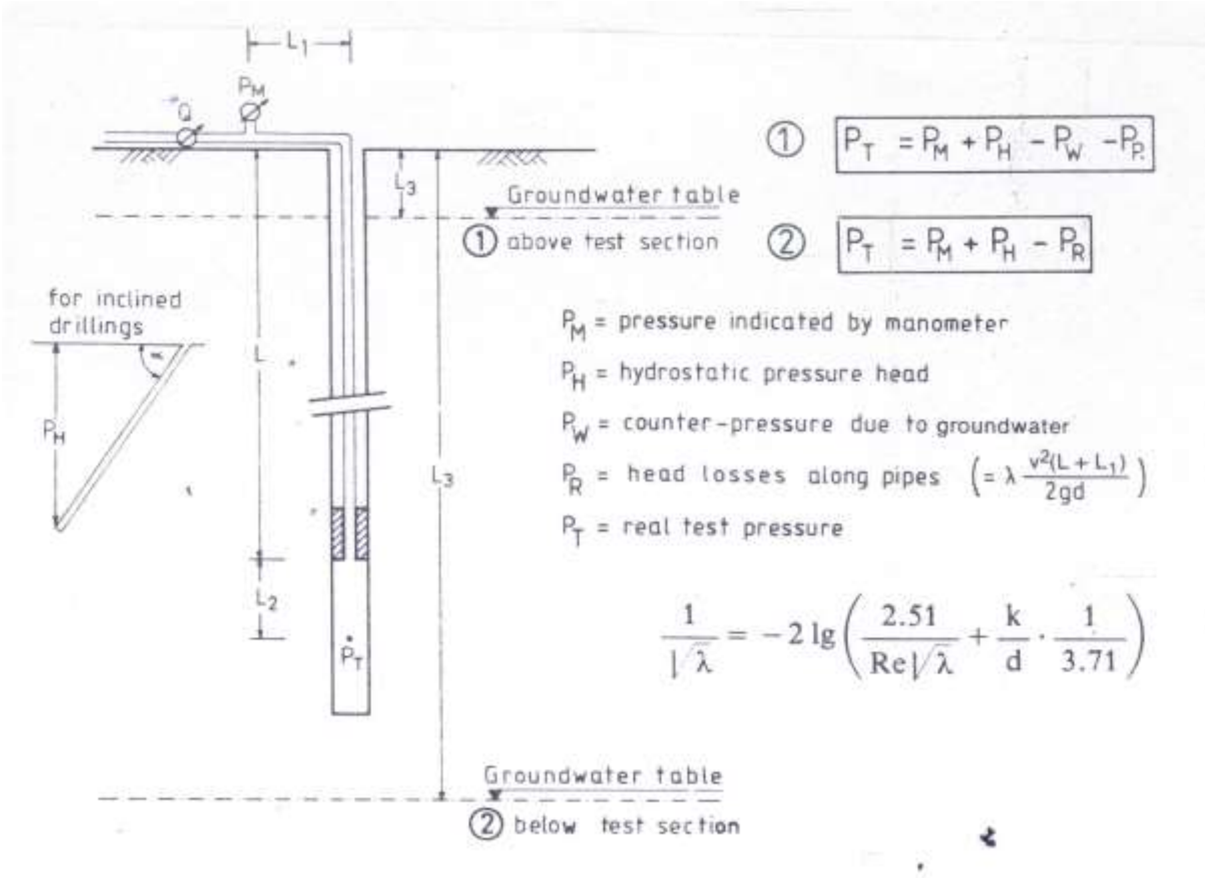
26.

Different Conditions

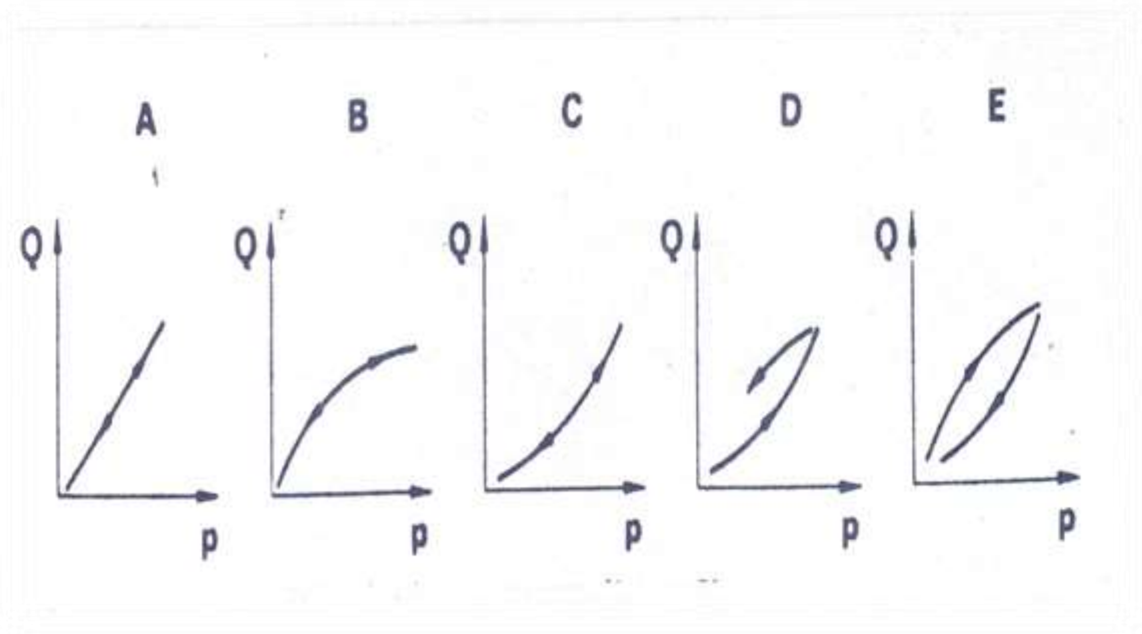


27. Water Pressure Test

- **Tests may be conducted in cycles 1 to 5**
- **Corrections in pressure are required to take care of elevations and inclinations**
- **Groundwater back pressure, skin friction**
- **Natural permeability needs only one cycle**
- **Pressure may be 0.1 or 0.2 MPa in the first cycle**
- **Results are plotted in two ways**

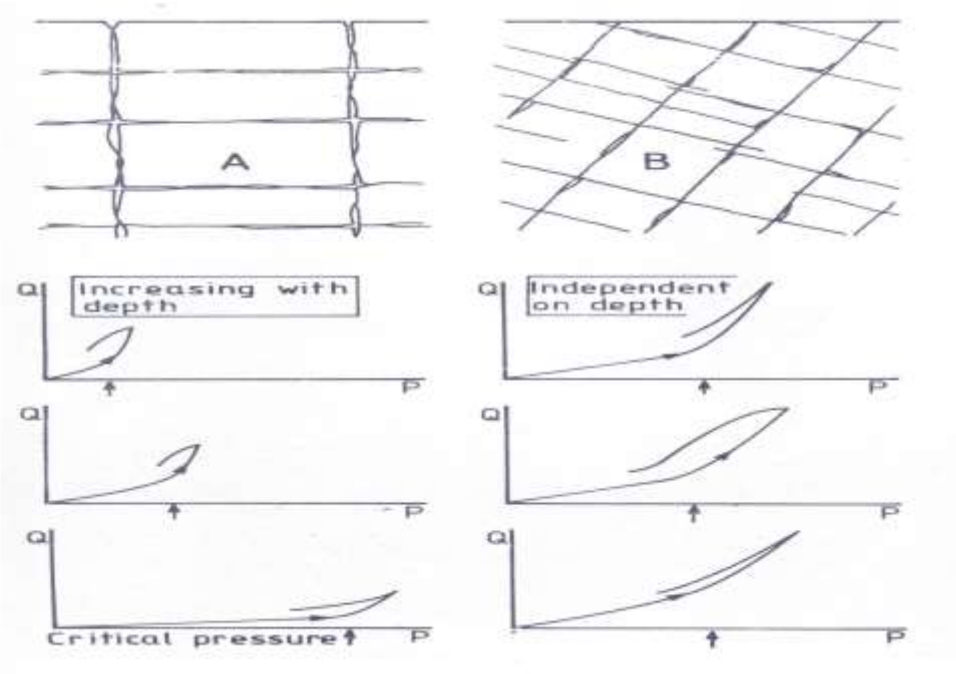


28. Pattern of WPT

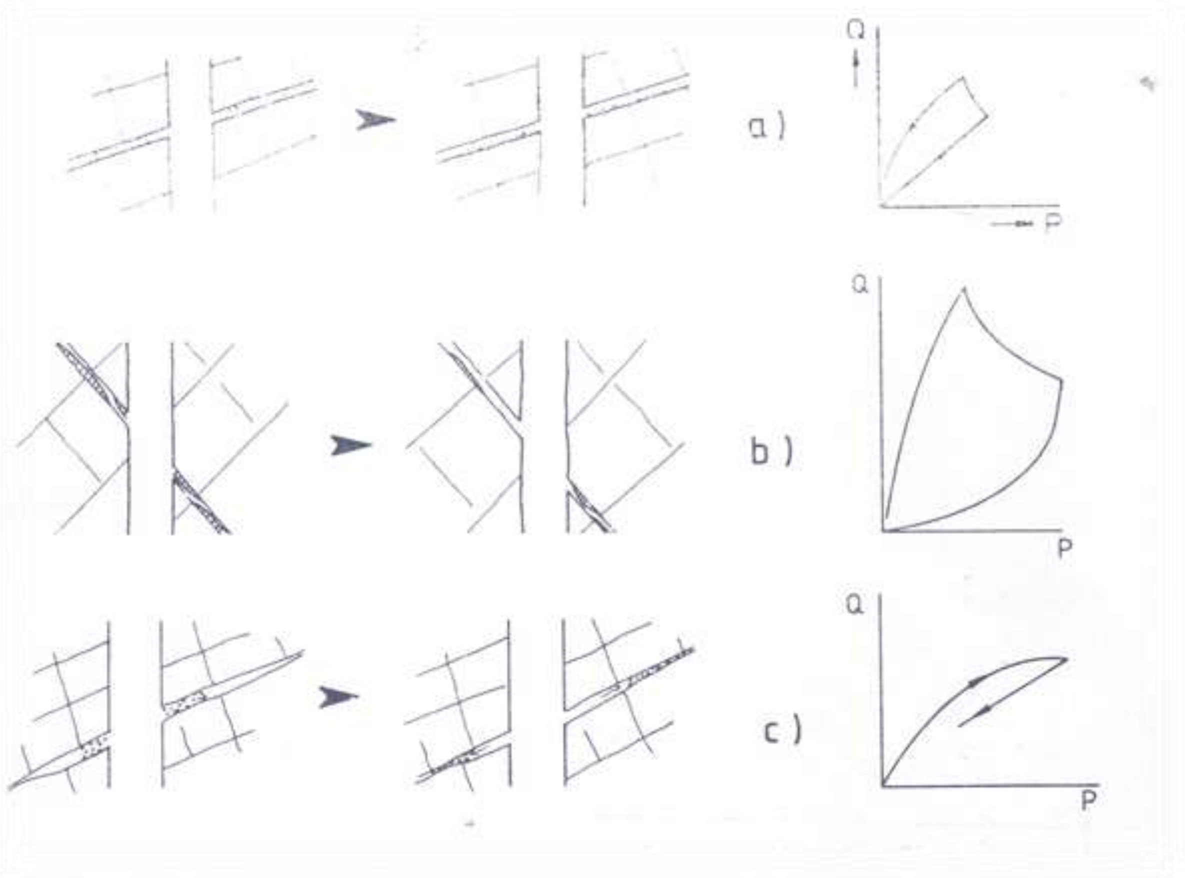


29. Interpretation

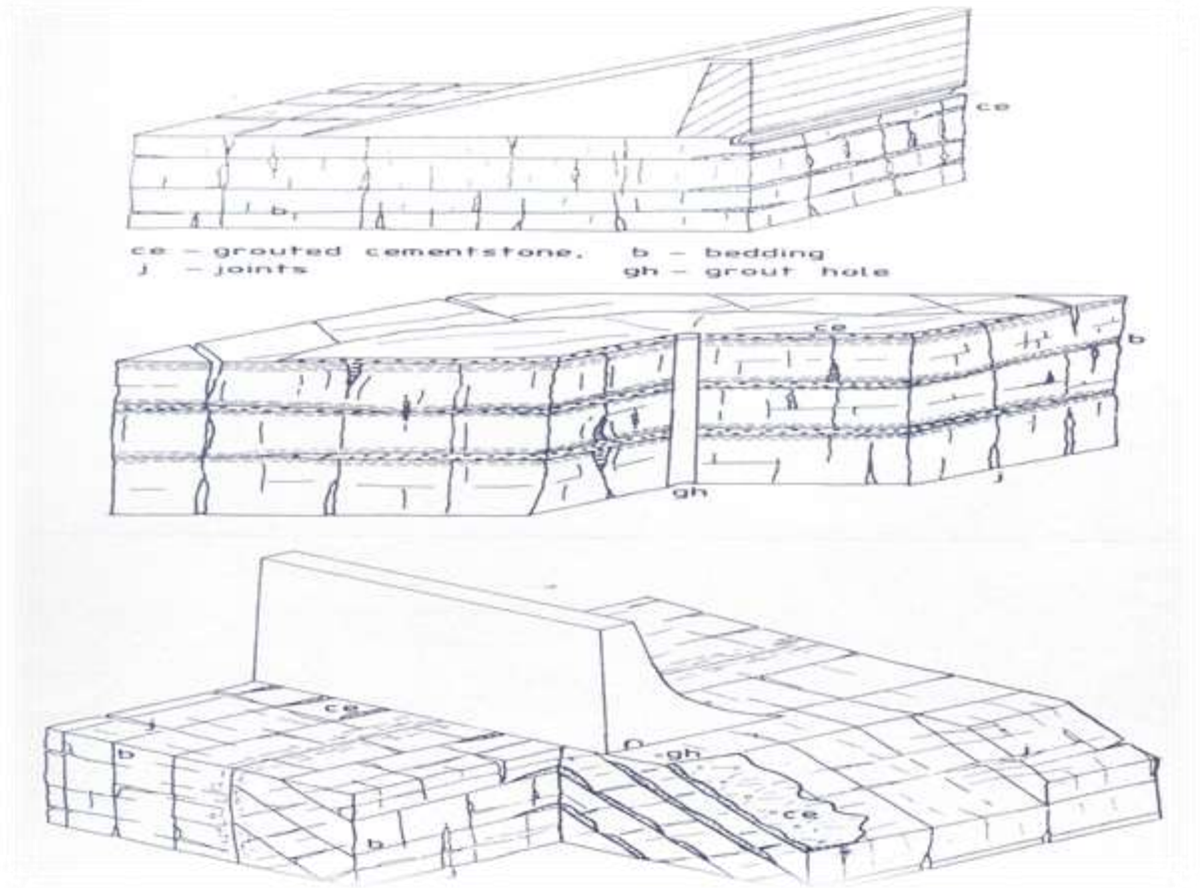
30. Flow vs. Orientation



Flow vs. Orientation Contd.



31. Geologic Structures



32. Conclusions

- **Consider, if grouting is really required.**
- **Take the geological structures and their orientation into account.**
- **Design Grout Mix appropriately.**
- **Limited utility of Water Pressure Test.**
- **Review the Acceptance criterion.**
- **Consider chemical grouting in special cases.**

33. PARTIAL & POSITIVE CUT-OFF

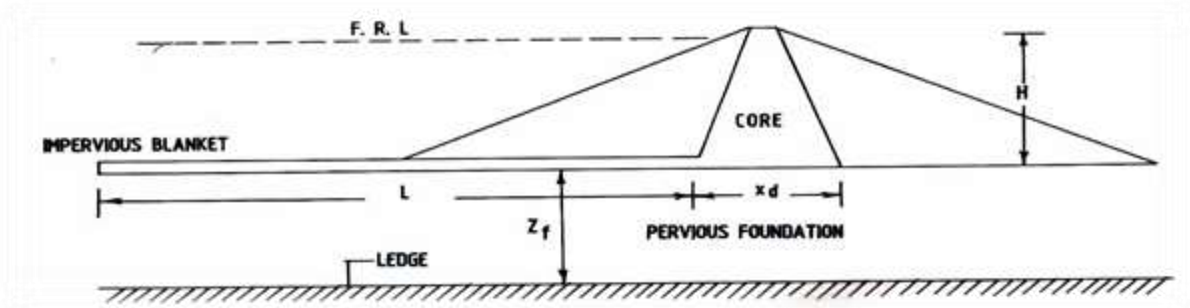
- **Functions & Design Requirements of Cut-off:**
- **<u>Functions & of Cut-off:</u>**
  - **To reduce loss of stored water through foundations and abutments.**
  - **To prevent subsurface erosion by piping.**

- The alignment of the cut-off should be fixed in such a way that it's central line should be within the base of the impervious core.
- In case of positive cut-off, it should be keyed at least to a depth of 600 mm into continuous impervious sub-stratum/ Rock.
- The partial cut-off is specially suited for horizontally stratified foundations with relatively more pervious layer near top.
- The depth of the partial cut-off in deep pervious alluvium will be governed by:
    - Permeability of substrata
    - Relative economics of depth of excavation governed usually by cost of dewatering versus length of upstream impervious blanket.

34. IMPERVIOUS BLANKET

The horizontal upstream impervious blanket is provided to increase the path of seepage when full cut-off is not practicable on pervious foundations.

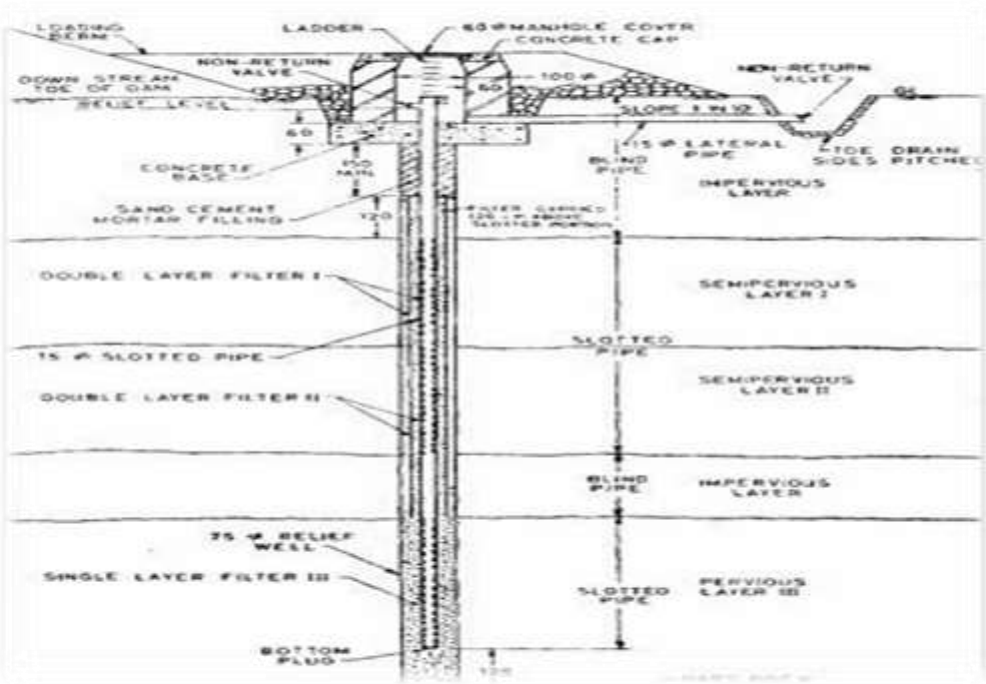
- Impervious blanket shall be connected to core of the dam.
- The material used for impervious blanket should have far less permeability than the foundation soil.
- To avoid formation of cracks, the material should not be highly plastic.
- Reference may be made to IS: 1498 for suitability of soils for blanket.
- A 300 mm thick layer of random material over the blanket is recommended to prevent cracking due to exposure to atmosphere.
- Impervious blanket with a minimum thickness of 1.0 m and a minimum length of 5 times the maximum water head may be provided.

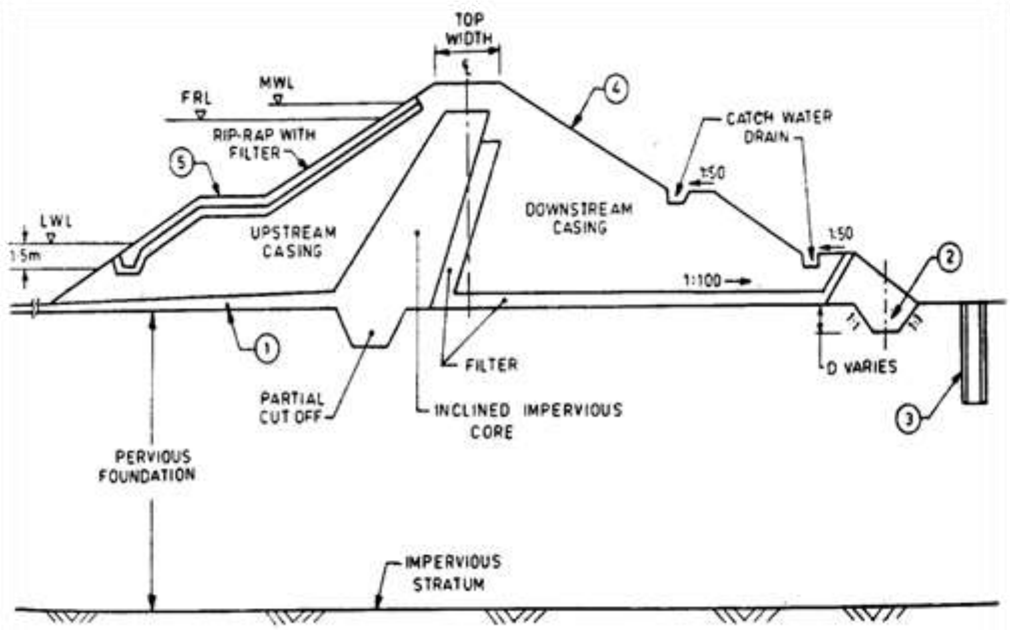


35. RELIEF WELLS

- To ensure safety of the earth dam in cases where the cut-off is partial or reliance is placed on an upstream blanket for controlling under seepage.

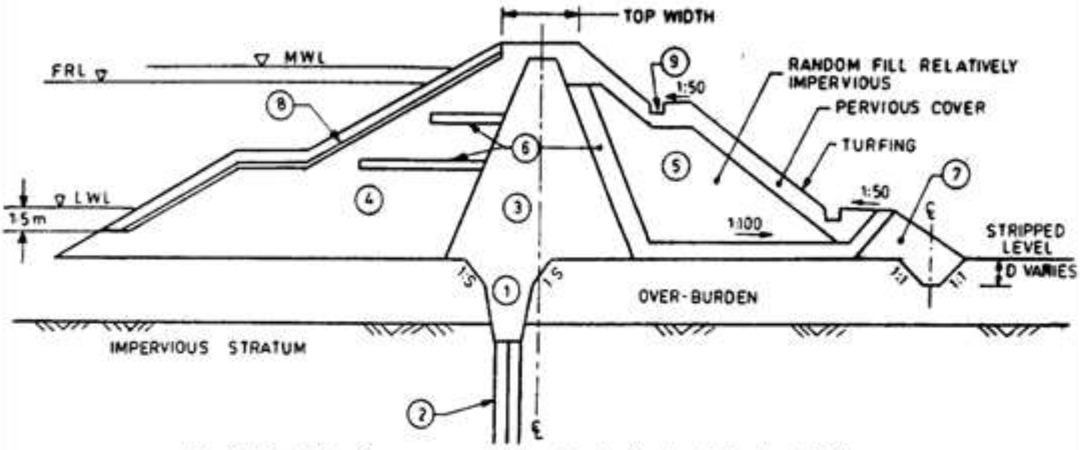
- This may be effectively done by installing a system of relief wells suitably spaced which will reduce the intensity of the under seepage pressure and render the seepage water practically.
- A relief well is a small drainage well (45 to 90 cm in dia) near about the downstream toe, with a pipe having narrow slots, placed in the centre and surrounded by graded filter.





1. Impervious Blanket
2. Rock Toe and Toe Drain
3. Relief Well
4. Turfing
5. Berm

2B Embankment Dam with Inclined Core and Partial Cut-Off



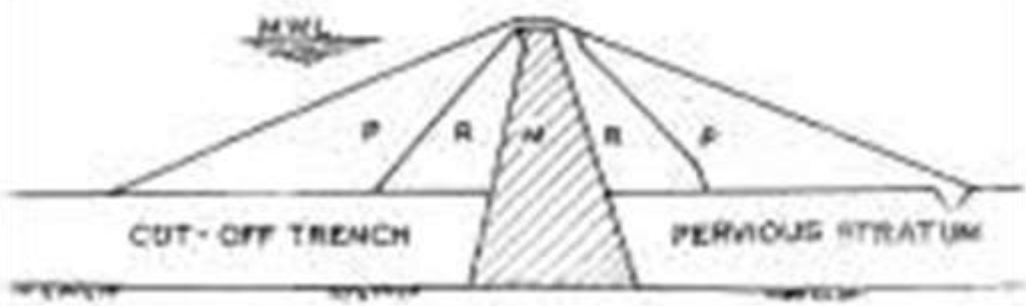
1. Positive Cut-off
2. Grout Curtain
3. Central Impervious Core
4. Upstream Casing
5. Downstream Casing
6. Inclined and Horizontal Filter
7. Rock Toe and Toe Drain
8. Riprap with Filter
9. Catch Water Drain

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

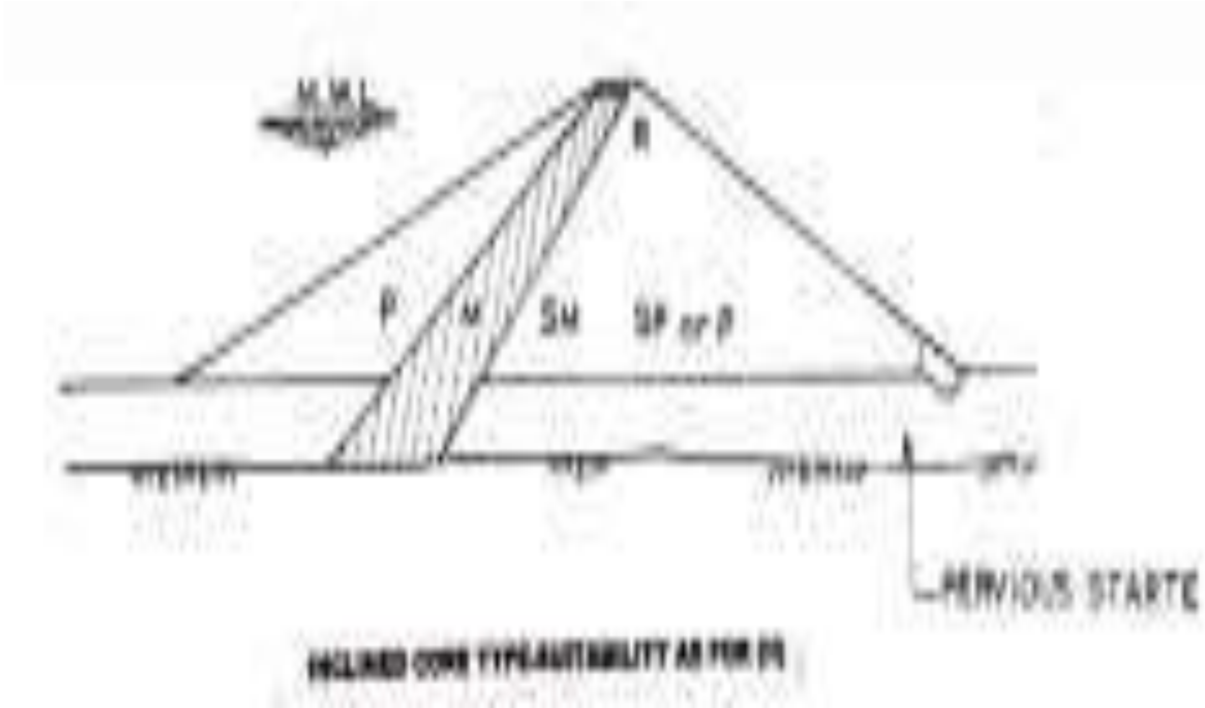
2A Embankment Dam with Central Core and Positive Cut-Off

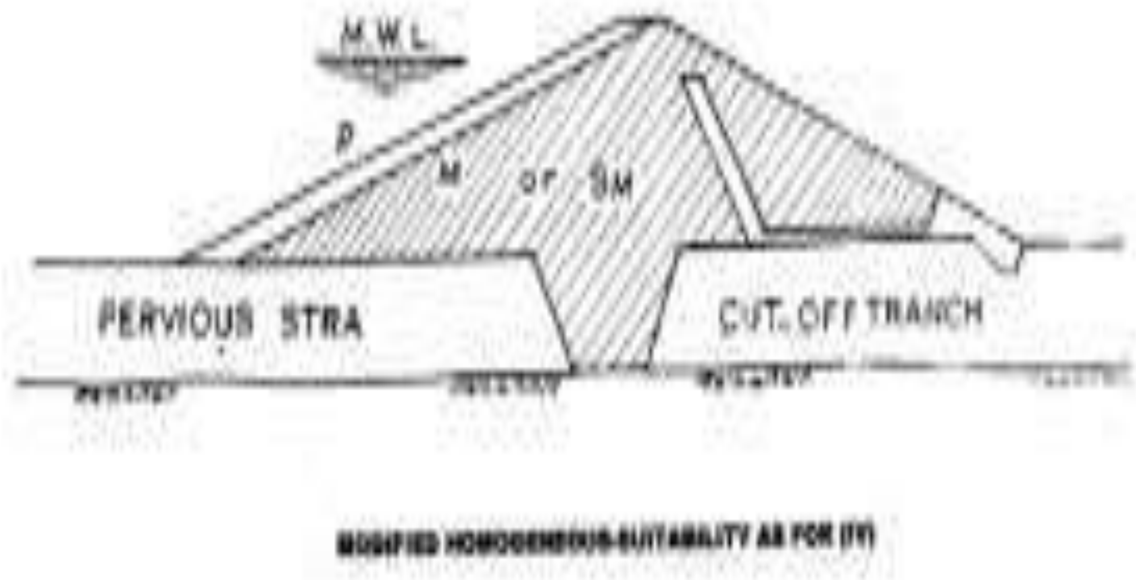
36. METHODS OF TREATMENT OF DIFFERENT FOUNDATION MATERIAL-OVERBURDEN & ROCK (IS 4999 :1991 & IS 6066 :1994)

DAMS ON SHALLOW PERVIOUS FOUNDATIONS

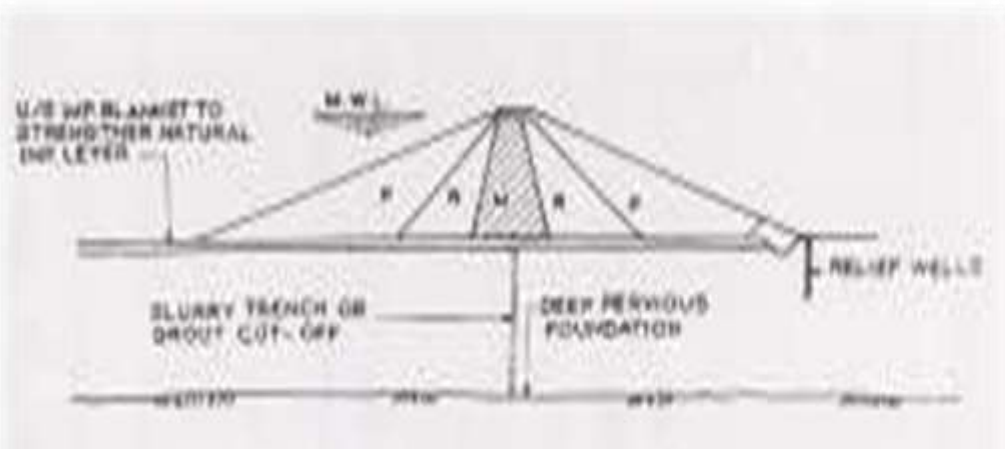


CENTRAL CORE TYPE-SUITABILITY AS FOR (I) CORE THICKNESS ADJUSTABLE ACCORDING TO AVAILABLE IMPERVIOUS

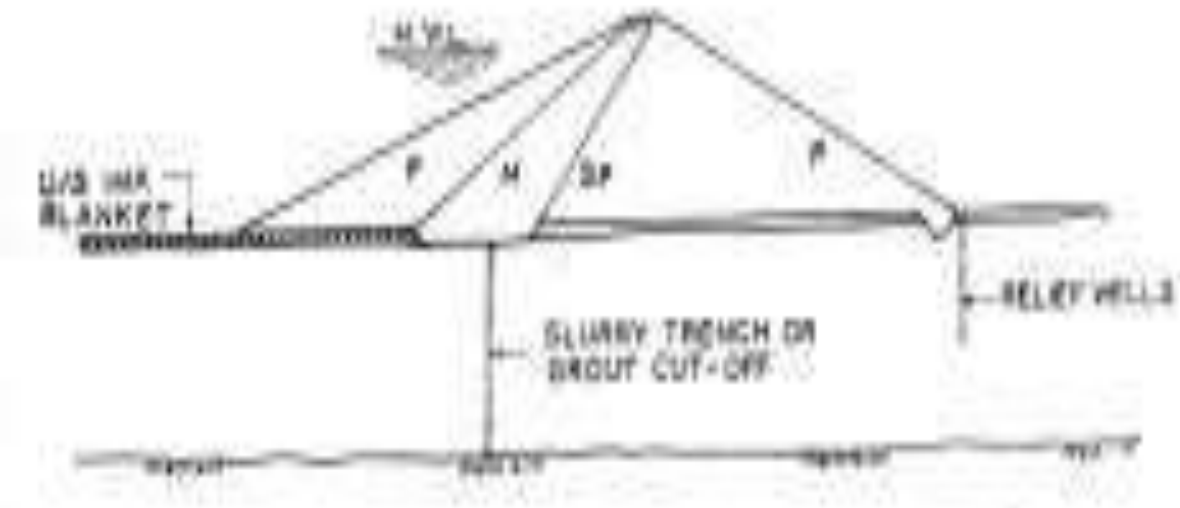


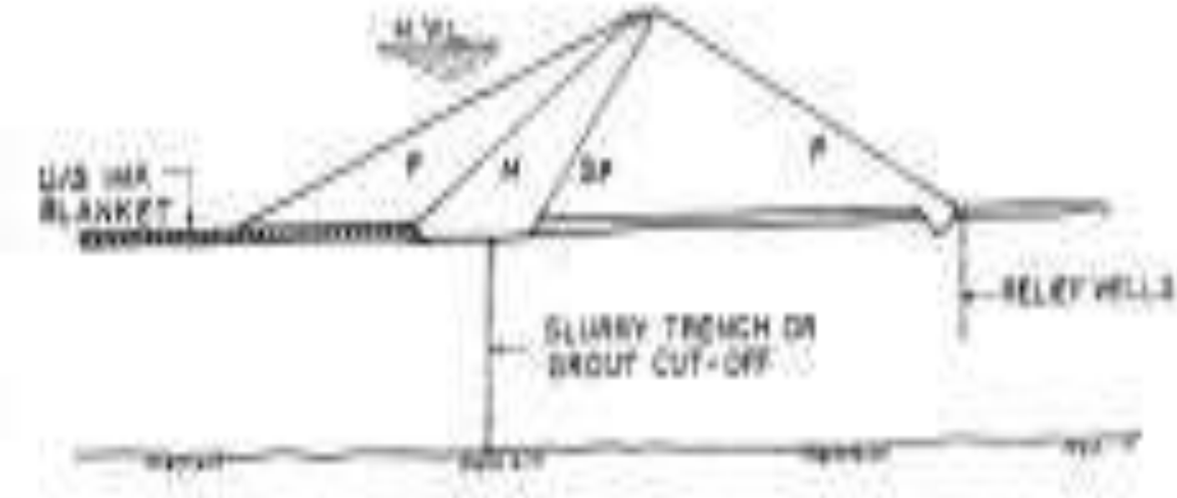


37. DEEP PERVIOUS FOUNDATIONS WITH IMPERVIOUS TOP STRATUM



CENTRAL CORE TYPE-SUITABILITY AS FOR [illegible] POSITIVE CUT-OFF MAY REPLACE IMPERVIOUS BLANKET




U/S IMP.
BLANKET
P
H
SP
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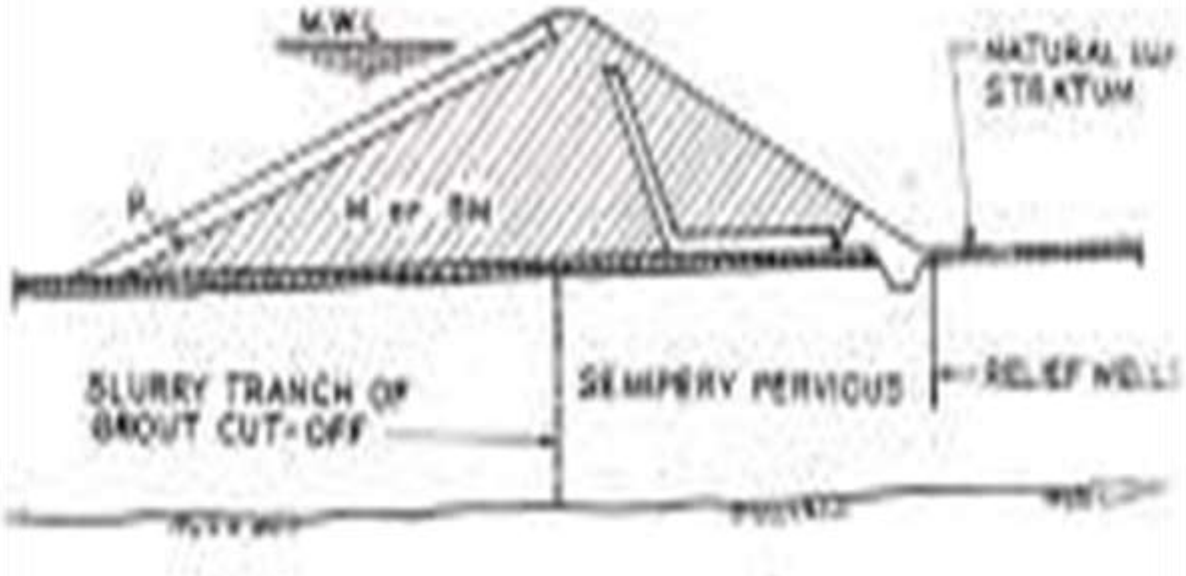
RELIEF WELLS
SLURRY TRENCH OR
GROUT CUT-OFF


M.W.L.
NATURAL
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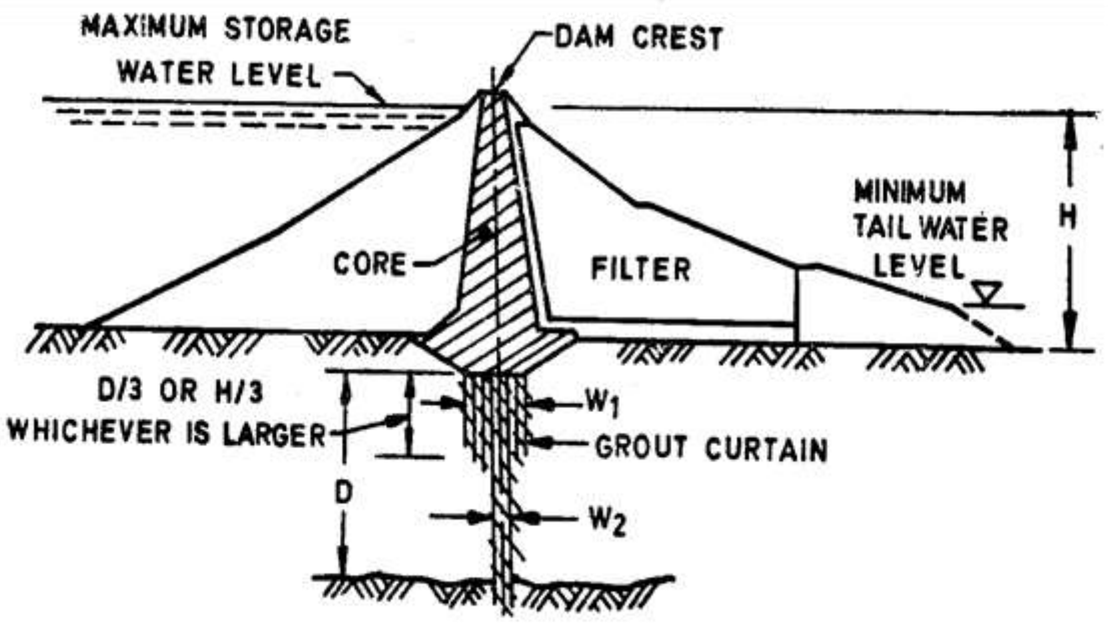
GROUT CUT-OFF
SEMIPERV PERVIOUS
RELIEF WELL



Recommended Maximum Curtain Width

W_1 = $H/3$ to $H/5$ for stable grouts clay cement, bentonite cement.

W_2 = $H/7$ Sodium silicate — aluminate, acrylamide.

FIG. 1 GROUT CURTAINS IN PERVIOUS SOILS

38. INVESTIGATION REQUIRED

- **To judge the overall permeability in order to enable a preliminary assessment to be made of the degree of impermeabilization desired and the feasibility of achieving the same.**
- **To explore the local variation in the grain size distribution and permeability in order to ascertain the groutability of the various strata and the extent of ungroutable layers.**
- **To investigate salt content of the soil as well as ground water to identify presence of salts which may inhibit gellation.**

39. PERMEABILITY TESTS IN-SITU

- **The in-situ permeability is governed by the size, extent and spacing of layers of high permeability zone.**
- **This is conducted by constant mead method in accordance with IS 5529 (Part-I) :1985.**

- **The average permeability of pervious deposits may be determined by pumping out test in accordance with IS 5529 (Part-I):1985.**

40. GROUTING OF PERVIOUS SOIL

- **These are applicable where the primary purpose of grouting is to reduce the permeability of the soil. In such cases consolidation of the soil is the primary objective.**
- **The seepage through a pervious stratum is generally governed by the presence of a few pockets or layers of high permeability.**
- **It is necessary to treat only such layers to achieve the necessary reduction of permeability.**
- **It is judge not merely in terms of the grain sige distribution of each individual layer or lense but in terms of its contribution to the over permeability.**

41. GROUTING METHODS & THEIR SELECTION

- **Pervious soils are generally heterogeneous, and the grain size distribution may change abruptly over a short distance.**
- **The grout flow generally concentrates along layers or pockets of coarser and relatively pervious soils.**

Hence, it is necessary to treat short lengths of grout holes at a time and repeat injections to ensure that the least pervious and fine grained soils are treated thoroughly.

- **The method of grouting selected should, therefore, satisfy the following requirements:**

  A. **Soils of different characteristics should be treated individually.**
  B. **It should be possible to treat short sections of the bore-holes in any desired sequence and repeat the injection, if required.**
  C. **Leakage along the boreholes shall be prevented.**

42. Description of Grouting Methods

The following methods are generally followed for grouting of overburden soils:

<u>Rising Tube</u>

- **In this method, grouting is done through the casing which is driven to the bottom of the hole.**
- **The tube is withdrawn a short distance and grout is injected through the open end into the cavity left by the tube as it is raised.**

- In this manner the tube is lifted progressively until the entire depth required to be grouted is treated.

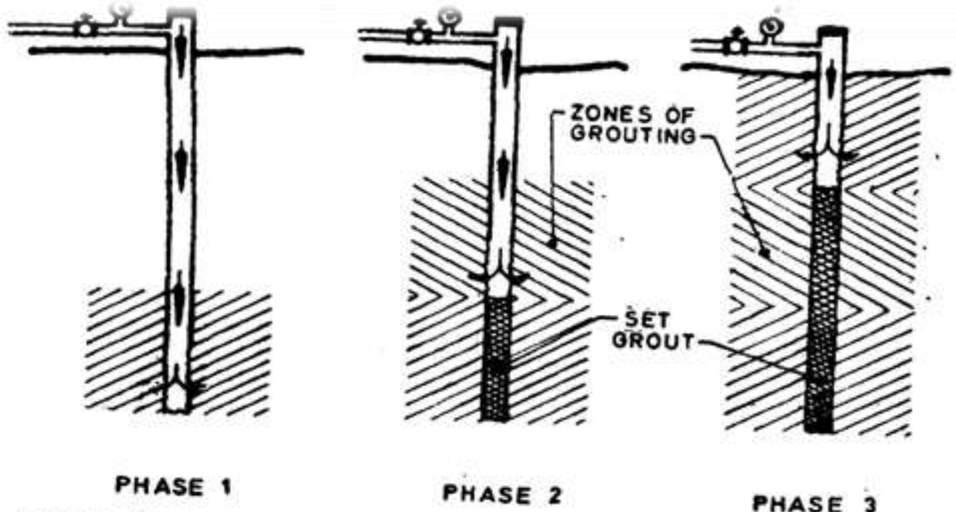


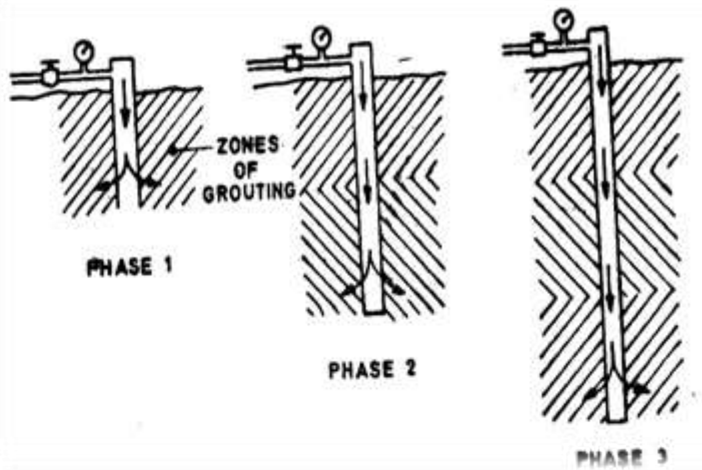
FIG. 1 GROUTING BY ASCENDING STAGES RISING TUBE METHOD

3

Description of Grouting Methods

Descending Stage

- **In this method, grouting is done through the lower open end of the grout pipe in short stages of 1 to 2 metres starting with the top of the grouted zone.**
- **The process involves repetition of a sequence of operations comprising drilling through the length of each stage and grouting followed by redrilling.**



__Grouting Through Tubes with Sleeves__

- **In this system of grouting, a pipe, with rubber sleeves fitted at 30 cm intervals, is installed in the borehole by filling the annular space around the tube by a sheath of clay cement grout.**

- **Grouting is done by seating a set of double packers opposite the sleeves which open under pressure. The sheath grout is cracked under pressure every time injections are made.**

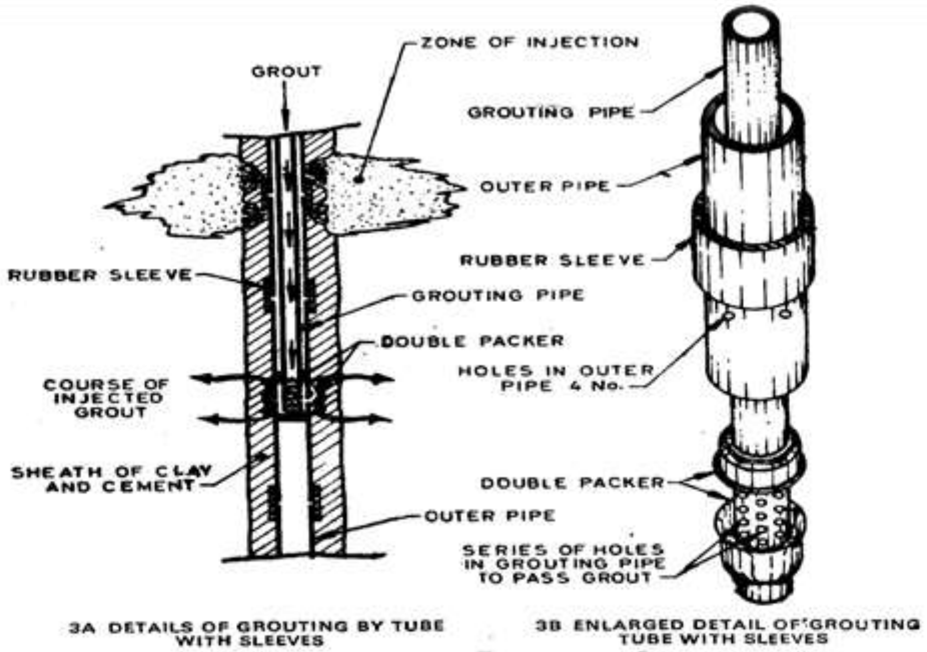


FIG. 3 GROUTING THROUGH TUBES WITH SLEEVES

43. Grouting in Rock- CHOICE OF GROUTING MATERIALS & MIXTURES

- **Particle size (in suspension) should be small enough, so that the grout can penetrate the soil easily.**

- **The viscosity of the grout mix should be sufficiently low so that the mix canStravel sufficient distance in the soil to achieve an economical and practicable spacing of holes.**

- **After penetration into the soil, the grout should form a deposit which will not be eroded by the pressure gradient imposed on the curtain over the entire serviceable life of the structure.**

44. Grouting in Rock- BLANKET GROUTING/CONSOLIDATION GROUTING

- **The normal practice of splitting the spacing starting with an initial spacing of 6 to 12.0 m for each of the rows.**

- Final spacing depends upon the joints pattern, normally 3.0 m is provided.
- Special geological condition requires closer spacing.

45. Grouting in Rock- CURTAIN WIDTH

- The curtain width at the core contact should match the core base, usually width in the range of 1/3 to 1/5 head is provided.
- The main curtain should extend to rock or impervious stratum and the width should be reduced from the width at core contact to the main curtain width, about ¼ the depth of the pervious alluvium.
- The main curtain should have two or more rows depending upon the requirements strata. For clay cement silicate aluminate grouting the main curtain should have a width of 1/7 of head.

46. MIX PROPORTION & GROUTING PRESSURE

✓ **Grout Mixture**

- Rock grouting is usually performed with a mixture of cement and water with or without additives.
- The cement generally used are.
  - Ordinary Portland IS 269:1989 IS 8112:1989 IS 12269:1987
  - Portland Slag IS 455:1989
  - Sulphate Resisting Portland IS6909:1990
  - Supersusulphated Cement IS 6909:1990
  - Portland Pozzolana IS 1489 (Part 1&2) 1991

Other solid materials may be used as additives to the grout mixture are:

- Pozzolanas such as flyash (IS 3812:1981) and calcined shale (IS 1344:1981).
- As early strength is important on most grouting jobs, the pozzolana may behave only as inert non-cementing fillers.
- Fine sands (IS 383:1970) are economical additives widely used in grouting.

✓ **Grout Mixture- Admixture**

Use in small quantities the following admixtures impart certain desirable characteristics:

- Retards to delay setting time.
- Accelerator to hasten setting time.
- Lubricants for increasing workability.
- Protective colloids to minimize segregation.

Expansion materials to minimize shrinkage.

✓ **Grout Mixture - <u>Grout Mix Proportion</u>**

For determining the mix proportions the viscosity& bleeding of grout, strength and economy shall be the main consideration:

- **Mix proportion should be exercised according to the following guidelines (IS-6066:1994)**
- **The choice of grouting mixtures is based on results of percolation tests conducted prior to grouting.**
  - **Ranging from 5:1 to 0.8:1 are recommended.**
- **The grouting should be continued till refusal stage is achieved i.e the rate of intake becomes almost negligible say 1.0 litre/min averaged over a period of 10 minutes at desired limiting pressure up to 3 kg/ cm2 and 1.5 lirte/min for pressure between 3 and 10 kg/cm2.**
- **It is desirable to carry out the grouting with the mixture of cement and water only. However if the intake is more, the grout may be thickened by using inert materials like.**
- **Pozzolan as or fine sand, rock powder, clay bentonite etc.**

✓ **Grout Mixture- <u>Grout mix for Multiple Line Grout Curtains</u>**

- **In case of multiple line grout curtain differs from single line curtains.**
- **In the outer line thick grouts may be used to prevent over travel and to block the more pervious zones.**
- **In the central lines grout may be thickened very gradually and comparatively thinner grout may be used at the start.**
- **Thickening of grouts may be carried out more gradually in tertiary holes as compared to primary and secondary holes.**
- **Thickening of grouts may be carried out more gradually in tertiary holes as compared to primary and secondary holes.**
- **In order to prevent over travel of grout in wide joints, sodium silicate or sodium hexa meta phosphate is sometimes added.**
- **For increasing the flowability in case of thin joints, 2 to 3% bentonite is added.**

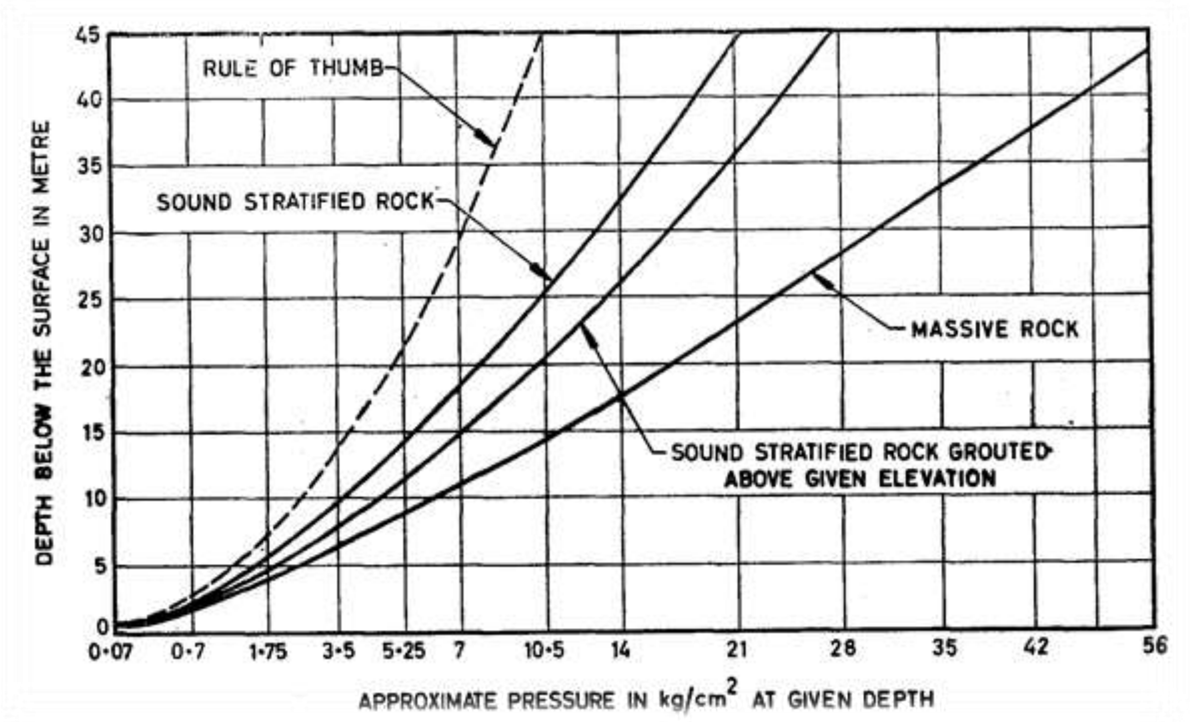
✓ **Grout Pressure- <u>Control of Grout Pressure</u>**

Control of pressure should be exercised according to the following guidelines (IS-6066:1994):

- **Control of grouting pressure is very important to avoid excessive pressure and resulting damages. The pressure shall be built up gradually from a less value to desired value.**
- **The limiting value of pressure for each zone and depth may be established initially from the results of trial grouting, along with observations of upheaval. Fig-2 may be used as guide.**
- **Pressure limits may be decided by analysis of the results of percolation test.**

- The grout pressure should be such that it should travel through the cavities to the maximum possible distance at the same time, it shall not be so excessive that it may cause upheaval.
- Review the Acceptance criterion.

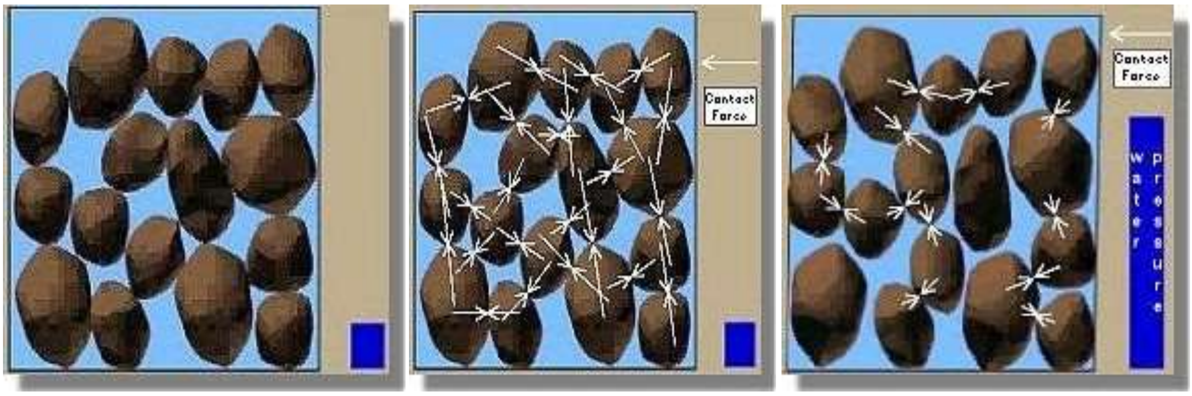
  Consider chemical grouting in special cases



Liquefaction and Testing

1. Soil liquefaction

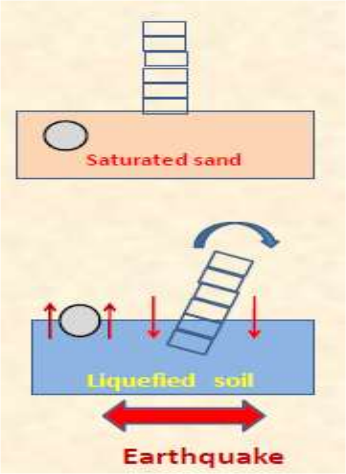
Soil liquefaction describes a phenomenon whereby a saturated or partially saturated soil substantially loses strength and stiffness in response to an applied stress, usually earthquake shaking or other sudden change in stress condition, causing it to ***behave like a liquid***.

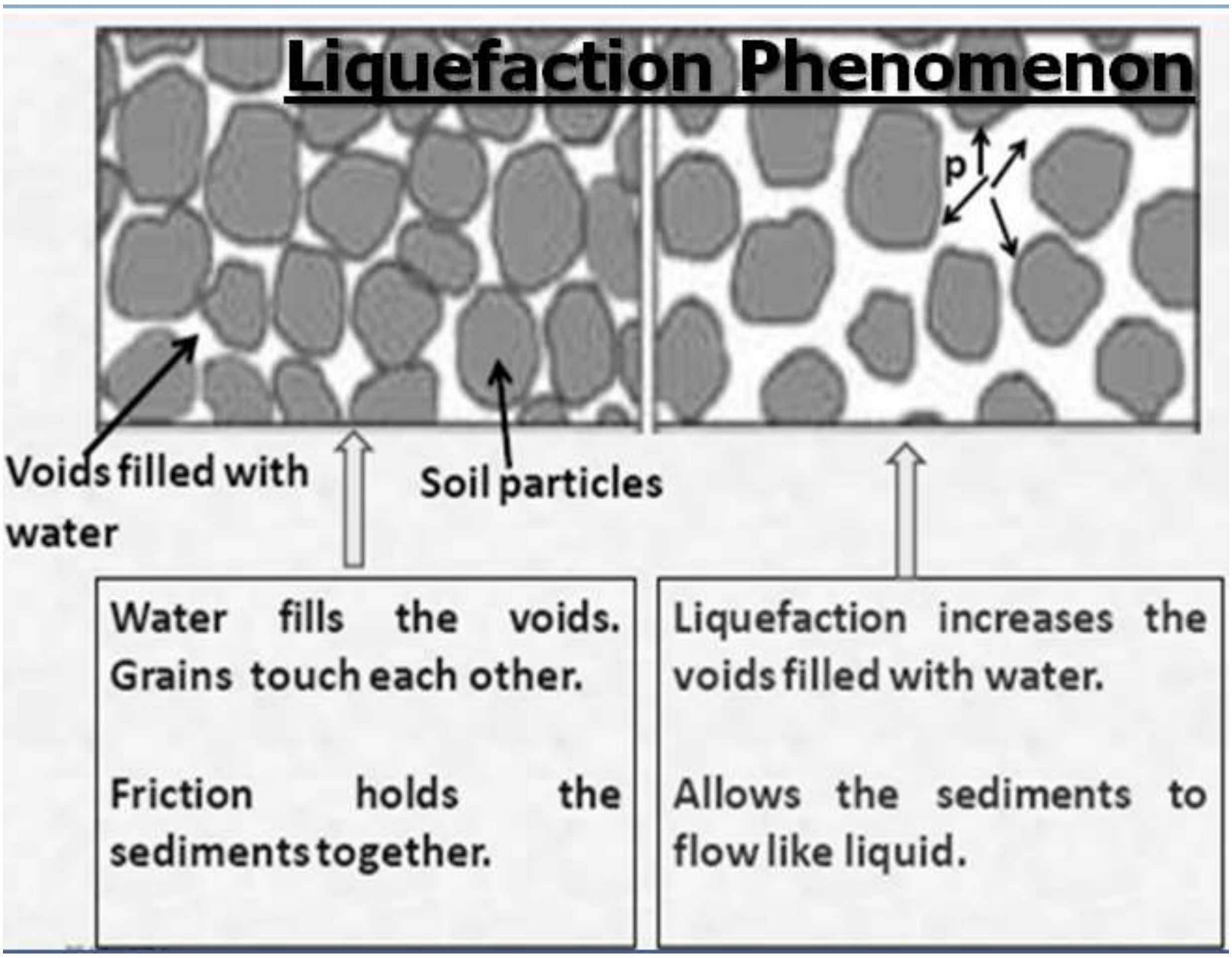


2. Damages due to Liquefaction

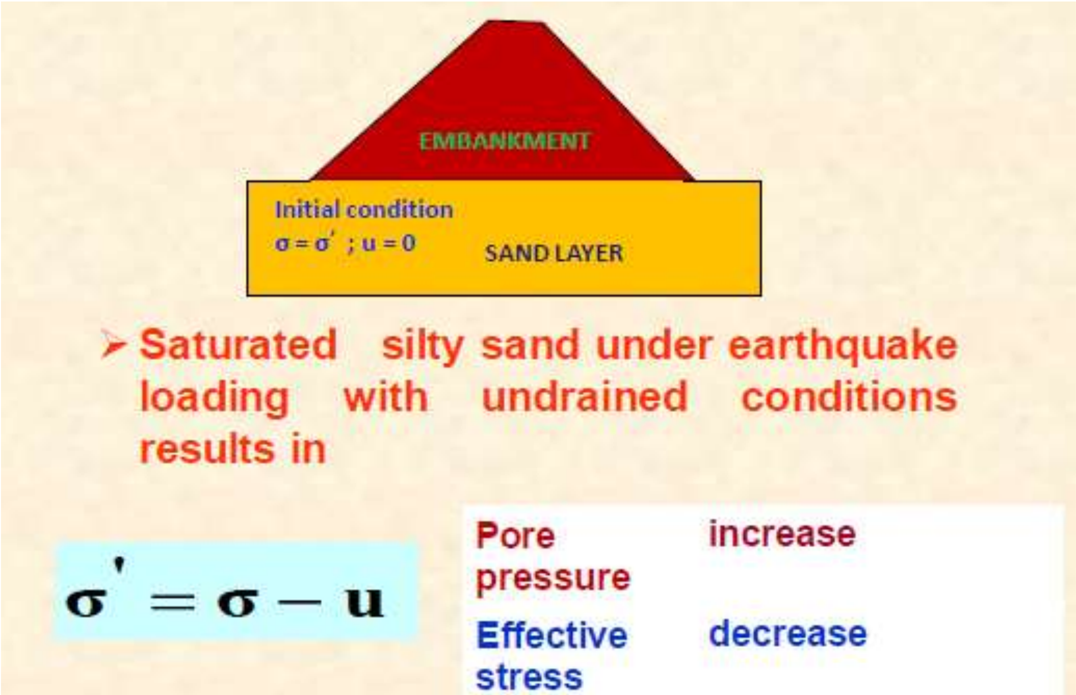
3. Liquefaction Potential of Foundation

- It is a phenomenon in which a Saturated Silty Sand strata in foundation losses its shear strength during an Earthquake.
- In liquefaction, the soil behaves like viscous fluid.
- It is due to an increase in pore water pressure in soil.
- It causes structures to sink and tilt, buried pipelines to float.





4. **Total and Effective stress**



5. **Steps in Liquefaction Analysis**

- Determination of Site specific design Earthquake
- Determination of Dynamic Elastic Properties (G,D)
- Evaluation of Shear Stresses likely to be induced in foundation soil from Dynamic Response Analysis by FEM
- Determination of Cyclic Strength of Soil against liquefaction.
- Evaluation of Liquefaction depth of Foundation by Comparison of induced shear stresses with cyclic strength.

6. **Tests used in Liquefaction studies**

   a. **Laboratory tests**

- Cyclic Triaxial Test

- Simple Shear Test

- Resonant Column Test

b. IN SITU TESTS

- Standard Penetration Test

- Dynamic Cone penetration Test

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- Static Cone Penetration Test

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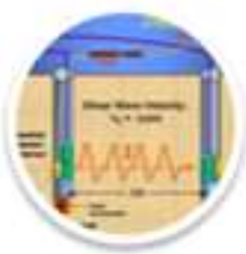


- Geophysical Cross Hole Test

7. Types of liquefaction

- Flow liquefaction

✓ Occurs when shear stress required for equilibrium of a soil mass (the static shear stress) is greater than the shear strength (residual strength) of the soil in its liquefied state.

✓ Potentially very large post-liquefaction lateral deformations are driven by the static shear stress.

- Cyclic mobility

✓ Occurs when the static shear stress is less than the shear strength of the liquefied soil.

✓ Deformations are driven by both cyclic and static shear stresses.

✓ Deformations develop incrementally during earthquake shaking

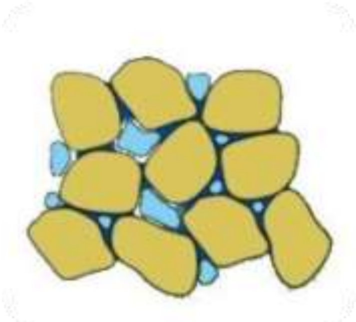
8. Why does liquefaction occur

✓ **If** the soil is loose and is being shaken, the particles will settle due to gravity.

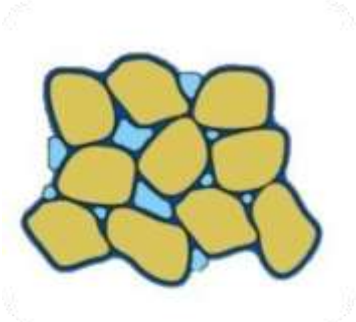
✓ When the soil is saturated, the pore-water is unable to move out of the way quickly enough (because the soil permeability is relatively low), and more and more particles start to partially float in the water (this leads to excess pore- pressure build up).

✓ Eventually as shaking continues, the particles float in the water temporarily as they settle downwards and reach a new densified and consolidated state.

9. Soils Susceptible to liquefaction



Most susceptible would be very loose cohesion less soils. The low permeability of non-plastic silts and sands is a disadvantage



Higher permeability, higher relative density, and higher cohesion (plasticity) reduce the susceptibility.

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10. Notes

Objectionable deformations might still occur if r_u values are high, even if liquefaction does not occur). Looser soils are more vulnerable.

As pore pressure builds-up, stratified soil profiles (particularly with permeability contrasts) may cause water to be temporarily trapped under a relatively impervious layer or seam (e.g., due to alluvial or hydraulic fill construction, or presence of an upper clay stratum), generating a low friction interface and possibly leading to major lateral deformations. This mechanism actually is a driver of what we commonly observe as sand boils where this water escapes upwards through any available high permeability locale (e.g., taking advantage of a crack in the ground, or similar imperfection, …).

11. Evaluation of Liquefaction Potential and Consequences

I. Is the soil susceptible to liquefaction?

II. If the soil is susceptible, will liquefaction be triggered?

1) Cyclic stress approach (will be further discussed below)

2) Other methods: effective-stress response analysis approach,
cyclic strain approach, energy dissipation approach, probabilistic approach.

III. If liquefaction is triggered, how much damage would occur?

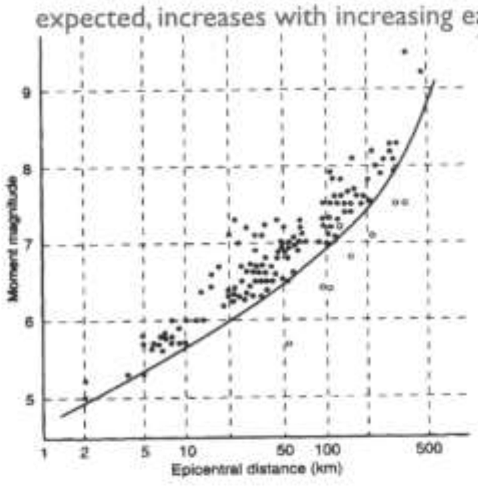
- Settlements
- Lateral deformations due to cyclic mobility: a) empirical approach, and effective-stress response analysis approach

Flow Failure (see Kramer 1996).

12. Is the soil susceptible to liquefaction?

I. Historical criteria

The epicentral distance to which liquefaction can be expected, increases with increasing earthquake magnitude.



From Kramer (1996)

Figure 1. Relationship between limiting epicentral distance of sites at which liquefaction has been observed and moment magnitude for shallow earthquakes. Deep earthquakes (focal depths > 50 km) have produced liquefaction at greater distances. After Ambraseys (1988).

Geologic criteria

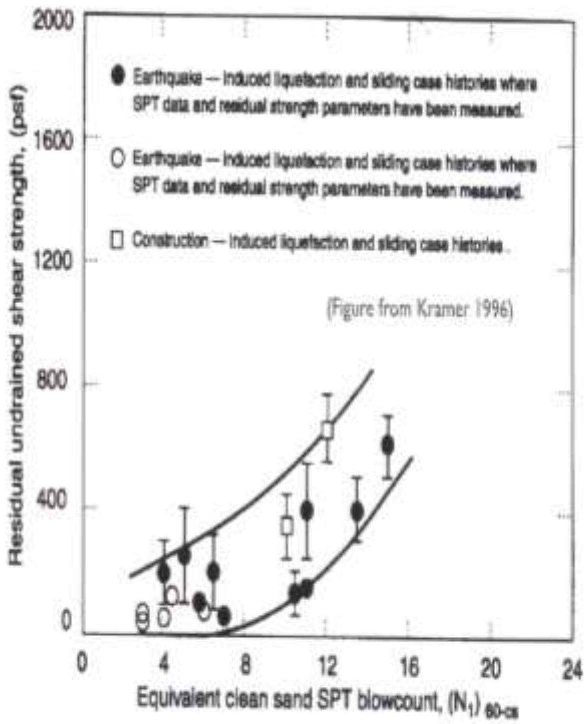
- Depositional environment - Saturated loose fluvial, colluvial, and aeolian deposits are more susceptible to liquefaction.
- Age - Newer soils are more susceptible to liquefaction than older soils.
- Water table - Liquefaction susceptibility decreases with increasing groundwater depth.
- Human-made soil strata - Uncompacted soils (e.g., hydraulic fill) are more susceptible to liquefaction than compacted soils.

Compositional criteria

- Grain size and plasticity characteristics - Sands, non-plastic silts, and gravelly soils, under conditions of low permeability, are susceptible to liquefaction.
- Gradation - Well graded soils are less susceptible to liquefaction than poorly graded soils.
- Particle shape - Soils with rounded particles are more susceptible to liquefaction than soils with angular particles

Initial stress state criteria (for flow liquefaction)

A loose soil will be susceptible to flow liquefaction only if the (Figure from Kramer 1996) static shear stress exceeds its steady state (or residual) strength.Residual strength can be estimated as shown in Figure.



13. Liquefaction Analysis using SPT data : Problem Statement

The measured SPT resistance and results of sieve analysis for a site in Zone IV are given in Table 1.1. Determine the extent to which liquefaction is expected for a 7.5 magnitude earthquake. The site is level, the total unit weight of the soil layers is 18.5 kN/m3, the embankment height is 10 m and the water table is at the ground surface. Estimate the liquefaction potential immediately downstream of the toe of the embankment.

Table 1.1: Result of the Standard penetration Test and Sieve Analysis

Depth (m)	N_{60}	Soil Classification	Percentage fine
0.75	9	Poorly Graded Sand and Silty Sand (SP-SM)	11
3.75	17	Poorly Graded Sand and Silty Sand (SP-SM)	16
6.75	13	Poorly Graded Sand and Silty Sand (SP-SM)	12
9.75	18	Poorly Graded Sand and Silty Sand (SP-SM)	8
12.75	17	Poorly Graded Sand and Silty Sand (SP-SM)	8
15.75	15	Poorly Graded Sand and Silty Sand (SP-SM)	7
18.75	26	Poorly Graded Sand and Silty Sand (SP-SM)	6

14. Liquefaction Analysis using SPT data :Solution

Site Characterization:

This site consists of loose to dense poorly graded sand to silty sand **(SP-SM).**

The **SPT values** ranges from **9 to 26**

The site is located in **zone IV.**

The peak horizontal ground acceleration value for the site will be taken as **0.24g** corresponding to zone factor **Z = 0.24**

a. Liquefaction Analysis using SPT data :Solution

Liquefaction Potential of Underlying Soil

Step by step calculation for the depth of 12.75m is given below.

$$a_{max} = Z \times I \times S$$

$$a_{max} = 0.24 \times 1 \times 1 = 0.24$$

$$M_w = 7.5, \ \gamma_{sat} = 18.5 \ kN/m^3,$$

$$\gamma_w = 9.8 \ kN/m^3$$

Considering water table at ground surface, sample calculations for 12.75m depth are as follows.

Initial stresses:

$$\sigma_v = 12.75 \times 18.5 = 235.9\ kPa$$

$$u_0 = (12.75 - 0.00) \times 9.8 = 124.95\ kPa$$

$$\sigma_v' = (\sigma_v - u_0) = 235.9 - 124.95$$

$$= 110.95\ kPa$$

Stress reduction factor:

$$r_d = 1.174 - 0.0267z = 1.174 - 0.0267 \times 12.75 = 0.83$$

Critical stress ratio induced by earthquake:

$$a_{max} = 0.24g,\ M_w = 7.5$$

$$CSR = 0.65 \times (a_{maz} / g) \times r_d \times (\sigma_v / \sigma_v^{'})$$

$$CSR = 0.65 \times (0.24) \times 0.83 \times (235.9 / 110.95)$$

$$= 0.28$$

Correction for SPT (N) value for overburden pressure:

$$(N_1)_{60} = C_N \times N_{60}$$

$$C_N = 9.79 (1 / \sigma_v^{'})^{1/2}$$

$$C_N = 9.79 (1 / 110.95)^{1/2} = 0.93$$

$$(N_1)_{60} = 0.93 \times 17 = 16$$

Critical stress ratio induced by earthquake:

For $(N_1)_{60} = 16$, fines content of 8%

$CRR_{7.5} = 0.22$ (Figure A-5)

Correction for SPT (N) value for overburden pressure:

$$CRR = CRR_{7.5}\ k_m\ k_\alpha\ k_\sigma$$

K_m = Correction factor for earthquake magnitude other than 7.5 (Figure A-1)

$= 1.00$ for $M_w = 7.5$

K_α = Correction factor for initial driving static shear (Figure A-3)

$= 1.00$, since no initial static shear

K_σ = Correction factor for stress level larger than 96 kPa (Figure A-2) $= 0.88$

Detailed calculations for all the depths are given in Table

Depth	%Fine	σ_v (kPa)	σ_v' (kPa)	N_{60}	C_N	$(N)_{60}$	r_d	CSR	$CRR_{7.5}$	CRR	FS
0.75	11.00	13.9	6.5	9.00	2.00	18	0.99	0.33	0.24	0.27	0.82
3.75	16.00	69.4	32.6	17.00	1.71	29	0.97	0.32	0.32	0.34	1.04
6.75	12.00	124.9	58.7	13.00	1.28	17	0.95	0.31	0.21	0.20	0.65
9.75	8.00	180.4	84.8	18.00	1.06	19	0.91	0.30	0.23	0.21	0.69
12.75	8.00	235.9	110.9	17.00	0.93	16	0.83	0.28	0.22	0.19	0.70
15.75	7.00	291.4	137.0	15.00	0.84	13	0.75	0.25	0.16	0.13	0.53
18.75	6.00	346.9	163.1	26.00	0.77	20	0.67	0.22	0.22	0.18	0.80

***This table provides the factor of safety against liquefaction (FS), maximum depth of liquefaction below the ground surface.**

Factor of safety against liquefaction:

$$FS = CRR / CSR = 0.19 / 0.28 = 0.70$$

It shows that the considered strata is liable to liquefy.

III. If liquefaction is triggered, how much lateral deformation occurs?

Residual Strength (see Fig. 2). In addition, for the residual shear strength S_r, Olson and Stark (2002) proposed:

$S_r/\ '_{vo} = 0.03 + 0.0075\ (N_1)_{60}$ plus or minus 0.03 for $(N_1)_{60}$ less or equal to 12

and

$S_r/\ '_{vo} = 0.03 + 0.0143\ (q_{c1})$ plus or minus 0.03 for q_{c1} less than or equal to 6.5 Mpa

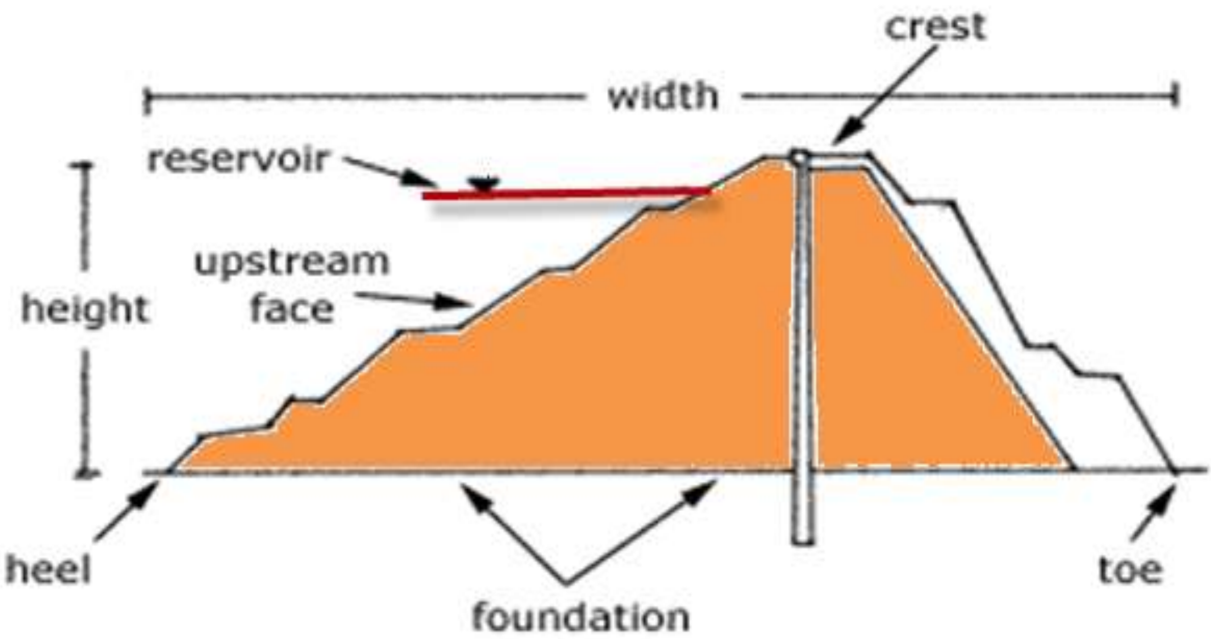
Earlier, Baziar and Dobry (1995) proposed for loose silty sands:

$S_r = 0.12 - 0.19\ (\ '_{vo})$

Seepage through Body of the Dam and Slope Protection

1. What is a Dam?

A dam is a barrier built across a stream, river or estuary to hold and control the flow of water for such uses as drinking water supplies, irrigation, flood control and hydropower generation etc.



2. Most Failures are Seepage/Piping Related

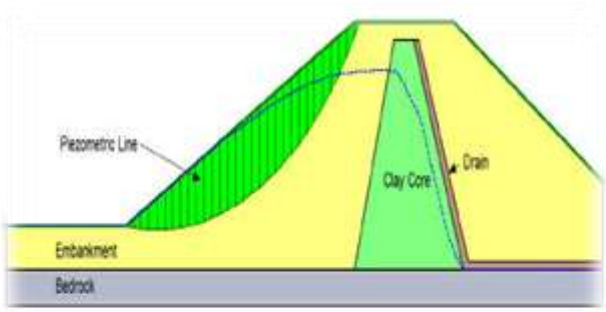
- Teton Dam, Idaho
- Started as seepage through cracks in the rock of the right abutment
- Dam had no filter-drainage zone

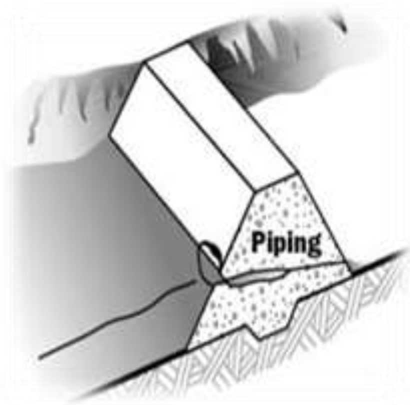
- New dam – first filling
- Failed with small amount of water stored in the reservoir
- Foundation and embankment cracks suspected
- Dispersive clay soils were involved

3. Most Failures are Seepage/Piping Related

Seepage through dam body creates the following two problems, apart from causing excessive water loss

- Seepage Force in the form of Pore Water Pressures
- Piping





4. Seepage Control & Drainage Features

- Seepage control features are categorized as either Water Barriers or Controlled Drainage Features.
- A Combination of these two types of features usually incorporated in every embankment dam design.

- It is a Convectional practice to provide Impervious cores within the body of embankment dam as a water barrier.
- In rockfill dams, instead of providing impervious core, an impervious membrane on the upstream face is adopted.

The convectional types of seepage control and drainage features generally adopted for the Embankment Dam are:

- Impervious Core,
- Inclined/Vertical filter with Horizontal filter,
- Network of inner Longitudinal and cross drains,
- Horizontal filter,
- Transition zones/transition filters,
- Intermediate filters,
- Rock toe, and
- Toe drain.

The selection of drainage features depends upon:

- The availability of Filter Materials,
- Types of dam
- Whether Earthfill / Rockfill or Whether zoned / Homogeneous
- Type of Shell Materials, etc.

5. Criteria for Selection of Drainage Features

- The most common drainage features adopted in earth dam are inclined filter, horizontal filter, rock toe and toe drain.
- In case of a Homogeneous section of earth dam either only horizontal filter or a combination of vertical/inclined filter and horizontal filter shall be provided.
- For rockfill dam with central impervious core upstream inclined filter shall be provided.
- When the shell material is relatively impervious intermediate filters may be provided in the upstream and down stream shell.
- Toe drain located on the D/S toe of the embankment dam is necessary to collect seepage. The collected water in the toe drain should be taken away from the toe of the dam by providing outfalls at suitable locations along the toe drain.
- In Rockfill dams, transition zones/transition filters may be required between the impervious core and rockfill and between the overburden in dam seat and rockfill.
- Where a single single zone of graded filter is costly, progressive zoning consisting of one or more transition zones may be considered as an alternative.

Seepage Analysis

1. **Significance of Seepage Analysis in Dam Safety**
    - Seepage is simply reservoir water finding its way downstream through pervious material or through imperfections in the dam or its foundation.
    - Stored water represents stored energy that is continually seeking release downstream.
    - The force or pressure behind the seeping water can create new or enlarge existing seepage pathways until the dam is breached.
    - Thus, the control of seepage is extremely important in the design, construction, and safe operation of dams.

2. **Seepage through Dam/Embankment**

    - The passage of water through embankment, foundation, or abutment material.
    - Water may flow through the pores of soils used to construct the embankment or in the foundation.
    - Water may flow through cracks in the soil or along the contact of soil with concrete or metal appurtenances.
    - Water can also flow through features in the bedrock.

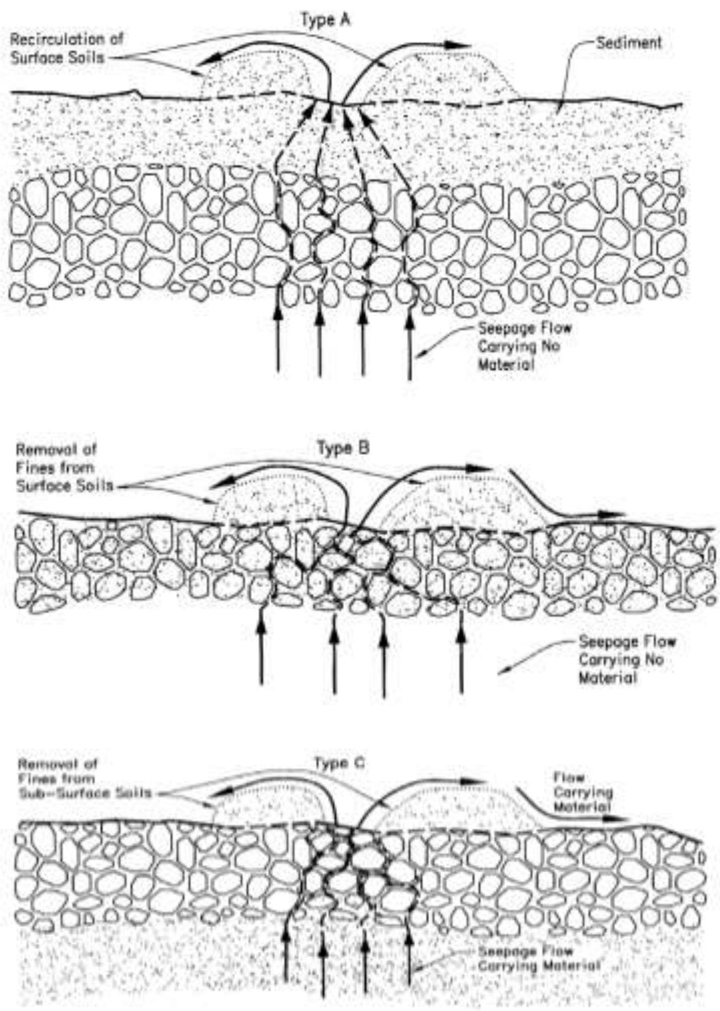
3. **Modes of Seepage Failure Conditions**

    - Seepage causing excessive uplift, heave, or blowout
    - Seepage causing solutioning of soluble rock
    - Seepage causing piping
    - Seepage causing internal erosion
    - Seepage causing excessive internal pressures
    - Saturation causing sloughing or failure of slopes

4. **Seepage causing excessive uplift, heave, or blowout**
    - Foundation seepage pressure in pervious layers can exert significant uplift force on a confining layer of lower permeability soil downstream from a dam.
    - Failure begins when the pore pressure on the bottom of the confining layer exceeds the overburden pressure created by the weight of the overlying soils.
    - The resulting uplift eventually breaches or breaks through the confining layer in what is known as a BLOWOUT.
    - When upward flow of seepage water is strong enough to carry sand particles, the sand is deposited around the springs in a conical ring, referred to as a SAND BOIL.

- If a sand boil continuously removes material due to an excessive hydraulic gradient, they may eventually lead to piping, collapse or failure of the structure.
- Piezometers can be used to monitor downstream foundation uplift pressure and detect unsafe conditions before failure occurs.
- A key indicator of a potential problem developing is fine soil carried in water draining from a boil.

  - ✓ Type A is indicative of a static condition for the current hydraulic gradient and is not necessarily indicative of an immediate problem developing.
  - ✓ Type B is a boil that is carrying material, but the material is originating from near surface soils rather than deeper zones.
  - ✓ Type C is indicative of a critical condition, where the present hydraulic gradient is removing subsurface soils.



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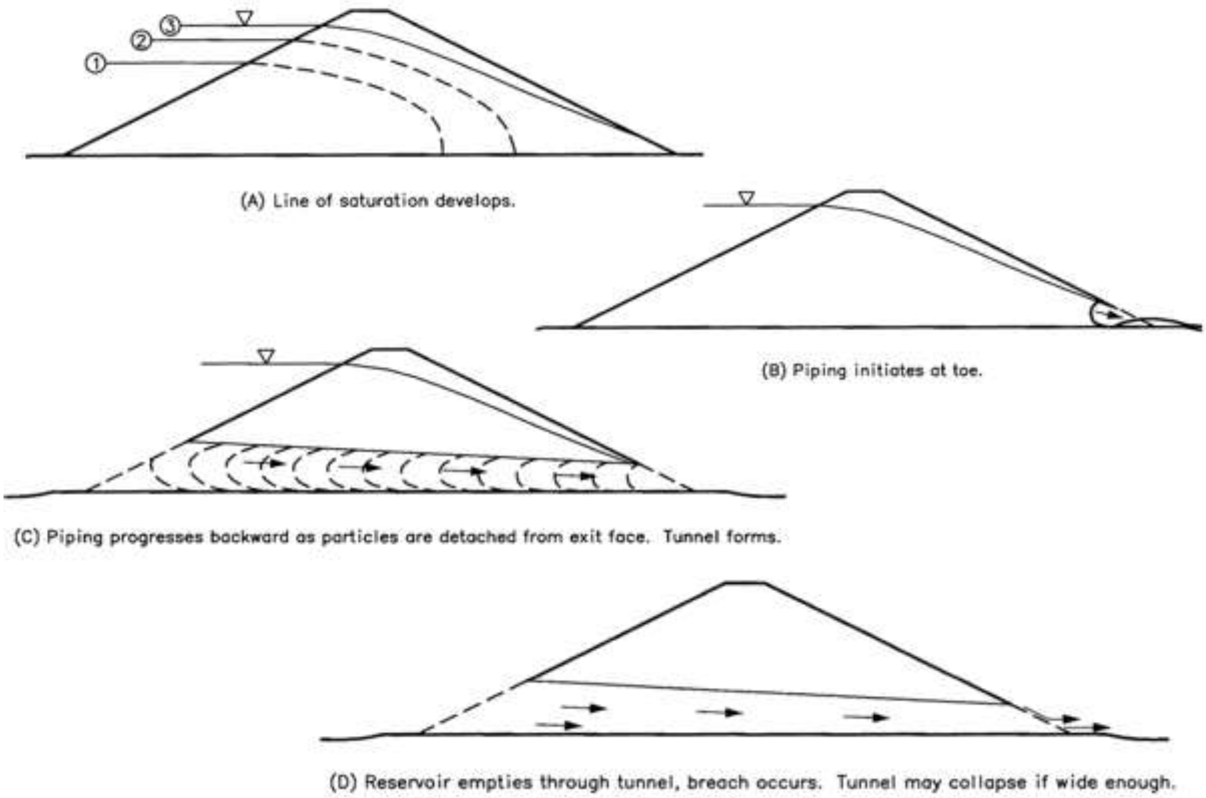
5. **Seepage causing Piping**
   - Piping occurs when reservoir water moving through the pores of the soil (i.e., seepage) exerts a tractive force on the soil particles through which it is flowing, sufficient to remove them at an unprotected exit point of the seepage.
   - The initial physical expression of piping is often a cone shaped mound of soil called a boil or a stream of muddy water exiting the slope.
   - The removal of soil may progress upstream forming a characteristic open tube or “pipe” through the dam, from which the phenomenon derives its name

6. **Five Conditions must exist for piping to occur**
    - There must be a flow path/source of water.
    - The hydraulic gradient must exceed a certain threshold value that is dependent on the type of soil through which the flow path travels.
    - There must be an unprotected exit (open, unfiltered) from which material can escape.
    - Soils that are susceptible to piping must occur within the flow path near the discharge point of the seepage.
    - The material being piped or the soil directly above it must be able to form and support a “roof” to keep the pipe open.

7. **Seepage causing Piping**



(A) Line of saturation develops.

(B) Piping initiates at toe.

(C) Piping progresses backward as particles are detached from exit face. Tunnel forms.

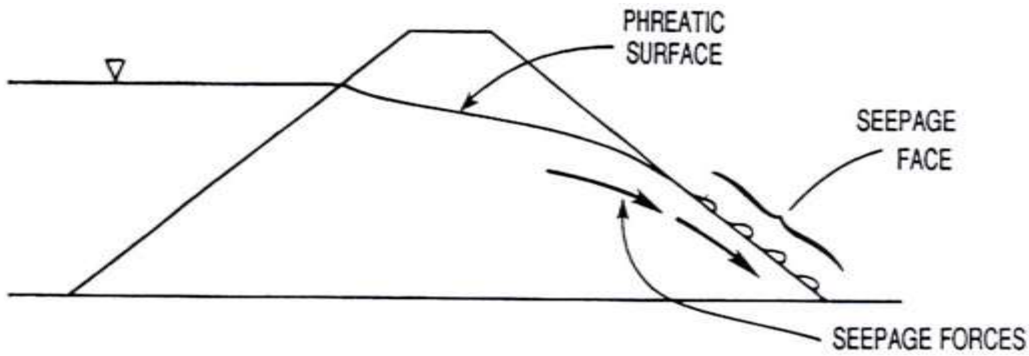
(D) Reservoir empties through tunnel, breach occurs. Tunnel may collapse if wide enough.

8. **Piping happens most commonly when**
    - Seepage occurs through soil layers that are susceptible to piping and seepage reduction methods are not used to reduce the hydraulic gradient that causes piping, or
    - Filters and pressure relief measures are not used at seepage discharge points to prevent the particle movement of susceptible soils, or
    - Seepage reduction measures are not properly maintained.
    - Soils most susceptible to piping are loose, poorly graded fine sands.

- Highly susceptible are silts and sands with low-plasticity fines (Plasticity Index less than 6), as well as loose, well graded sand and gravel mixtures that are very broadly graded and have low-plasticity fines.
- Clay soils with significant plasticity (Plasticity Index greater than 15) are less susceptible to piping.

9. Seepage causing excessive Internal Pressures

- A failure resulting from internal erosion may appear similar to a piping failure. In the case of piping, the tractive forces result from the intergranular flow of water between soil particles. Internal erosion, occurs when water flows:
- Along cracks or other defects in the soil or bedrock in the C/S.
- Along boundaries between soil and bedrock.
- Between soil and concrete or metal appurtenances.



10. To evaluate seepage, required Project data are:

- The geology of the dam site.
- How the dam was designed and constructed.
- The materials used to construct the dam.
- The seepage control measures incorporated into the dam and reservoir.
- How seepage could affect the project.
- The physical features of the dam.
- The instrumentation to monitor seepage pressures and quantity.
- **It is necessary to review project data to gain an understanding of the physical features and performance history of the dam, and to identify any known or potential design, Construction, or operating deficiencies.**

11. Types of documentation for a dam will include:

- Results of field and laboratory investigations.
- Design analyses and reports, and construction plans and specifications.
- Construction reports, logs, records, photographs, and as-built drawings.
- Past inspection reports.

- Operation and maintenance records.
- Monitoring instrumentation records.
- Any special reports prepared for the project.

12. Field and Laboratory Investigations

- Regional and site geology, including engineering characteristics of foundation rock and soil
- Geologic features of the dam foundation, abutments, and reservoir rim.
- Relationship of the geologic features to the components of the dam.
- Adequacy of the data as it pertains to evaluation of the specific problem being addressed.
    - **Evaluation of geologic data should be performed by a qualified individual, who understands how various earth and rock materials behave under the loading conditions imposed upon them by the construction of a dam and reservoir.**
    - **For each geologic setting, there are known types of defects.**

13. Design Analyses and Construction Plans and Specifications

- What assumptions regarding seepage were made and what problems were anticipated in the original design or Subsequent modifications.
- What methods of seepage control were incorporated into the dam and foundation design, and how they were to be constructed.
- How these seepage control methods were originally designed to work and if more recent information is available, does it require a re-evaluation of the seepage control design.
- Whether any of these seepage control and construction methods are outdated.

14. Review of details reports & logs to determine :

- How the foundation was prepared and treated.
- If grouting was done and if there were any zones of large grout takes.
- If the original design intent is still consistent with latest information about existing field conditions.
- If design changes were made to accommodate field conditions.
- If the proper materials and gradations were utilized in all embankment zones.
- What construction methods were used to prevent contamination of the filter or drainage zones.
- If any major problems were encountered during construction and, if so, how they were resolved.
- How seepage control methods were installed and if problems were encountered during installation.
- If as-built drawings are accurate.

15. Important indicators of seepage-related problems

- A progressive increase in volume of flow.
- Evidence of piping (sand boils), internal erosion, solutioning of solids, or increased turbidity of seepage.
- Increase/decrease in hydrostatic pressures.
- A changing pattern of seepage.
- Seepage appearing at a critical location, such as adjacent to a conduit.
- A progressive increase in volume of flow.
- Evidence of slope instability as a result of seepage (sloughing).
- Appearance of sinkholes.
- Soft, unstable areas downstream.
- Unusual vegetative growth (green grass in an arid environment).

16. Area of field-Investigations

- How does the volume of seepage vary with seasons, rainfall, and fluctuating reservoir levels?
  - ✓ Does the seepage increase with increasing reservoir levels?
  - ✓ Does this increase occur suddenly or over a long period of time?
  - ✓ As the reservoir level recedes, does the seepage volume decrease to the previous amount or to some higher volume?
  - ✓ Does seepage exit the ground at different points?
  - ✓ Do seepage measurements have an annual cycle?
  - ✓ Do seepage measurements rapidly track rainfall, or track a seasonal rainfall pattern?
- How do seepage pressures or seepage forces respond to various reservoir levels?
  - ✓ Is the response instantaneous?
  - ✓ Do the seepage pressures or forces respond at an increasing rate with increasing reservoir levels?
- Visual Evidence From Onsite Observation
  - ✓ Put in perspective, clarify, and focus the information obtained from the project records.
  - ✓ Provide a mental picture for reference in conducting future work.
  - ✓ Help in interpreting and calibrating what others have reported.
  - ✓ In most cases, provide a "feel" for what the problem is and how serious it is.

17. INTERVIEWS

- How the seepage area differs or reacts with changing conditions as compared with the rest of the project.
- If seepage occurs at certain reservoir levels and how quickly it appears.

- What happens in the area following rain, both with and without a reservoir level increase.
- Whether the seepage water is ever turbid.
- Whether the seepage water carries particles.
- Whether accumulations of particles occur in the area of seepage.
- Whether seepage changes with climatic conditions.
- Whether seepage always exits at the same location.
- If the seepage changes on a cyclic annual, seasonal or daily pattern.
- If seepage is related to operation of the gates or other control structures.
- If the area was wet before the dam was built.

18. Field - Investigations :- Instrumentations

- For new or developing seepage problems, additional instrumentation may beneeded to supplement existing instruments or provide coveragein areas where there are no instruments.
    - Vibrating wire, pneumatic, or hydraulic piezometers
    - Downhole flow meters
    - Thermal probes
    - Downhole cameras
    - Observation wells
    - Weirs
- Some general guidelines for instrumentation selection and location are as follows:
    - A reasonable array of piezometers is usually needed to trace the path and the real extent of seepage from the source to the exit point as well as the pressures generated by the seepage. This arrangement is particularly necessary in rock where seepage flows through the joints, fractures, faults, and shear zones rather than through the intact rock.
    - The location of instruments should be coordinated with the need for field investigation. To save time and money, a hole drilled for a piezometer can also provide geologic information, sampling, and a location for in situ testing.

19. Basis For Analysis

- Many of history's dam failures resulted from the lack of a consistent and logical framework for analyzing and anticipating seepage problems.
- Empirical rules based on observed good and bad performance, while of some help, were often not applicable to slightly different materials, foundations, and other circumstances.
- The logical analysis of seepage started with the development of Darcy's law in 1856, and the realization that the Laplace equation governing heat and current flow was also applicable to the steady-state flow of an incompressible fluid through a porous media.

20. Darcy's Law

- Henri Darcy, a formula governing flow through porous media, now known as Darcy's law, was based on the study of water flow through vertical filters in laboratory experiments.The experiments indicated that the quantity of flow is expressed by the equation:

  Q = **kiA** & **V*d* = Q/A**

  Q = rate of seepage (cm3/sec)

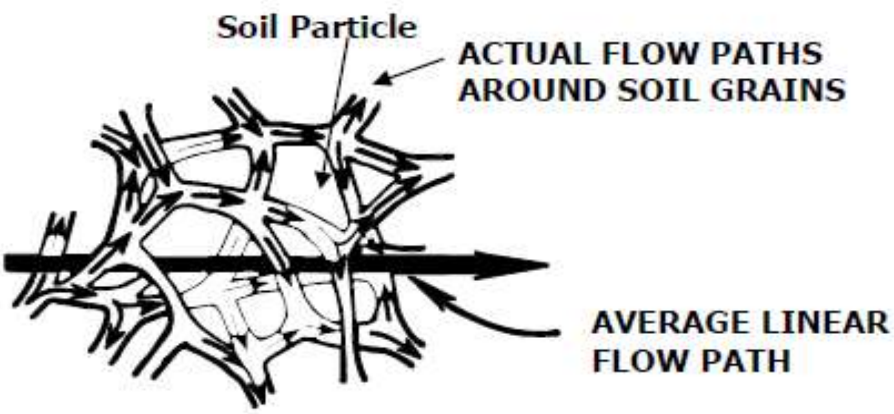
  V***d*** = discharge velocity, also equal to ki (cm/sec)

  k = Darcy's coefficient of permeability (cm/sec)

  i = hydraulic gradient equals the head loss (cm) divided by the length over which head loss occurs (cm)

  A = C/S area normal to the direction of flow (cm2)

- The discharge velocity Vd is an average fluid velocity and is defined as the gross quantity of fluid that flows through a unit cross-sectional area of soil in a unit of time.
- Since flow only occurs through the interconnected soil voids, the real velocity of flow or seepage velocity (Vs) for a single molecule of water traveling a unique path in the soil voids is greater than the discharge velocity.
- The seepage velocity is roughly equal to discharge velocity divided by the porosity of the soil.

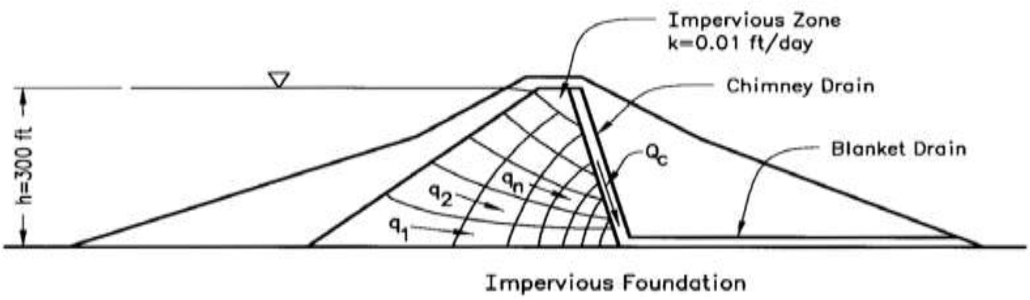


- Darcy's law is applicable only to laminar flow (adjacent flow lines are parallel and straight and Vd is directly proportional to i).

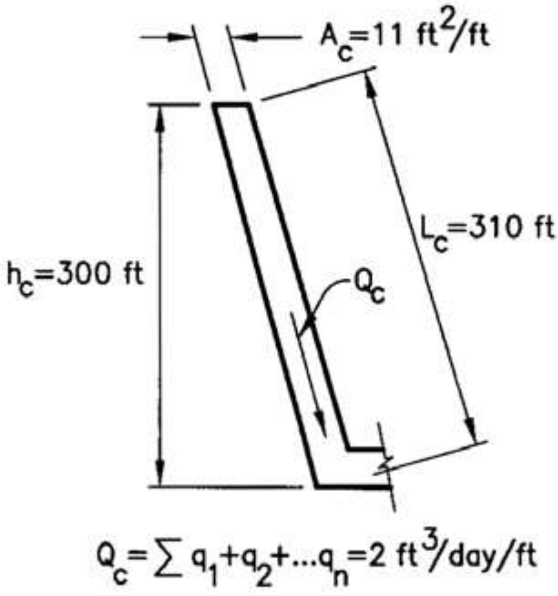
- This law is reasonable for most soils, but flow through coarse gravels and rock openings may become turbulent and Vd is proportional to approximately the square root of i.
- Darcy's law is limited to flow through saturated materials. Flow through unsaturated materials is in a transient state and is time dependent.
- Darcy’s law is not useful in studying flow through cracks or fractures and similar features in rocks or soil.

Darcy's law has many applications in seepage analysis, including:

- Determining permeability, both in the field and laboratory.
- Predicting quantity of laminar flow.
- With approximate modifications, Darcy's law can be applied to problems of turbulent, transient (time-dependent), and partially saturated flow.
- Darcy's law is also used in solving many seepage and drainage problems associated with dams.



A common example is determining the necessary permeability or dimensions of inclined or horizontal drains in a dam



Chimney Drain

21. LAPLACE EQUATION

- The flow of water through a porous medium (soil) is only one of several forms of streamline flow that obey similar fundamental relationships, which can be represented by the Laplace equation.
- In two dimensions, the Laplace equation can be solved by drawing two families of curves that intersect at right angles to form a pattern of "square" figures, commonly known as a flownet.
- The Laplace equation for three-dimensional flow is:

$$\frac{\delta^2 h}{\delta x^2} + \frac{\delta^2 h}{\delta y^2} + \frac{\delta^2 h}{\delta z^2} = 0$$

- Developing the Laplace equation for flow of water through porous media requires the following assumptions:
  - ✓ The porous media (soil) is homogeneous.
  - ✓ The voids are completely filled with water (i.e.,saturated).
  - ✓ The soil and water are incompressible
  - ✓ Flow is laminar and Darcy's law is valid.

22. FRACTURE FLOW

- Darcy permeability is not directly applicable for studying water flow through discrete open fractures, joints, and other types of cracks in rock and soil.
- Evaluating fracture flow is complex because the flow is dependent on fracture geometry, fracture roughness, fracture fill material, and the size of fracture openings. Hence, the problem may require extensive field data to solve.
- Simplifications are often used, including simplifying the problem so that Darcy's law may be indirectly used to solve the problem by using a bulk hydraulic conductivity for a highly fractured rock mass.
- Flow through fractures in soil may lead to internal erosion.
- Evaluating the potential for internal erosion is often empirical because suitable mathematical or other models are not available and due to the problem of characterizing the fracture characteristics.
- Evaluation often considers only whether the appropriate measures were designed and constructed properly on the assumption that internal erosion is likely to be a problem.
- Fracture flow can be the dominant mode of seepage through rock foundations and abutments.
- It is also a primary mode of fluid transport common to internal erosion. Darcy's law does not strictly apply to flow through an open fracture, as it was derived from experiments on flow through a column of homogeneous sand.
- However, both Darcy and the Laplace equations may

- be applied for approximating flow through a uniformly fractured rock mass if the volume of rock under consideration is uniformly fractured and can be assumed isotropic.
- The methods used to solve the Laplace equation & the Darcy permeability used in the Darcy equation are prone to scale effects when considering fracture flow.
- Fracture flow varies from highly anisotropic to a relatively isotropic phenomenon, depending on the size or scale of the rock volume under consideration and the spacing of interconnected fractures.
- Fracture flow can be approximated as flow through two parallel plates.
- Experiments with flow through parallel plates led to the development of an equation for determining the hydraulic conductivity (kf) of a fracture.

$$k_f = \frac{\rho g a^2}{12 f \mu}$$

where a is the size of the fracture aperture and μ is the fluid viscosity, f is a fracture roughness factor that accounts for friction, ρ is the density of the fluid and g is the acceleration of gravity.

The quantity of flow through a fracture (Q) is dependent on the hydraulic gradient, the hydraulic conductivity of the fracture, and the cross sectional area perpendicular to flow and may be expressed as:

$$Q = vA$$

The validity and quality of any seepage analysis depend upon the information available for input into the analysis.

- The location of various boundaries and flow paths.
- The type of flow.
- The permeability of the various materials through which the seepage flows.

Seepage problems are common because information available during the design and construction phase of a dam is often insufficient for accurately predicting seepage.

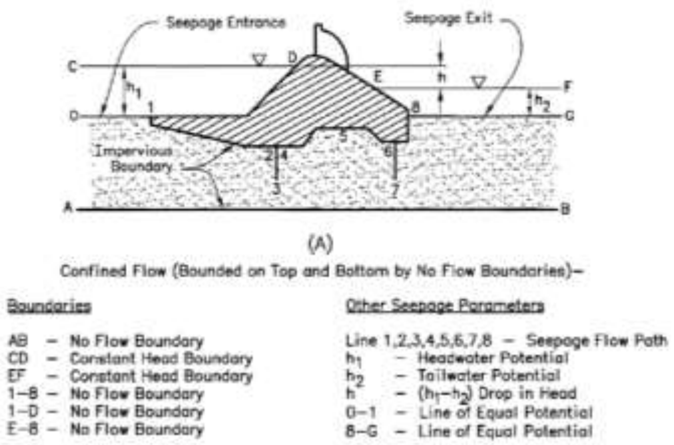
Therefore, post-construction field observations are required and can supply much additional information needed to analyze and rectify any seepage problems once they develop.

23. BOUNDARY CONDITIONS

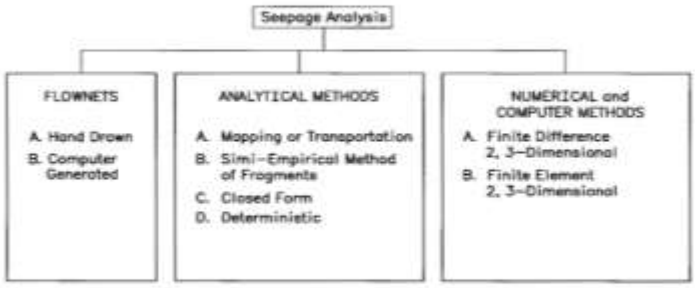
- Boundaries define the limits and conditions of flow in the cross-section being analyzed.
- Boundaries include an impervious layer in the foundation below which no seepage is expected

- to occur, the entrance face for seepage, and the exit face. One must also define whether boundaries are fixed or transient.
- The nature and location of various boundaries are determined by:
- • Field exploration and knowledge of site geology.
- • Assumptions based on engineering judgment.
- • Conditions imposed by design and type of structure.
- • Geometry of the dam and its zoning.
- In most cases, simplifying assumptions are required to establish boundaries that will permit an analysis.

24. Several Types of Boundaries

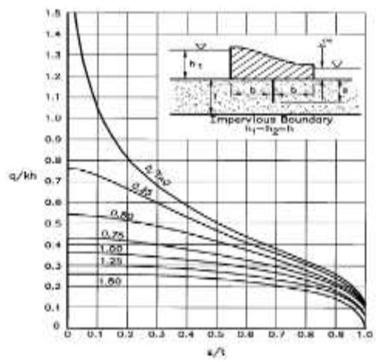


25. COMMONLY USED SEEPAGE ANALYSIS



26. FlowNets

IMPERVIOUS STRUCTURE WITH PARTIAL CUTOFF ON LAYER OF FINITE DEPTH



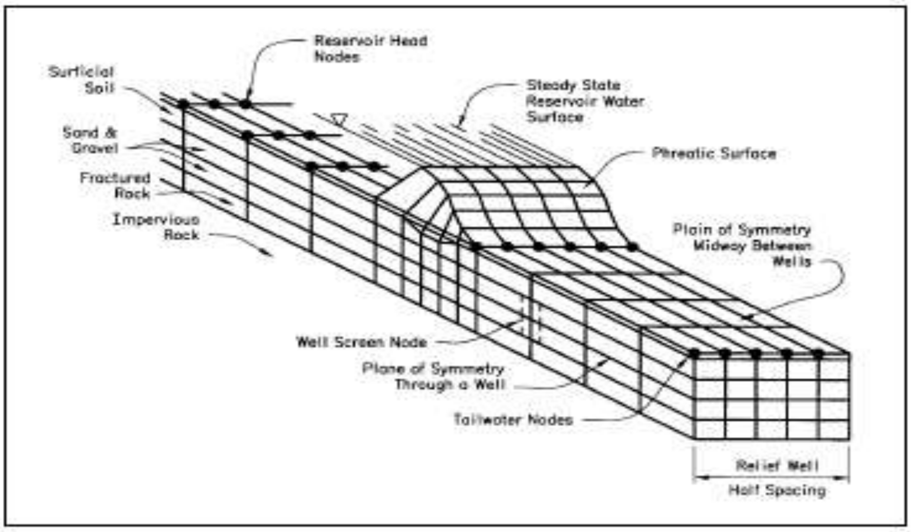
27. NUMERICAL COMPUTER SOLUTIONS

- Computer models are used increasingly to make acceptable approximations for the Laplace equation in complex flow conditions.
-
- The two primary methods of numerical solution are finite difference and finite element.
- Both can be used for two-dimensional and three-dimensional problems and software is available from several sources.
- Very simple problems can be solved by hand, but more difficult problems require a computer.
- Both methods use a grid system to divide the flow region into discrete elements.
- Element intersections are called nodes.
- In either system, a series of linear algebraic equations are used to approximate the Laplace equation.
- In the finite element method, if the grid consists of N elements, there will be N equations in N unknowns to solve.

28. Advantages of Numerical Methods

- Either two-dimensional or three-dimensional problems of very complex geometry, including layers and stratification as well as pockets of material, can be modeled.
- Zones where seepage gradients or velocities are high can be more accurately modeled by varying the size of elements.
- No transformation of dimensions or properties is necessary.
- Results are printed in digital form for easy plotting of flownets.
- Various programs have options and capability for computing seepage forces and handling transient or time-dependent flow and variable saturations.

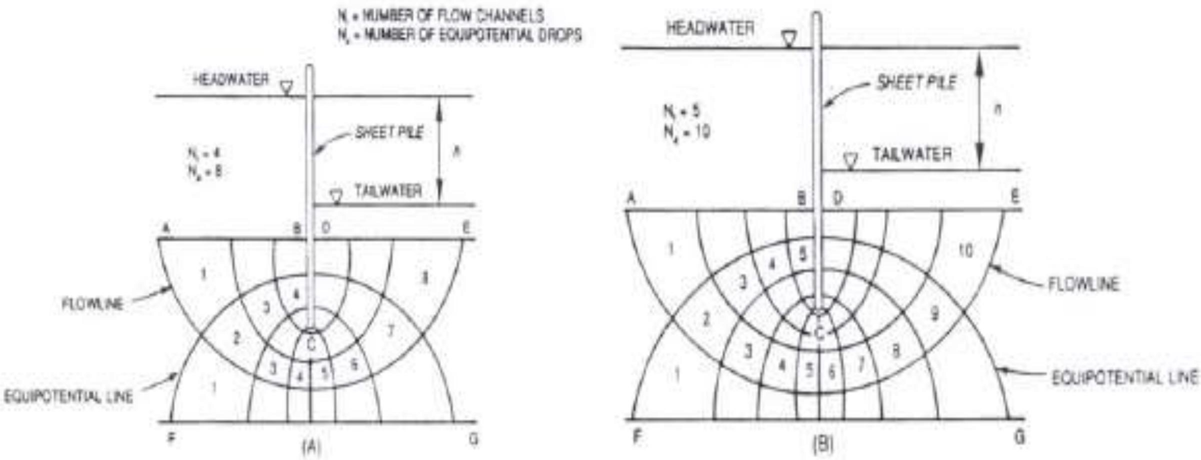
29. Finite Element Relief well model



30. FlowNets

To draw a flownet, several basic properties of the seepage problem must be known or assumed:

- The geometry (zoning) of the porous media must be known.
- The boundary conditions must be determined.
- The assumptions required to develop Laplace's equation must hold.
- Anisotropic permeability must be considered.



31. WHERE AND WHEN TO USE METHODS OF ANALYSIS

Some general considerations when selecting a method of analysis include:

- What point in a dam's history is being considered?
- How complex is the problem?
- What information is available?
- What information can be obtained or is required, and at what cost?
- Is the problem urgent, or is there time for detailed analyses?

32. General Guidelines for Seepage Analysis

SITUATIONS	TYPICAL INVESTIGATIONS	SUGGESTED ANALYSIS METHODS
Homogeneous embankment, impervious foundation, 2D steady state	Phreatic surface, pore pressure, seepage force (stability)	Graphical (Casagrande) or flownets
Zoned embankment, impervious foundation, 2D steady state	Phreatic surface, pore pressure, seepage force (stability)	Flownet or numerical model
Homogeneous embankment, uniform pervious foundation, 2D steady state	Phreatic surface, pore pressure, seepage force (stability)	Flownet
	Exit gradient, seepage quantity	Method of fragments (see Harr's Mechanics of Particulate Media (Appendix B))
	Seepage control alternatives, material properties variations	Numerical model
Zoned embankment, pervious foundation, 2D steady state	Same as above	Numerical model

SITUATIONS	TYPICAL INVESTIGATIONS	SUGGESTED METHODS
Situations involving relief wells, heterogeneous foundation, quasi-3D steady state	Phreatic surface, pore pressure reduction, exit gradient, seepage quantity, seepage control alternative, material properties variation, relief well spacing, and flow	Numerical model
Situations involving relief wells, uniform foundation, quasi-3D steady state	Relief well spacing, pressure reduction, and flow	Equations (see Design, Construction and Maintenance of Relief Wells, U.S. Army Corps of Engineers, EM 1110-2-1914 (Appendix B))
Uniform pervious abutment, 3D steady state	Phreatic surface, seepage quantity	Plan flownet
Heterogeneous pervious foundation and abutments, 3D steady state	Phreatic surface, seepage quantity, exit gradient, materials, and control alternatives	Numerical model
2D transient flow, steady boundary conditions	Tracking saturation, time to steady state	Transient flownets
Situations involving nonsteady 2D flow, saturated/unsaturated, zone or homogeneous embankment, heterogeneous foundation, transient boundary conditions, 2D transient state	First fill, flood cycle, cyclic operation, moisture content and pore pressure changes, precipitation and evaporation effects	Numerical model (See Introduction to Groundwater Modeling, Wang, Herbert F., Anderson, Mary P.)

COMPOSITE DAMS AND INTERFACE DETAILS

1. Composite Dams and Interface Details

- Made with Two or more types of materials having different behaviour especially load transfer.
  - Rock-fill/Earth dam with Gravity dam as spillway.
  - Gravity dam with Rock-fill/Earth fill as Backing material to strengthen it.
- Rock-fill/Earth dam with Gravity dam as spillway.
  - When width of River is Large
  - Design Discharge is Less
  - Height is more and in High Seismic Area

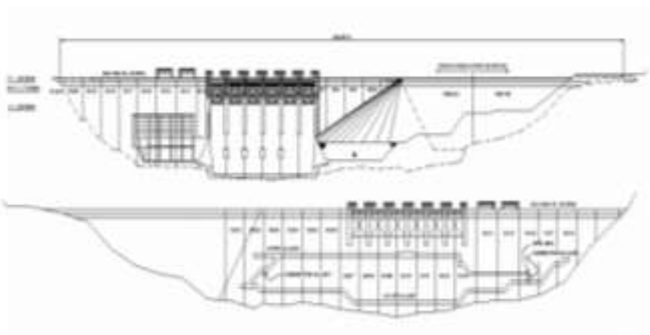
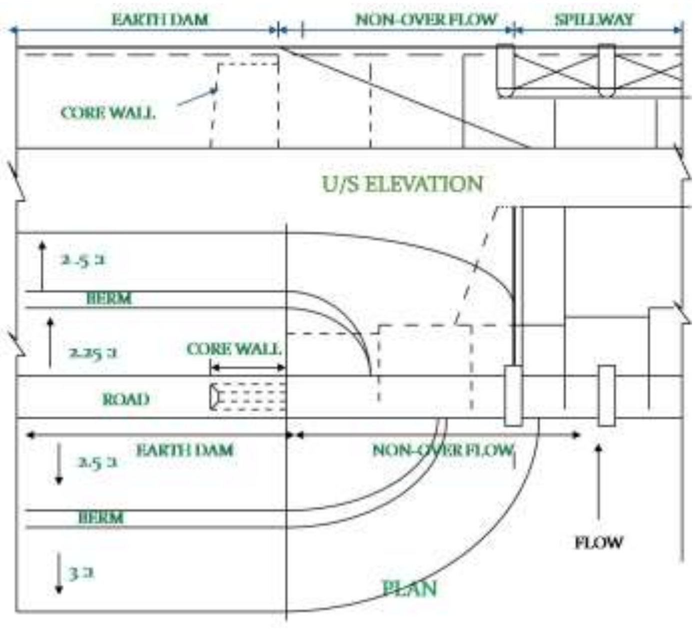
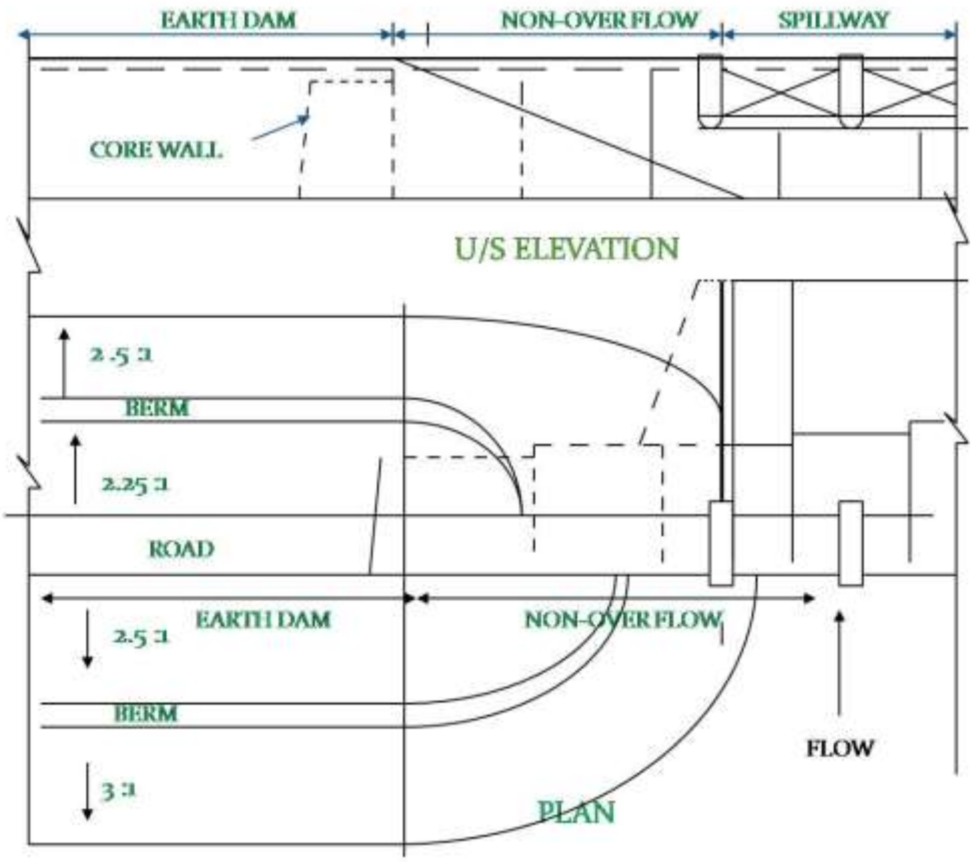


Figure 3. Plane and Longitudinal Section of Composite Dam

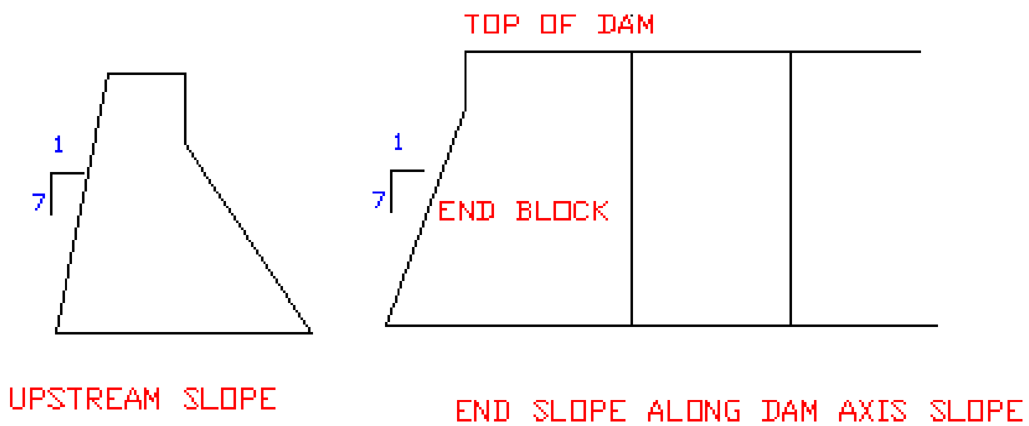
2. Junction Of Embankment Dam With Gravity Dam : CONVENTIONAL DESIGN



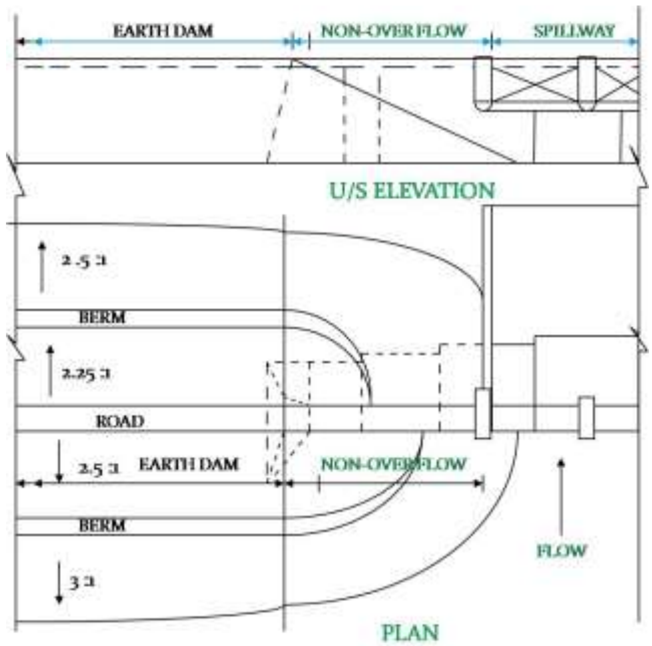
3. **Junction Of Embankment Dam With Gravity Dam**



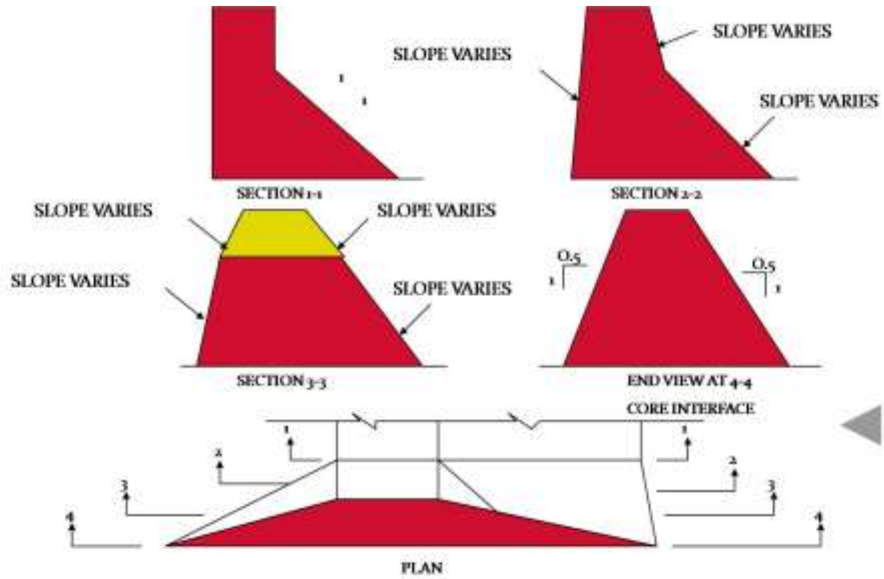
4. **Junction Of Embankment Dam With Gravity Dam – Elevation**



5. **Junction Of Embankment Dam With Gravity Dam**



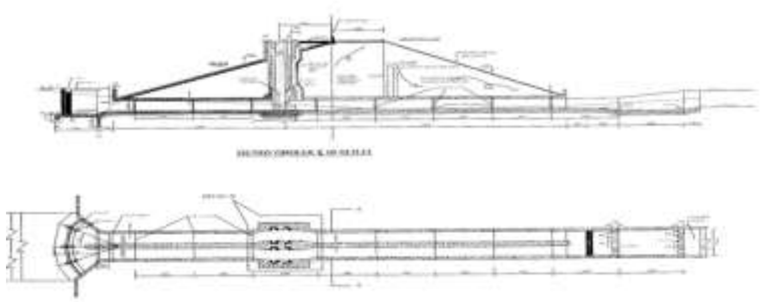
6. **Junction Of Embankment Dam With Gravity Dam - Details Of Junction Block Details Of Junction Block**



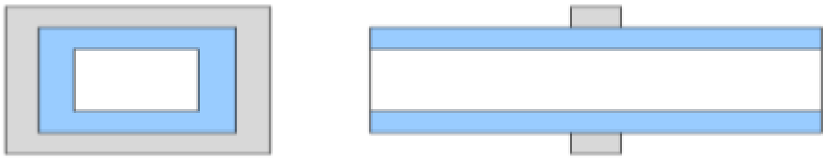
7. **Contact with Spillway**

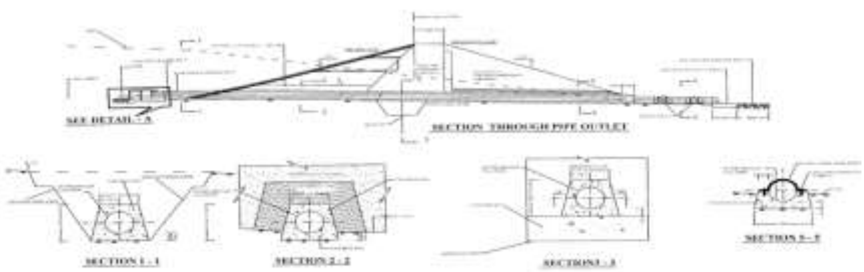
- Adequate length of seepage (core wall or key wall) should be provided
- Foundation treatment similar to that of the Spillway should be provided
- Foundation treatments of embankment and concrete dam should provide a continuous impervious barrier
- Good bond between earth work and foundation.
- Protection of earth slope against scouring.

8. OUTLET INTERFACE

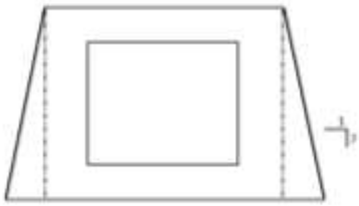


9. EARLIER INTERFACE FOR OUTLET





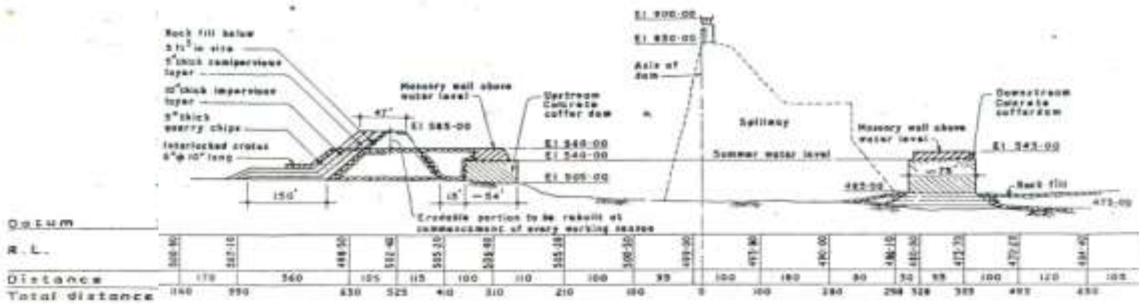
10. Latest Interface For Outlet



11. Gravity Dam with Rock-fill backing

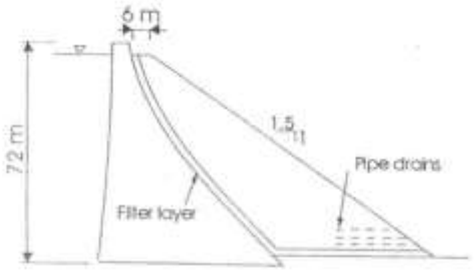
- Strengthening existing Gravity dams having inadequate stability to resist Hydrostatic force and Seismic force.
- Economic alternate to conventional construction to reduce concrete volume.
- As an impervious core

12. Layout of Coffer Dam for Srisailam Project

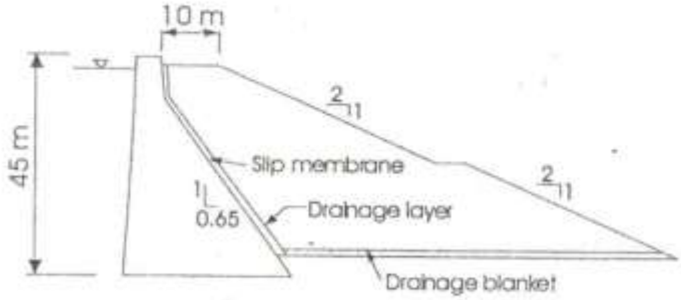


13. Concrete Coffer Dam with earthfill backing

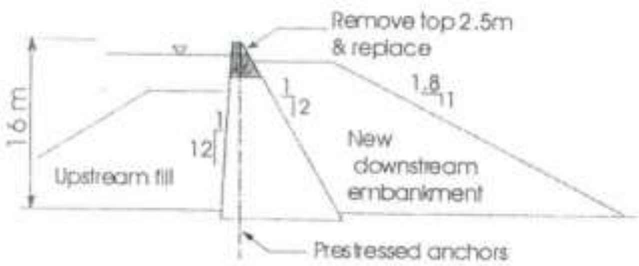
14. Dams with Earth backing



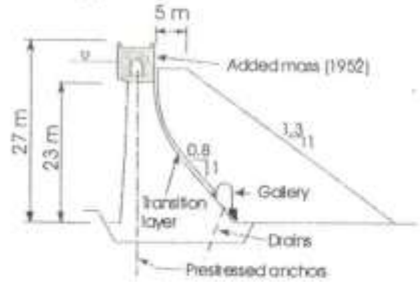
Avon



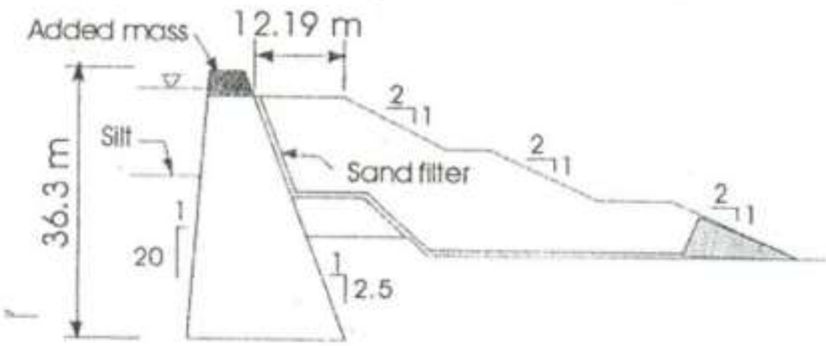
Upper Glendevon



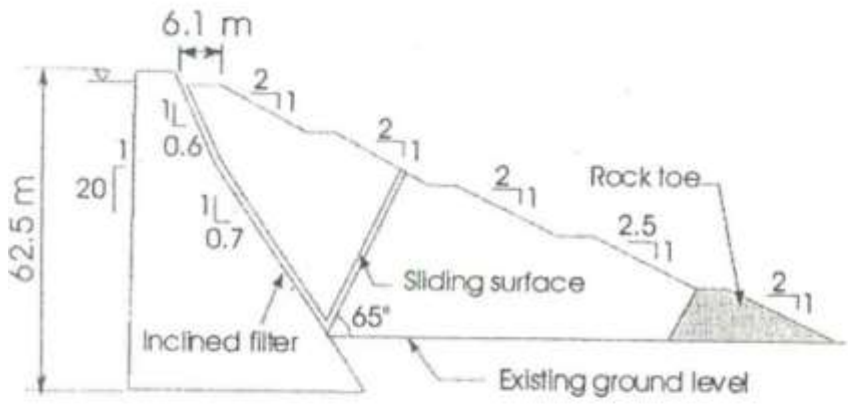
Matabitchuan



Joux



Talakalale



Kahadakwasala

15. LOADS & MAIN PROPERTIES

- Wt of concrete & Rock-fill
- Lateral thrust by Rock-fill
- Hydrostatic force
- Uplift
- Angle of Concrete Rock-fill friction
- Angle of internal friction of Rock-fill.
- Cohesion between concrete and foundation
- Concrete foundation friction angle

16. ANALYSIS

- Minimum Earth Pressure method
- To ensure safety of Strengthened dam for Normal and MWL condition
- Force calculated by Rankine equilibrium in Rock-fill
- Maximum Earth Pressure Method
- Reservoir empty condition
- To check for overturning and sliding in upstream direction
- Internal friction in the fill $\varnothing = o$ check

17. Sliding safety factor (SFF)

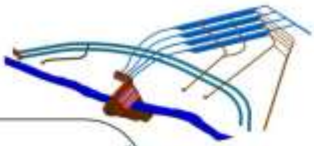
SFF = $[(W_c-U+R_v)(\tan\varnothing_c)+(CA)] / P-R_H$

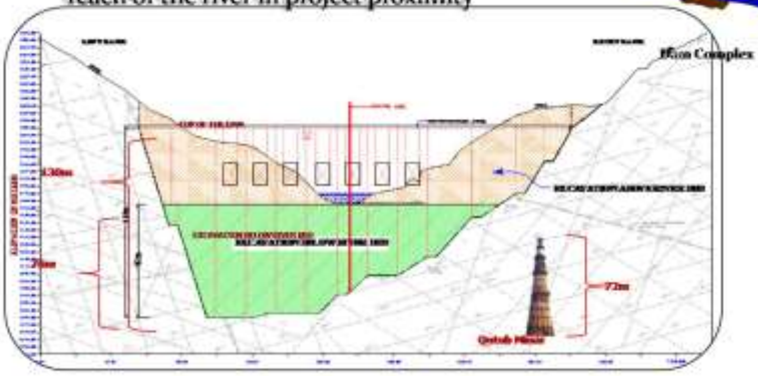
- W_c = Wt of concrete
- U = uplift
- $\varnothing_c$ =concrete-foundation friction angle
- C = cohesion concrete-foundation
- A =Area of contact where cohesion can develop
- P = Hydrostatic force
- R_v =Vertical force of Rock-fill

R_H =Horizontal force of Rock-fill

18. Problem appreciation :- Punatsangchhu-I H.E. Project (1200 MW), Bhutan

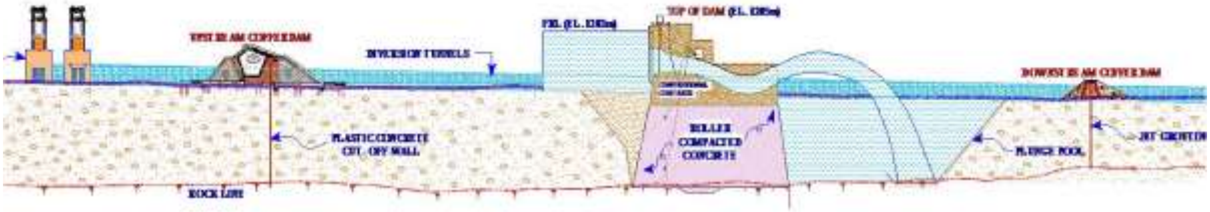
- Foundation grade rock **70m** within the entire reach of the river in project proximity





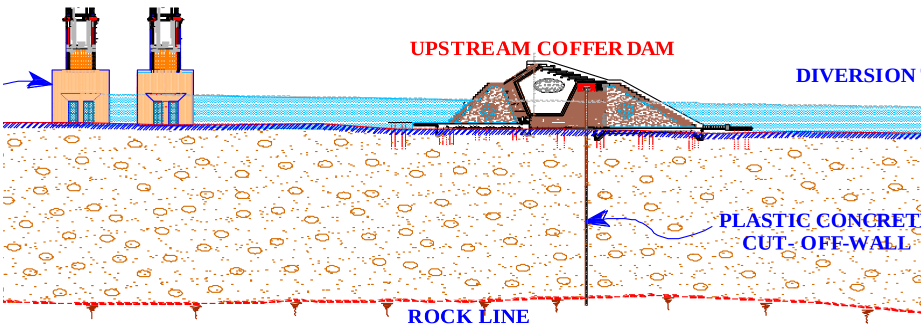
Dam – Upstream view

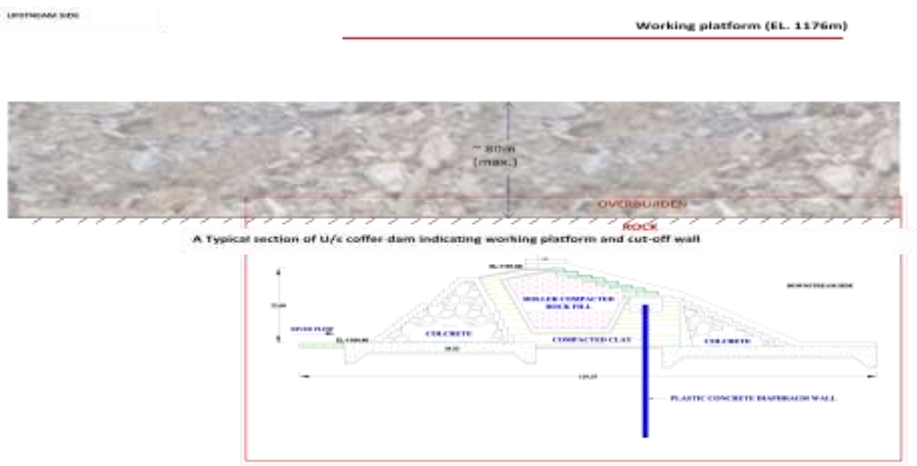
- **Seepage free Dam pit a pre-requisite**



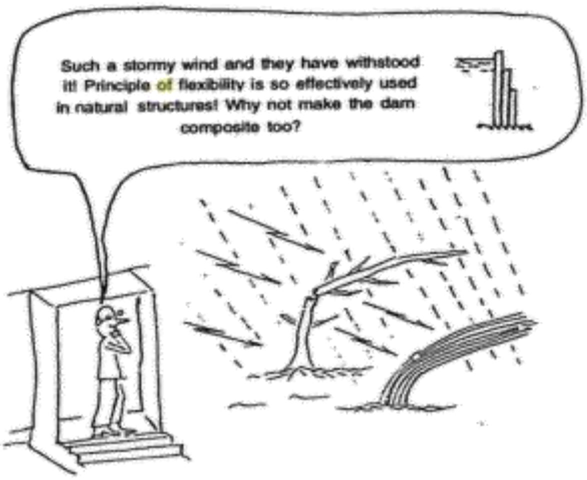
A view of deep seated rock and huge overburden in the vicinity of Dam

- Large quantity of excavation below river bed (**20 lakh m³** approx.)





A Typical section of U/s coffer dam indicating working platform and cut-off wall



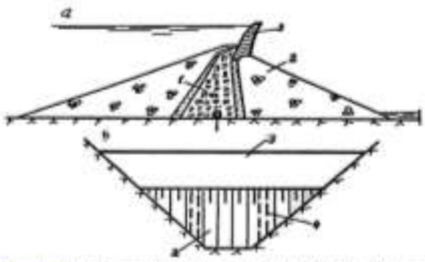


Fig. IV.59: Combined dam: a) cross-section; b) view from tailrace. 1—Core; 2—Dam using earthen materials; 3—Arch dam; 4—Inspection shaft.

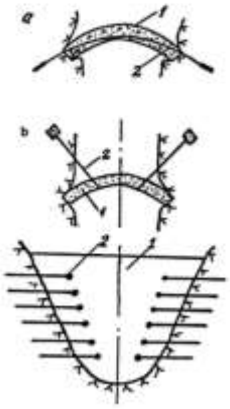


Fig. IV.48: Arch dam with prestressed anchors in arch direction: a)—along entire length of arch (plan-section); b)—in end sections at an angle to the arch (a.c. No. 325926, plan section and view from headrace). 1—Dam; 2—Anchor.

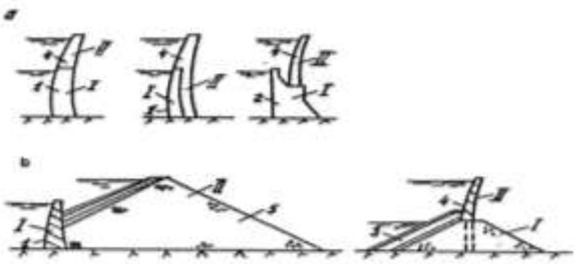
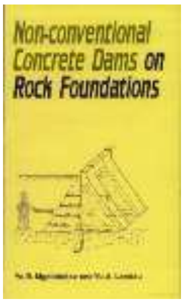
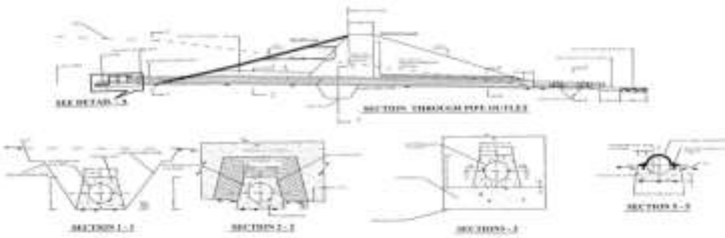


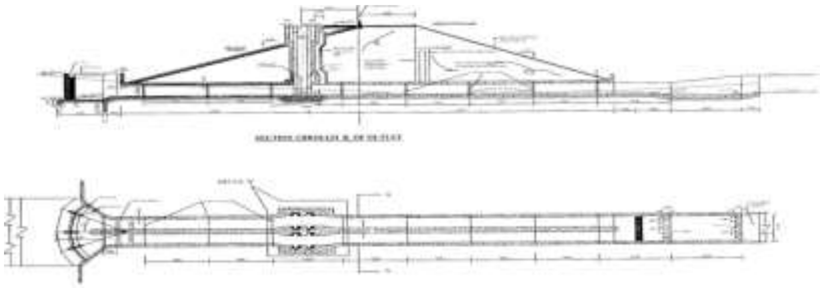
Fig. IV.65: Erection of arch dams in stages: a—with concrete dam of second stage in contact with the crest or downstream face of stage I dam; b—with formation of combined dam. 1—Arch dam of stage I; 2—Gravity dam of stage I; 3—Earth dam of stage I; 4—Arch dam of stage II; 5—Earth dam of stage II.



DESIGN OF OUTLET FOR EMBANKMENT DAMS

1. **FUNCTION**
    - Flood control
    - Power generation
    - Irrigation
    - Water Supply
    - Industrial
    - Environmental

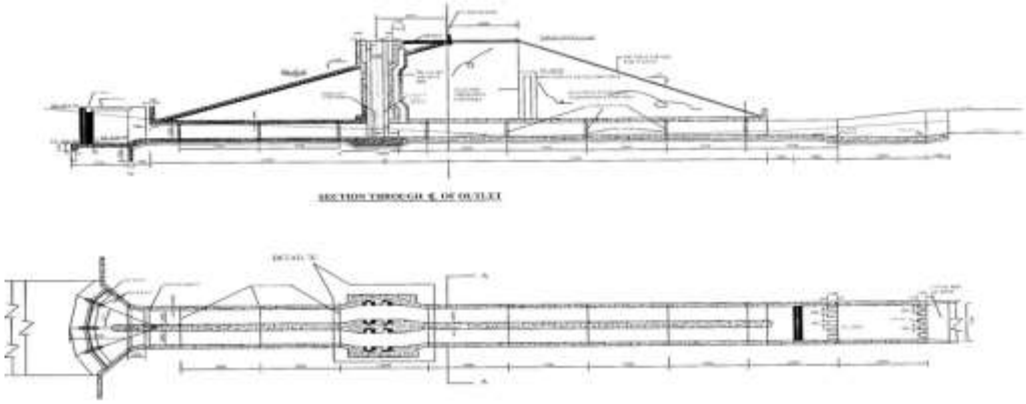




2. **BASIC ISSUES**
    - Alignment
    - Conduit Elevation
    - Shape
    - Spacing

3. **DESIGN CONSIDERATIONS**
    - Sill Level Close To Mddl
    - Pressure Conduit Upto Gate
    - Free Flow After Gate. 75% Full
    - Energy Desipation At The End Of Conduit
    - To Join With D/S Canal With Smooth Transtion Slopes

4. **Design Data**
    - Discharge
    - MDDL
    - Canal Bed Level at the downstream
    - Canal FSL
    - Dam Section
5. **Geological and other Data**
    - Not very high loads are expected to come.
    - The foundation shall be hard compacted.
    - Can be constructed on any type of soil/rock.
    - Seismology and Hydrology not required.
6. **Components of Outlet**
    - Trash rack
    - Entrance shape Bell Mouth
    - Outlet conduit
        - Pressure flow
        - Free flow
    - Control Shaft
    - Energy Dissipation arrangement
    - Joining with downstream canal
7. **Control gate/shaft position**
    - Upstream
    - Intermediate point
        - Outside control
        - Underground control
    - Downstream
8. **Issues**
    - Ungated
    - Length of pressure flow
    - Accessibility
    - Design Loading



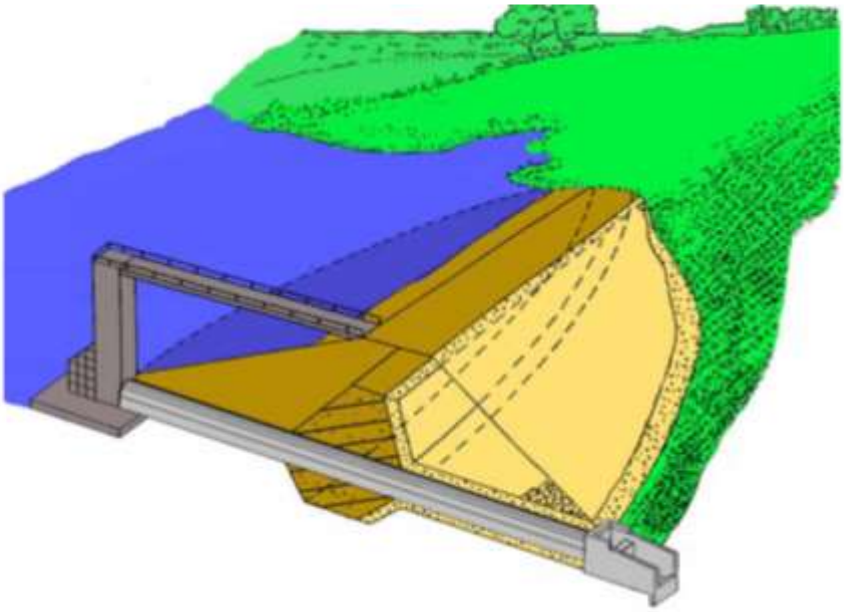


Figure 89.—Typical design used for embankment dams with distinctly different materials in the core zone and exterior shells of the dam.

9. Design Assumptions

- Trash rack bars area = 20 % of total trash rack area.
- Trash rack is designed for a velocity of 0.6 m/sec with 50 % clogging.
- Value of Manning's Coefficient ‘n’ is 0.018.
- Depth of flow in free flow portion shall not be more than 75% of total depth.
- The flow is supercritical in the free flow portion and Frouds number slightly more than 1.0

10. Design Criteria/ Conditions of Analysis

- Intake is designed
  - Minimum hydraulic losses
  - Provide smooth entry to outlet
  - Prevent trash entering the outlet
- Bell Mouth Entrance for smooth entry of flow to avoid cavitation pressure develop.
- An elliptical curved shape to minimize negative pressure
- $X^2/D^2 + Y^2/(0.33D)^2 = 1$ where D = height of outlet
- The piers of trash rack shall be sharp nosed and stream lined with the structure.
- Sill Level close to or Lower than MDDL

11. Design

- Size of outlet.
- Single or Multi barrel outlet.
- Length of pressure and free flow conduit.
- Depth of free flow.
- Losses (maximum & minimum)
- Minimum operating head.
- Full discharge with Maximum head loss (check).
- Water profile in free flow portion.
- Design of energy Dissipation.
- Transition details between conduit and stilling basin.
- Transition between stilling basin and canal
- Shape of bell mouth.
- Size and dimensions of trash rack piers etc.
- Structural design
  - Conduit
  - Control Shaft
  - Stilling Basin

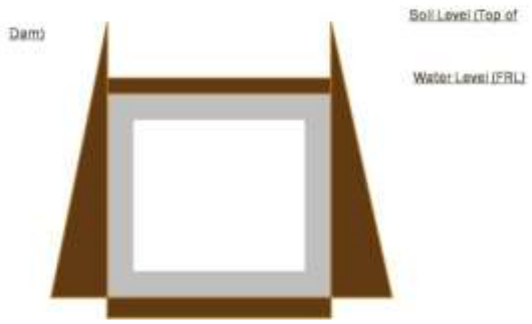
12. Losses

- Trash rack loss
- Entrance loss
- Stop log and Valve loss
- Friction loss
- Transition loss due contraction
- Loss due to Gate grove
- Transition loss due to Expansion
- Exit Loss

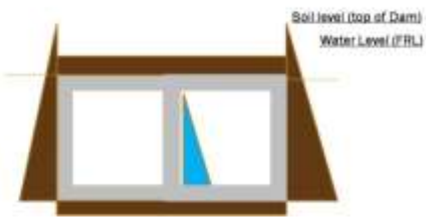
13. Structural design

- Design parameters
  - Angle of internal friction
  - Saturated, Submerged & Moist unit wt. of Soil.
- Design Assumptions
  - Foundation is placed on firm foundation with uniform upward reaction
  - There is no differential settlement
  - Full earth and water load above the outlet are acting on the top and side walls.

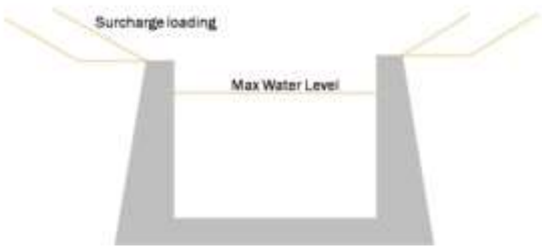
LOADING



DOUBLE BARREL

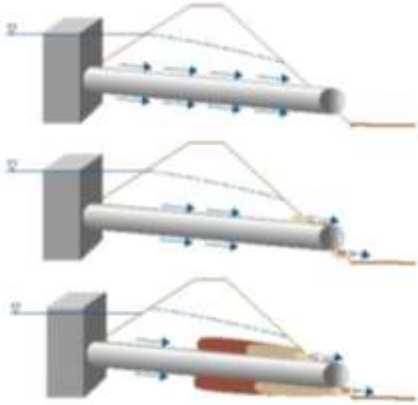


ENERGY DISSIPATION



14. PIPE AND CONCRETE OUTLET

- Pipe outlets
  - For less discharge
  - Difficult to maintain
- Prone to rusting.
  - ✓ Concrete outlets shall be preferred.
  - ✓ The cost compared to other components is not much





General Characteristics of Rock fill Dams

Rock fill dams are basically embankment dams. High inherent stability and high perviousness are their special characteristics. The impervious element in a rock fill dam is provided either by an impervious membrane of manufactured material or by an earth core. Even in the case of concrete-faced rock fill dams, the requirement of cement and other manufactured materials is much less than for concrete dams, while in earth core dams, it is similar to earth dams.

As in other embankment dams, in rock fill dams also the spillway and outlets have normally to be provided separate and away from the dam section. However, by placing heavier rock pieces near the downstream toe and tying them to the main rock fill by anchor bars, they can withstand limited overtopping. This can be taken advantage of in the determination of diversion flood during construction and freeboard for highest flood.

Due to free drain ability and high frictional strength, rock fill dams have a high inherent stability. In fact, there is no recorded case of failure of a rock fill dam due to slope sliding. Their high damping capacity and flexibility also give them a high level of protection against earthquake damage.

Rock fill dams require foundations intermediate in strength between concrete gravity dams and earth dams.

Membrane Versus Earth Core Rock fill Dams

The selection of the dam is mainly governed by the site conditions. This equally applies to the choice between membrane and earth core for rock fill dams. The circumstances in which the membrane type is likely to prove advantageous are:

- When suitable soil for an earth core is not available within a reasonable distance, as for example in high rocky areas.
- When foundations of hard rock with little overburden are available. In this situation there is no possibility of failure through the foundations and the dam slopes can be designed solely on the basis of shear strength of rock fill. The slopes can be kept somewhat steeper than those for the earth core section.
- Continuously rainy weather. In continuously rainy whether, it is difficult to maintain proper moisture control for compaction of earth cores. There is no such problem in the placement of rock fill. The placement of the membrane would require fair weather but it can be completed in a relatively short time after placement of the rock fill.

- In sand-gravel dams and at locations where large-size stones for wave protection are not available, a membrane can serve for wave protection as well as water tightness.

The following can be considered as the important advantages of an upstream membrane over earth core.

- Greater Stability: It provides greater safety against shear failure than any other type of embankment dam. The high stability is obtained due to total absence of adverse seepage forces and the availability of the entire rock fill mass to resist the water pressure acting on the upstream face.
- Greater tolerance for leakage: If ordinary leakage develops through the membrane, it drains out through the rock fill and does not endanger the dam. In fact, rock fills can withstand considerable flows before sliding occurs [4, 5, 6] and, unlike earth cores, the membrane itself is not subject to progressive erosion by leaks. If the membrane and the underlying drainage layer are bulk headed into sections and a drainage gallery is provided, the leaks can be easily located and treated. In the case of earth cores, location as well as treatment of leaks is difficult.
- Accessibility of membrane for inspection and repairs: This is normally feasible up to the lowest drawdown level. But even below this modern techniques enable underwater inspection, photographing or video recording and effective repairs.
- Speed of construction: For earth cores, the speed of construction is limited by the necessity of compaction of the soil in relatively thin layers and placement of graded filters on either side of the core. With a membrane, the placement of rock fill can be done faster and can continue without interruption.
- Stage construction facility: It is often advantageous to limit investment by building the dam to a lower height in the first instance and raising it later when utilization increases or the reservoir silts up. The upstream membrane type of dam is the simplest to raise in height. More rock fill can be placed on the downstream side and the membrane extended in continuation.

However, there are also some factors unfavorable to membrane relative to earth cores:

- Limited life: Earth cores can last indefinitely as they are not subject to deterioration. They can only be damaged by leaks. Most membranes deteriorate with time due to weathering, chemical action or other causes, and may require heavy repairs during the lifespan of the dam.

- Higher cost: At a location where suitable soil material for an earth core is available within a reasonable distance, a membrane would generally be more expensive.
- Limitation on height: The highest existing membrane-type dam is 160m while earth core dams have reached up to and beyond 300m. Larger heights are being contemplated for both. Cooke [2] has expressed the opinion that membrane-type dams could well be built to heights of 250m. Nevertheless, in the range of very large heights, there is less existing experience and consequent concern about cracking of the membrane due to rock deformation under immense water pressures.
- Possibility of Leakage: Leakage on initial filling has occurred in several dams through cracks in membranes. Another vulnerable location is the joint between the membrane and the cut-off. This latter problem has been resolved by provision of a horizontal toe slab and grout curtain in place of a vertical cut-off. Though leakage through the membrane does not usually endanger the dam, it may require lowering of the reservoir and expensive repairs. Again, better compaction of rock fill, resulting in lesser deformations, and use of thinner, more flexible membranes are proving to be effective measures against membrane cracking and consequent leakage.

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Due to free drain ability and high frictional strength, rock fill dams have a high inherent stability. In fact, there is no recorded case of failure of a rock fill dam due to slope sliding. Their high damping capacity and flexibility also give them a high level of protection against earthquake damage.

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- When suitable soil for an earth core is not available within a reasonable distance, as for example in high rocky areas.
- When foundations of hard rock with little overburden are available. In this situation there is no possibility of failure through the foundations and the dam slopes can be designed solely on the basis of shear strength of rock fill. The slopes can be kept somewhat steeper than those for the earth core section.
- Continuously rainy weather. In continuously rainy whether, it is difficult to maintain proper moisture control for compaction of earth cores. There is no such problem in the placement of rock fill. The placement of the membrane would require fair weather but it can be completed in a relatively short time after placement of the rock fill.
- In sand-gravel dams and at locations where large-size stones for wave protection are not available, a membrane can serve for wave protection as well as water tightness.

The following can be considered as the important advantages of an upstream membrane over earth core:

- Greater Stability: It provides greater safety against shear failure than any other type of embankment dam. The high stability is obtained due to total absence of adverse seepage forces and the availability of the entire rock fill mass to resist the water pressure acting on the upstream face.
- Greater tolerance for leakage: If ordinary leakage develops through the membrane, it drains out through the rock fill and does not endanger the dam. In fact, rock fills can withstand considerable flows before sliding occurs and, unlike earth cores, the membrane itself is not subject to progressive erosion by leaks. If the membrane and the underlying drainage layer are bulkheaded into sections and a drainage gallery is provided, the leaks can be easily located and treated. In the case of earth cores, location as well as treatment of leaks is difficult.
- Accessibility of membrane for inspection and repairs: This is normally feasible up to the lowest drawdown level. But even below this modern techniques enable underwater inspection, photographing or video recording and effective repairs.
- Speed of construction: For earth cores, the speed of construction is limited by the necessity of compaction of the soil in relatively thin layers and placement of graded filters on either side of the core. With a membrane, the placement of rock fill can be done faster and can continue without interruption.
- Stage construction facility: It is often advantageous to limit investment by building the dam to a lower height in the first instance and raising it later when utilization increases or the reservoir silts up. The upstream membrane type of dam is the simplest to raise in height. More rock fill can be placed on the downstream side and the membrane extended in continuation.

However, there are also some factors unfavorable to membrane relative to earth cores:

- ♦ Limited life: Earth cores can last indefinitely as they are not subject to deterioration. They can only be damaged by leaks. Most membranes deteriorate with time due to weathering, chemical action or other causes, and may require heavy repairs during the lifespan of the dam.
- ♦ Higher cost: At a location where suitable soil material for an earth core is available within a reasonable distance, a membrane would generally be more expensive.
- ♦ Limitation on height: The highest existing membrane-type dam is 160m while earth core dams have reached up to and beyond 300m. Larger heights are being contemplated for both. Cooke [2] has expressed the opinion that membrane-type dams could well be built to heights of 250m. Nevertheless, in the range of very large heights, there is less existing

experience and consequent concern about cracking of the membrane due to rock deformation under immense water pressures.

- Possibility of Leakage: Leakage on initial filling has occurred in several dams through cracks in membranes. Another vulnerable location is the joint between the membrane and the cut-off. This latter problem has been resolved by provision of a horizontal toe slab and grout curtain in place of a vertical cut-off. Though leakage through the membrane does not usually endanger the dam, it may require lowering of the reservoir and expensive repairs. Again, better compaction of rockfill, resulting in lesser deformations, and use of thinner, more flexible membranes are proving to be effective measures against membrane cracking and consequent leakage.

GEOTECHNICAL INVESTIGATION

1. INTRODUCTION

Geotechnical investigations are performed to evaluate those geologic, seismologic, and soils conditions that affect the safety, cost effectiveness, design, and execution of a proposed engineering project. Insufficient geotechnical investigations, faulty interpretation of results, or failure to portray results in a clearly understandable manner may contribute to inappropriate designs, delays in construction schedules, costly construction modifications, use of substandard borrow material, environmental damage to the site, post-construction remedial work, and even failure of a structure and subsequent litigation. Investigations performed to determine the **geologic setting** of the project include: the *geologic*, *seismologic*, and *soil conditions* that influence selection of the project site; **the characteristics of the foundation soils and rocks**; **geotechnical conditions** which influence project safety, design, and construction; **critical geomorphic processes**; and **sources of construction materials**. Geotechnical investigations are to be carried out by engineering geologists, geological engineers, geotechnical engineers, and geologists and civil engineers with education and experience in geotechnical investigations. Geologic conditions at a site are a major influence on the environmental impact and impact mitigation design, and therefore, a primary portion of geotechnical investigations is to observe and report potential conditions relating to environmental impact.

2. SCOPE OF INVESTIGATION

From project conception through construction and throughout the operation and maintenance phase, geotechnical investigations are designed to provide the level of information appropriate to the particular project development stage. In most instances, initial geotechnical investigations will be general and will cover broad geographic areas. As project development continues, geotechnical investigations become more detailed and cover smaller, more specific areas. For large, complex projects, the geotechnical investigation can involve highly detailed geologic mapping such as a rock surface for a structure foundation.

It must be understood in the beginning itself that an unreasonably extensive, and hence, expensive site investigation in an attempt to define in infinite detail every aspect of ground conditions is not appropriate. A rational approach would be to carry out investigations to a level, which will allow the design and construction of the project to proceed with an acceptable level of confidence. Accordingly, Ministry of Water Resources, Govt. of India has published a "**Guidelines for preparation of Detailed Project Reports of Irrigation & Multipurpose River Valley Projects**", which prescribes norms for carrying out the geotechnical investigations systematically during preparation of Detailed Project Reports. A generalized scope of the various increments of investigation is described in the following paragraphs, but these may be modified depending up on particular project requirements and local conditions.

Reconnaissance and Feasibility Studies

Reconnaissance studies are made to determine whether a problem has a solution acceptable to local interests and is in accordance with administrative policy. If so, reconnaissance studies provide information to determine whether planning should proceed to the feasibility phase. **Feasibility studies** identify and evaluate the merits and shortcomings of environmental, economic, and engineering aspects of the proposed project. *Geotechnical investigations during planning studies should be designed to provide information at a level such that critical geotechnical features of candidate sites may be compared in the feasibility study report.* These investigations should be sufficiently complete to permit selection of the most favorable site areas within the regional physical setting, determine the general type of structures best suited to the site conditions, evaluate the influence of hydrogeology on site design and construction, assess the geotechnical aspects of environmental impact, and to ascertain the costs of developing the various project plans in sufficient detail to allow comparative cost estimates to be developed.

Planning-level geotechnical investigations are generally performed in two parts:

(i) Development of regional geology, and,
(ii) Initial site investigations.

The regional geology investigations are carried out during early stages of the study. A schematic diagram for the development of regional geology is given in Fig. 1. Initial field investigations begin after the regional studies are sufficiently detailed to identify areas requiring geotechnical clarification. The schematic diagram for initial field investigations is given in Fig. 2.

The feasibility report should contain summaries of the regional geology, soils, hydrogeology, and seismological conditions plus brief summaries of the aerial and site geotechnical conditions for each detailed plan. These summaries should include local topography, geomorphic setting and history, thickness and engineering character of overburden soils, description of rock types, geologic structure, degree of rock weathering, local ground water conditions, possible reservoir rim problems, description of potential borrow areas and quarries, accessibility to sources of construction materials, etc. In addition, special foundation conditions such as excavation or dewatering problems, low strength foundations, and cavernous foundation rock should be described. The summaries should conclude with a discussion of the relative geotechnical merits and drawbacks of each plan.

Preconstruction Engineering and Design Studies / Detailed Project Report Studies

Preconstruction engineering and design (PED) or Detailed Project Report (DPR) studies are typically initiated after a feasibility study has been completed. PED / DPR studies are developed *to reaffirm the basic planning decisions made in the feasibility study, establish or reformulate the scope of the project based on current criteria and costs, and formulate the design memoranda which will provide the basis for the preparation of plans and specifications*. Fig. 3 schematically outlines the engineering tasks for the PED / DPR studies with the requirements for geotechnical information. The provisions of Ministry of Water Resources, Govt. of India "**Guidelines for preparation of Detailed Project Reports of Irrigation & Multipurpose River Valley Projects**" is given in Annex A.

Geotechnical investigations performed during the PED / DPR studies should be in sufficient detail to assure that authorized measures can be implemented. The emphasis is toward site-specific studies, which will provide the detail and depth of information necessary to select the most suitable site and structures to achieve project goals. The studies are performed in a series of incremental steps of increasing complexity beginning with the site selection study on major projects and continuing through feature design studies, which are

(i) Site selection study (see Fig. 4)
(ii) Design investigations (see Fig. 5)
(iii) Formulation and evaluation of construction plans and specifications
(iv) Estimates of Costs & Benefits

Construction Stage

Geotechnical activities in support of the construction phase of a project can be divided into three phases: construction management, quality assurance, and compilation of summary reports. Fig. 6 outlines the tasks for construction geotechnical activities.

- **Construction management**

(a) **Claims and modifications** - Regardless of the intensity of geological investigations during the preconstruction phase, differing site conditions, claims, and modifications are to be expected on complex projects. Most common claim relates to overbreak due to geological reasons. On a few occasions due to adverse geological conditions or geological surprises, modifications in the project layout become necessary. Therefore, geotechnical engineers should provide necessary support to investigate claims and provide design and cost-estimating assistance for any claims and modifications.

(b) **Site visits for verification of quality** - On all projects, but especially those too small to support a resident geologist or geotechnical engineer, site visits should be made regularly by qualified personnel to verify that conditions match assumptions used in designing the project features and to assist construction personnel on any issues affecting construction. All visits should be well documented (including an extensive photographic and video record) and be included in appropriate summary reports.

- **Quality assurance**

The geotechnical engineer assigned to a particular project has the responsibility to monitor, observe, and record all aspects of the construction effort relating to foundations, embankments, cuts, tunnels, and natural construction materials. Fig.7. shows in tabulated form some of the particular items requiring quality assurance particular to geo-technically oriented features. An onsite laboratory should be required on major projects to perform all soil and concrete testing. During construction, considerable data are assembled by the project geotechnical quality assurance staff. These data consist generally of foundation mapping and treatment features, embankment-backfill performance data, grouting records, material testing data, pile driving records, and instrumentation results. Special treatment and problem areas, often requiring contract modification, should be well documented.

- **Construction Geotechnical Reports**

The purpose of the construction geotechnical report is to ensure the preservation for future use of complete records of foundation conditions encountered during construction and methods used to adapt structures to these conditions. It is an important document for use in evaluating construction claims, planning additional foundation treatment should the need arise, evaluating the cause of foundation or structural feature distress and planning remedial action to prevent failure or partial failure of a structure, planning and design of major rehabilitation or modifications to the structure, providing guidance in planning foundation explorations, and in anticipating foundation problems for future comparable construction projects in similar geologic settings. Site geotechnical personnel responsible for the foundation report must begin to formulate the report as soon as possible after construction begins so that completion of the report can be accomplished by those who participated in the construction effort. Fig. 8. presents the schematic diagram of construction geotechnical investigations and documentation.

This report should include collaboration with design and construction personnel. Detailed video recordings of foundation conditions should be an integral component of the foundation report.

3. INVESTIGATION METHODS

There are several possible methods for geotechnical investigations. The major factors influencing the **selection of methods** of investigation include:

(i) Nature of subsurface materials and groundwater conditions
(ii) Size of structure to be built or investigated
(iii) Scope of the investigation, e.g., feasibility study, formulation of plans and specifications
(iv) Purpose of the investigation, e.g., evaluate stability of existing structure, design a new structure
(v) Complexity of site and structure
(vi) Topographic constraints
(vii) Difficulty of application
(viii) Degree to which method disturbs the samples or surrounding grounds
(ix) Budget constraints
(x) Time constraints
(xi) Environment requirements/consequences
(xii) Political constraints

Primarily, there are two types of geotechnical investigations, i.e., surface and sub-surface. The detailed description of the various methods under these types are beyond the scope of this lecture. However, they are briefly described below;

SURFACE INVESTIGATIONS

Surface investigations include the field operations that do not involve significant disturbance of the ground at the time the investigation is conducted. This type of investigation typically occurs at a preliminary stage of projects and supplies generalized information. However, these investigations can involve mapping specific locations in great detail during construction. The end product is commonly a pictorial rendering of conditions at the site. The degree of accuracy and precision required in such a rendering varies with the application and purpose for which the information is to be used.

The surface investigations include;

3.1.1 Geological Field Mapping –

- **Aerial Mapping** - The purpose of aerial mapping is to develop an accurate picture of the geologic framework of the project area. The area and the degree of detail to be mapped can vary widely depending on the type and size of the project and on the complexity of the regional geology. In general, the area to be mapped should include the project site(s) as well as the surrounding area that could influence or could be influenced by the project.

  Geologic and environmental features within the reservoir and adjacent areas that should be studied and mapped include the following:

  (i) Faults, joints, stratigraphy, and other significant geologic features.
  (ii) Karst topography or other features that indicate high reservoir leakage potential.
  (iii) Water well levels, springs, surface water, water-sensitive vegetation, or other evidence of the
  (iv) Ground water regime.
  (v) Soluble or swelling rocks such as gypsum or anhydrite.
  (vi) Potential landslide areas around the reservoir rim.
  (vii) Valuable mineral resources.
  (viii) Mine shafts, tunnels, and gas and oil wells.
  (ix) Potential borrow and quarry areas and sources of construction materials.
  (x) Shoreline erosion potential.
  (xi) Landfills, dumps, underground storage tanks, surface impoundments, and other potential environmental hazards.

- **Site Mapping -** Large-scale and detailed geologic maps should be prepared for specific sites of interest within the project area and should include proposed structure areas and borrow and quarry sites. Investigation of the geologic features of overburden and rock materials is essential in site mapping and subsequent explorations. Determination of the subsurface features should be derived from a coordinated, cooperative study of information on origin, distribution, and manner of deposition of the overburden / rock and the engineering properties of the site foundation and potential construction materials, potential problem materials or conditions, application of geologic conditions to design, and the adaptation of proposed structures to foundation conditions.

- **Construction Mapping -** Construction maps record in detail geologic conditions encountered during construction. Traditionally, a foundation map is a geologic map with details on structural, lithologic, and hydrologic features. It can represent structure foundations, cut slopes, and geologic features in tunnels or large chambers. The map should be prepared for soil and rock areas and show any feature installed to improve, modify, or control geologic conditions. Some examples are rock reinforcing systems, permanent dewatering systems, and special treatment areas. The mapping of foundations is usually performed after the foundation has been cleaned just prior to the placement of concrete or backfill. The surface cleanup at this time is generally sufficient to permit the observation and recording of all geologic details in the foundation. An extensive photographic and videographic record should be made during foundation mapping.

3.1.2 Surface Geophysical Explorations

Geophysical exploration consists of making indirect measurements from the earth's surface or in boreholes to obtain subsurface information. Geologic information is obtained through analysis or interpretation of these measurements. Boreholes or other subsurface explorations are needed to calibrate or validate geophysical measurements. Geophysical explorations are of greatest value when performed early in the field exploration program in combination with limited subsurface exploration. They are appropriate for a rapid location and correlation of geologic features such as stratigraphy, lithology, discontinuities, ground water, and the in situ measurement of elastic moduli and densities. The cost of geophysical explorations is generally low compared with the cost of core borings or test pits, and considerable savings may be realized by judicious use of these methods.

The six major geophysical exploration methods are seismic, electrical resistivity, sonic, magnetic, radar, and gravity. Of these, the seismic and electrical resistivity methods have found the most practical application to the engineering problems.

SUB-SURFACE INVESTIGATIONS

Subsurface investigations require use of equipment to gain information below the ground surface. The equipment is typically invasive and requires disturbance of the ground to varying degrees. Most of these exploration techniques are relatively expensive, and therefore, should be carefully planned and controlled to yield the maximum amount of information possible. The major planning factors are;

- **Location of Investigations**

An important piece of information for all geotechnical investigations that seems obvious but commonly not given sufficient attention is the accurate determination of the location of investigation. It is always preferable to select boring and test pit locations that fully characterize geotechnical conditions. Although correlation of information from offsite may be technically defensible, because of variability of geologic materials, the legal defensibility of a piece of information is commonly lost if it is even slightly removed from the site. In several DPRs, it has been observed that extensive investigation is done on a particular site and then due to some geotechnical problem, that particular site gets rejected and an alternative site is identified. But only limited investigation is done at the new site, either due to time or cost constrains. Ministry of Water Resources, Govt. of India publication "*Guidelines for preparation of Detailed Project Reports of Irrigation & Multipurpose River Valley Projects*" enclosed as Annex A provides guidelines for the location of investigations.

➢ **Protection of the Environment**

After the locations for field investigations work have been determined, routes of access to the area and the specific sites for borings and excavations should be selected with care to minimize damage to the environment.

3.2.1 Borings / Drilling

Borings / drilling are most important method of sub-surface investigation. It is required to characterize the basic geologic materials at a project. The major uses for which borings are made are as follows:

a. Define geologic stratigraphy and structure.
b. Obtain samples for index testing.
c. Obtain ground water data.
d. Perform in situ tests.
e. Obtain samples to determine engineering properties.
f. Install instrumentation.
g. Establish foundation elevations for structures.
h. Determine the engineering characteristics of existing structures.

Borings are classified broadly as disturbed, undisturbed, and core. Borings are frequently used for more than one purpose, and it is not uncommon to use a boring for purposes not contemplated when it was made. Thus, it is important to have a complete log of every boring, even if there may not be an immediate use for some of the information. If there is doubt regarding the range of borehole use or insufficient information to determine optimum borehole size, then the hole should be drilled larger than currently thought needed. A slightly larger than needed borehole is considerably less expensive than a second borehole.

Many methods are used to make borings and retrieve samples. Most common methods are discussed in the following paragraphs. Some factors that affect the choice of methods are:

(i) *Auger borings.*
(ii) *Drive borings.*
(iii) *Cone penetration borings.*
(iv) *Undisturbed borings.*
(v) *Rock core boring.*

Use of any of the above methods depends on the following factors;

(1) Purpose and information required.
(2) Equipment availability.
(3) Depth of hole.
(4) Experience and training of available personnel.
(5) Types of materials anticipated.
(6) Terrain and accessibility.
(7) Cost.
(8) Environmental impacts.
(9) Disruption of existing structure.

A major part of field investigations is the compilation of accurate borehole logs on which subsequent geologic and geotechnical information and decisions are based. A field drilling log for each borehole can provide an accurate and comprehensive record of the lithology and stratigraphy of soils and rocks encountered in the borehole and other relevant information obtained during drilling, sampling, and in situ testing.

3.2.2 Exploratory Excavations

- **Test Pits and Trenches**

Test pits and trenches can be constructed quickly and economically by bulldozers, backhoes, pans, draglines, or ditching machines. Depths generally are less than 6 m (20 ft), and sides may require shoring, if personnel must work in the excavations. Test pits, however, hand dug with pneumatic jackhammers and shored with steel cribbing, can be dug to depths exceeding 18 m (60 ft). Test pits and trenches generally are used only above the ground water level. Test pits that extend below the water table can be kept open with air or electric powered dewatering pumps. Exploratory trench excavations are often used in fault evaluation studies. An extension of a rock fault into much younger overburden materials exposed by trenching is usually considered proof of recent fault activity. Shallow test pits are commonly used for evaluating potential borrow areas, determining the geomorphic history, and assessing cultural resource potential.

- **Large-scale, Prototype Investigations**

Whenever the size and complexity of a project warrant, a large-scale, prototype test programs can yield information unavailable by any other method. Because these investigations are expensive and require the services of a construction contractor in most cases. For large projects, it is desirable to perform these investigations during the PED / DPR phase, as they provide a number of benefits that will result in an improved, more cost-effective design. These benefits include: confirmation of assumptions for new or innovative design, improved confidence level allowing reduced safety factors, proof of constructability, confirmation of environmental compliance, and greater credibility in allaying public concerns.

- **Exploratory Tunnels / Adits / Shafts**

Exploratory tunnels / adits / shafts permit detailed examination of the composition and geometry of rock structures such as joints, fractures, faults, shear zones, other discontinuities, range & extent of weathering and solution channels. They are commonly used to explore conditions at the locations of large underground excavations and the foundations and abutments for large dams. They are particularly appropriate in defining the extent of marginal strength rock or adverse rock structure suspected from surface mapping and boring information or when drilling is not possible due to inaccessibility / depths greater than 300 m or so. For major projects where high-intensity loads will be transmitted to foundations or abutments, tunnels afford the only practical means for testing *in-situ* rock at locations and in directions corresponding to the structural loading. Although expensive, exploratory tunnels / adits / shafts provide exceptionally good preconstruction information to perspective contractors on major underground projects and can reduce bid contingencies and/or potential for claims. Long horizontal exploratory drill holes can also be used in lieu of, or in combination with, pilot tunnels to gather information about tunneling conditions prior to mining.

In the case of planned underground construction, an exploratory tunnel is often used to gain access to crown and roof sections of future large underground excavations. The tunnel can then be used during construction for equipment access and removal of excavated rock. A small bore or exploratory “pilot” tunnel is sometimes driven along the entire length of a proposed larger-diameter tunnel where difficult and often unpredictable ground conditions are anticipated. A pilot tunnel may be the most feasible alternative for long deep tunnels where deep exploratory drilling and access for in situ testing from the ground surface is prohibitively expensive. The pilot tunnel can be positioned to allow installation of roof support and/or consolidation grouting for critical areas of the full tunnel, or in some cases, to provide relief or “burn cuts” to facilitate blasting. Exploratory tunnels, that are strategically located, can be incorporated into the permanent structure. They can be used for drainage and post-construction observations to determine seepage quantities and to confirm certain design assumptions.

3.2.3 Laboratory Investigations

The purpose of laboratory tests is to investigate the physical and hydrological properties of natural materials such as soil and rock, determine index values for identification and correlation by means of classification tests, and define the engineering properties in parameters usable for design of foundations. The engineering geologist and/or geotechnical engineer, using the test data and calling upon experience, can then accomplish safe and economical designs for engineering structures. A summary of purpose and laboratory tests are given below;

Purpose of Test	Type of Test
Strength	Uniaxial Compression Direct Shear Triaxial Compression Direct Tension Brazilian Split Point Load
Deformability	Uniaxial Compression Triaxial Compress Swell Creep
Permeability	Gas Permeability
Characterization	Water Content Porosity Density (Unit Weight) Specific Gravity Absorption Rebound Sonic Velocities Abrasion Resistance

3.2.4 In-situ Tests

In-situ tests are often the best means for determining the engineering properties of subsurface materials and, in some cases, may be the only way to obtain meaningful results. Table, given below, lists in-situ tests and their purposes. In-situ tests are performed to determine in-situ stresses and deformation properties of the jointed rock mass, shear strength of jointed rock mass or critically weak seams within the rock mass, residual stresses within the rock mass, anchor capacities, and rock mass permeability. Large-scaled in-situ tests tend to average out the effect of complex interactions. In-situ tests are generally expensive and should be reserved for projects with large, concentrated loads. Well-conducted tests may be useful in reducing overly conservative assumptions. Such tests should be located in the same general area as a proposed structure and test loading should be applied in the same direction as the proposed structural loading.

Purpose of Test	**Type of Test**
Strength	Field Vane Shear[1]
	Direct Shear
	Pressuremeter[2]
	Uniaxial Compressive[2]
	Borehole Jacking[2]
Bearing Capacity	Plate Bearing[1]
	Standard Penetration[1]
Stress Conditions	Hydraulic Fracturing
	Pressuremeter
	Overcoring
	Flat Jack
	Uniaxial (Tunnel) Jacking[2]
	Chamber (Gallery) Pressure[2]
Mass Deformability	Geophysical (Refraction)[3]
	Pressuremeter or Dilatometer
	Plate Bearing
	Uniaxial (Tunnel) Jacking[2]
	Borehole Jacking[2]
	Chamber (Gallery) Pressure[2]
Anchor Capacity	Anchor / Rockbolt Loading
	Pull Out Tests
Rock Mass Permeability	Constant Head
	Rising or Falling Head
	Well Slug Pumping
	Pressure Injection

Notes:

1. Primarily for clay shales, badly decomposed, or moderately soft rocks, and rock with soft seams.
2. Less frequently used.
3. Dynamic deformability.

राष्ट्रीय जल अकादमी

पुणे स्थित राष्ट्रीय जल अकादमी, केन्द्रीय जल आयोग की एक विशिष्ट संस्था है। जल संसाधन क्षेत्र से जुडे राज्य तथा केन्द्र सरकार में विविध स्तर पर कार्यरत अभियंताओं के प्रशिक्षण के क्षेत्र में राष्ट्रीय जल अकादमी एक “उत्कृष्ट केन्द्र” के रूप में कार्य कर रही है। राष्ट्रीय जल अकादमी जल संसाधन के विकास एवं प्रबन्धन के क्षेत्र में अल्प एवं मध्यम अवधि के पाठ्यक्रमों के नियमित आयोजन के साथ-साथ केन्द्रीय जल अभियंत्रण (वर्ग ‘क’) सेवा के अंतर्गत चयनित अधिकारियों के लिए लम्बी अवधि का प्रवेशन कार्यक्रम भी आयोजित करता है।

राष्ट्रीय जल अकादमी की वेबसाइट http://nwa.mah.nic.in से इस संबंध में अधिक जानकारी प्राप्त की जा सकती है ।