Chapter-2
EARTHFILL DAM ESIGN &ANALYSIS

Embankment dam

- Any dam constructed of excavated materials placed without addition of binding material other than those inherent in the natural material

EMBANKMENT DAMS

- Earthfill

- Rockfill

- Earth-rockfill

Earthfill Dam

- An embankment dam, constructed primarily of compacted earth materials, either homogeneous or zoned, and containing more than 50% of earth granular materials ($0.001 \leq d \leq 100$ mm) .

compacted soils account for over 50% of the placed volume of material

Rockfill Dam

- An embankment dam constructed of natural rock materials, usually broken down to smaller fragments ($0.1 \leq d \leq 1000$ mm).

Earthfill-Rockfill Dam

- An embankment dam where large quantities of both granular materials (earth) and rock fragments are used (over 50% of the fill material may be classified as rockfill)

Grain-size classification of unconsolidated deposits

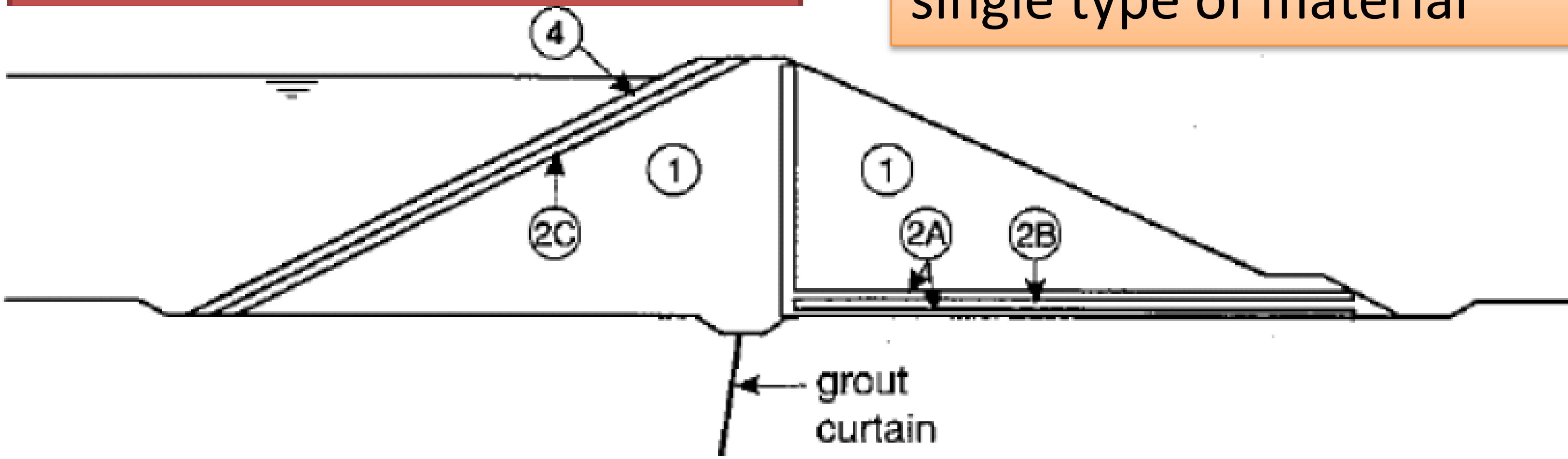
Designation	*Particle diameter (mm)*
Cobbles, bowlders, blocks	> 60
Coarse gravel	60–20
Medium gravel	20–6
Fine gravel	6–2
Coarse sand	2–0.6
Medium sand	0.6–0.2
Fine sand	0.2–0.06
Coarse silt	0.06–0.02
Medium silt	0.02–0.006
Fine silt	0.006–0.002
Clay	< 0.002

(Source : A M S , D A M F O U N D A T I O N S , A N D R E S E R V O I R S by ERNEST E. WAHLSTROM)

TYPE OF EARTHFILL DAMS

i. Homogenous earthfill dam

composed entirely of a single type of material



Zone 1 : earthfill (core) – controls seepage through the dam.

Zone 4 : rip rap – controls erosion of u/s slope by wave action.

Zone 2C: filter – controls erosion of zone 1 through rip rap.

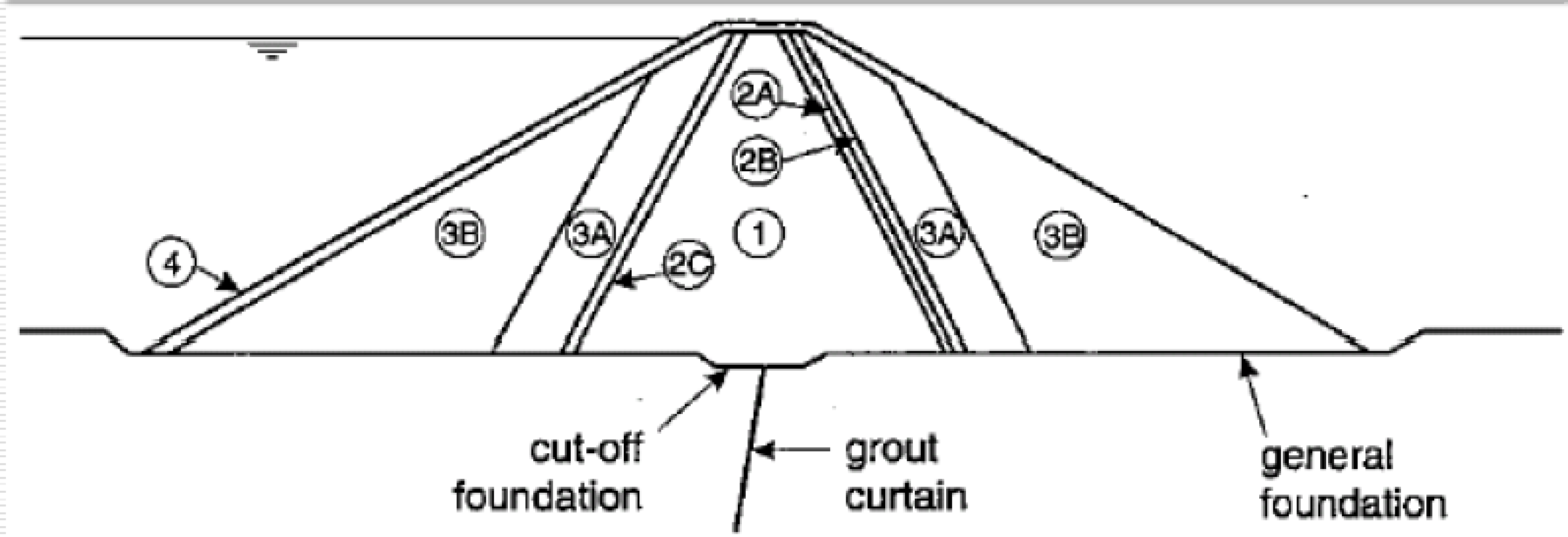
Zone 2A: fine filter – controls erosion of zone 1 and dam foundation.

- controls build up of pore pressure in d/s face.

Zone 2B: coarse filter – discharges seepage water collected in vertical or horizontal drain

ii. Zoned Embankment Dam

- The horizontal width of the impervious zone at any elevation equals or exceeds the height of embankment above that elevation in the dam and is at least 10 feet (USBR)



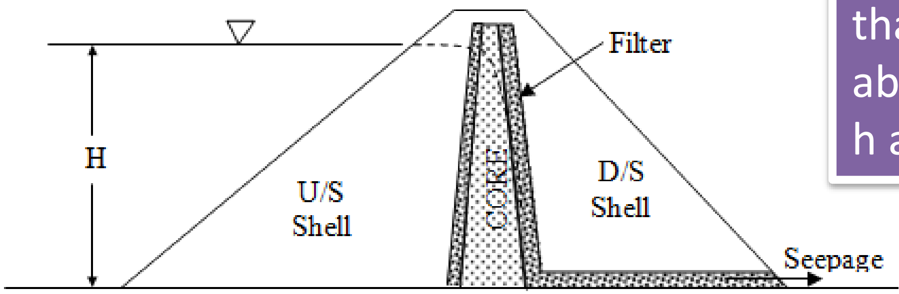
Zone 3A: earthfill/rockfill – provides stability, commonly free drainage to allow discharge . Prevents erosion of 2B into 3B.

Zone 3B: coarse rockfill – provides stability, commonly free drainage to allow discharge

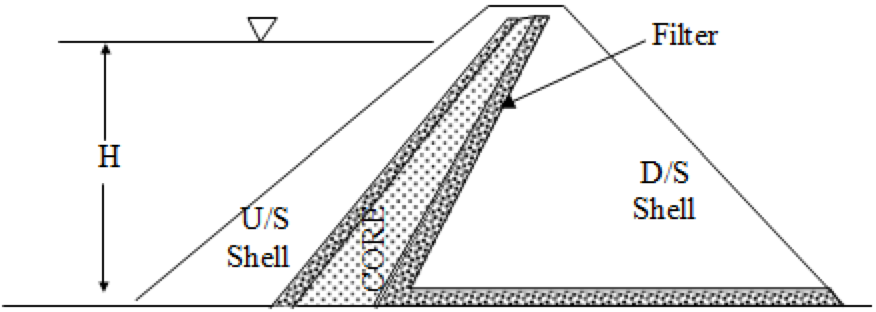
Table Embankment dam zones description and function.

Zone	Description	Function
1	Earthfill ("core")	Controls seepage through the dam
2A	Fine filter (or filter drain)	(a) Controls erosion of Zone 1 by seepage water, (b) Controls erosion of the dam foundation (where used as horizontal drain), (c) Controls buildup of pore pressure in downstream face when used as vertical drain
2B	Coarse filter (or filter drain)	(a) Controls erosion of Zone 2A into rockfill, (b) Discharge seepage water collected in vertical or horizontal drain
2C	(i) Upstream filter	Controls erosion of Zone 1 into rockfill upstream of dam core
	(ii) Filter under rip rap	Controls erosion of Zone 1 through rip rap
2D	Fine cushion layer	Provides uniform support for concrete face; limit leakage in the event of the concrete face cracking or joints opening
2E	Coarse cushion layer	Provides uniform layer support for concrete face. Prevents erosion of Zone 2D into rockfill in the event of leakage in the face
1–3	Earth-rockfill	Provides stability and has some ability to control erosion
3A	Rockfill	Provides stability, commonly free draining to allow discharge of seepage through and under the dam. Prevents erosion of Zone 2B into coarse rockfill
3B	Coarse rockfill	Provides stability, commonly free draining to allow discharge of seepage through and under the dam
4	Rip rap	Controls erosion of the upstream face by wave action, and may be used to control erosion of the downstream toe from backwater flows from spillways

iii. Diaphragm type



central vertical core



A diaphragm earthfill dam with inclined core

The inclined core is adopted instead of the center core where:

– a blanket zone is provided in the pervious foundation to be connected with the impervious core zone.

– different construction processes are available for the placement of core and rockfill materials.

– the site is a high seismic area

If the diaphragm material is earth, the horizontal thickness of the diaphragm at any elevation is less than 10 feet or the height of the embankment above the corresponding elevation of the dam ($W \leq h$ and $W \leq 10$ ft.)

A thin core dam becomes more economical for reasons as:

- Unit cost of placing impervious materials may be more than the unit cost of placing pervious materials.
- The amount of embankment volume can be reduced in a thin core dam more effectively
- The construction time available and weather conditions may not permit the use of an impervious core of large thickness.

The minimum thickness of core depends on a number of factors on:

- the tolerable seepage loss;
- minimum width which will allow proper construction (machinery considerations);
- type of materials chosen for the core and shoulders;
- design of proposed filter layers;
- past experience of similar projects.

Factors for dam type selection

Topography:

-A narrow V-shaped valley with sound rock in abutments would favor a concrete arch dam.
-A relatively narrow valley with rocky walls would suggest a rock fill or concrete gravity dam (or roller-compacted concrete dam).
-A wide valley with deep overburden would suggest an earth dam.
-Topography may also influence the selection of appurtenant structures Eg Natural saddles may provide a spillway location

Geology and foundation conditions

-Competent rock foundations with relatively high shear strength and resistance to erosion can be used for all types of dams.
-Gravel foundations, if well compacted, are suitable for earth or rock-fill dams with provisions for adequate seepage control.
-Silt or fine sand foundations can be used for low concrete (or RCC) and earth-fill dams but are not suitable for rock-fill dams.
-Non-dispersive clay foundations may be used for earth dams but require flat embankment slopes.

Materials available :

-If suitable soils for an earth-fill dam can be found in nearby borrow pits, an earth dam may prove to be more economical.

-The availability of suitable rock may favor a rock-fill dam.

-The availability of suitable sand and gravel for concrete at a reasonable cost locally or onsite is favorable to use for a concrete (or RCC) dam

Spillway:

-The size, type, and restrictions on location of the spillway are often controlling factors in the choice of the type of dam.

-When a large spillway is to be constructed, it may be desirable to combine the spillway and dam into one structure, indicating a concrete overflow dam.

-In some cases where required excavation from the spillway channel can be utilized in the dam embankment, an earth or rock-fill dam may be advantageous

Climate:

-The construction difficulty with earthfill during wet weather should be taken into consideration.

Economic:

-The final selection of the type of dam should be made only after careful analysis and comparison of possible alternatives, and after thorough economic analyses that include costs of all appurtenant structures, power and control structures, and foundation treatment.

Environmental:

-The need to consider protection of the environment affects the type of dam, its dimensions, and location of the spillway and appurtenant facilities.

Geotechnical Investigations

Two types of questions in dam projects

Engineering questions, which relate essentially to the **design, construction and operation** of any structure of the type proposed, and

Geological questions, which arise from **understanding of the site geological environment** and its likely influence on the design, construction and operation of the project

Most dam failures **occurred due to failure to fully understand & define the right geotechnical questions**

Checklist of Geotechnical questions for dam projects:

1.Source of Materials?

1. Sources of materials, for the following purposes:
   - Earthfill, for the core or other zones;
   - Filters;
   - Rockfill;
   - Rip-rap;
   - Concrete aggregates;
   - Road Pavements;

   For each material: Location of alternative sources, qualities/suitabilities, quantities, methods for winning and processing. Overburden and waste materials and quantities. Possible use of materials from required excavations, e.g. spillway, outlet works and dam foundations.

2. Reservoir?

- Watertightness;
- Effect on regional groundwaters – Levels or quality;
- Stability of slopes inside and outside of reservoir rim;
- Erodibility of soils – Possibility of turbidity problems;
- Siltation rates and likely location of deposits.

3. Dam?

- Location – To suit topographic and geological situations;
- Alternative* sites, for comparison of costs and of geotechnical and other issues;
- Type(s) of dam suited to site(s);
- Depths to suitable foundations for: concrete dam; earthfill; core; filters; rockfill; plinth or grout cap;
- Nature of materials to be excavated, excavation methods, and possible uses of materials;
- Stability of excavations, support and dewatering requirements;
- Permeability, compressibility and erodibility of foundations;
- Foundations treatment(s) required: grouting; drainage; slurry concrete; dental treatment; filter blanket; other;
- Embankment zones, methods of placement, and of control of quality, moisture and compaction;
- Stability of dam, and dam plus foundation in all situations;
- Monitoring systems: types, siting.

4. Appurtenant Structures?

- Location and type;
- Excavation method(s), possible use for excavated materials;
- Stability of excavations, need for temporary/permanent support;
- Channel, need for lining/drainage;
- Need for protection of the discharge area, or for excavation of a stilling basin.

5. Seismicity of the Region?

- Design earthquake, annual exceedance probability versus ground motion;
- Maximum credible earthquake.

MCE - The largest earthquake that appears capable of occurring under the known tectonic framework for a specific fault or seismic source, as based on geologic and seismologic data

Comment on the effects of first filling of a water supply dam in an arid region, in a steep sided valley underlain by a very weak sandstone.

- ❑ significant raising of the water table
- ❑ solution of water soluble mineral with resulting weakening and possible increase in permeability of the foundations, and
- ❑ possible instability in the storage area sides

specific geological questions to be answered during investigations of this site would include the following:

– What are the cementing agents in the sandstone?

– How much reduction in strength and stiffness will occur in the sandstone when saturated for long periods?

– Could solution effects during dam operations result in increase in permeability of the foundation?

– Could solution/strength reduction or water table rise result in instability in (a) the foundation or (b) the reservoir sides?

Checklist for engineering questions (design, construction and operation) of dam projects

Design

- *Stability of Dam Body*
  - **Stability against sliding failure of embankment**
    - Evaluation of pore-water pressure during and after construction,
    - Shear strength characteristics of fill materials
    - Deformation characteristics of fill materials
    - Settlements and internal cracking of dam materials
  - **Seismic stability**
    - Seismic coefficient method (No more at work!)
    - Liquefaction
    - Dynamic deformation characteristics of dam materials
    - Dynamic response analysis
    - Earthquake resistant design

Design ...

- **Stability at the contact face of dam body and base foundation**
  - Contact clay, Compaction, Relative displacement
  - Arching, Cracking
  - Limiting strength of dam body and base foundation

- *Seepage Through Embankment and Foundation*
  - **Seepage analysis**
    - Discharge, pore-water pressure
    - leakage through foundation
    - Piping, critical hydraulic gradient, hydraulic fracture

- *Foundation Treatment*
  - **Stability**
  - **Seepage**

Construction

- *Planning for Construction*
  - **Construction equipment**
    - roller, carrier, bulldozer, …
  - **Foundation treatment**
    - grouting, drainage
  - **Placement**
    - execution management, field and laboratory testing
  - **Observation**
    - pore-water pressure, settlement, earth pressure, deformation)
- *Maintenance and Repair*

Site Investigation Techniques.

- Topographic Maping & Survey
- Satellite Images & Arial Photographs
- Geological & Geotechnical Maps
- Geophysical Methods
- Test Pits & Trenches
- Drill Holes
- Sampling
- In Situ Tests (permeability, grouting, bearing capacity, compressibility)

Fill Materials

Check the required quality and quantity of materials

– Geological survey (stratum, volume)

– Laboratory testing (shear strength, compressibility, compaction, permeability,...

– In-situ testing (compaction, density log, field permeability, sampling)

Topographic Mapping & Survey

- ❖ Accurate location and level of all relevant data
- ❖ Topographic maps at several scales are required
  - Regional maps, 1:250 000 with 20–50 m contours to 1:25 000 with 10 m contours;
  - Catchment area, 1:25 000 with 10 m contours to 1:2000 with 2 m contours;
  - Project area, 1:1000 with 2 m contours to 1:200 with 1 m contours;
  - Individual engineering structures, 1:500 with 1 m contours to 1:200 with 0.5 m contours

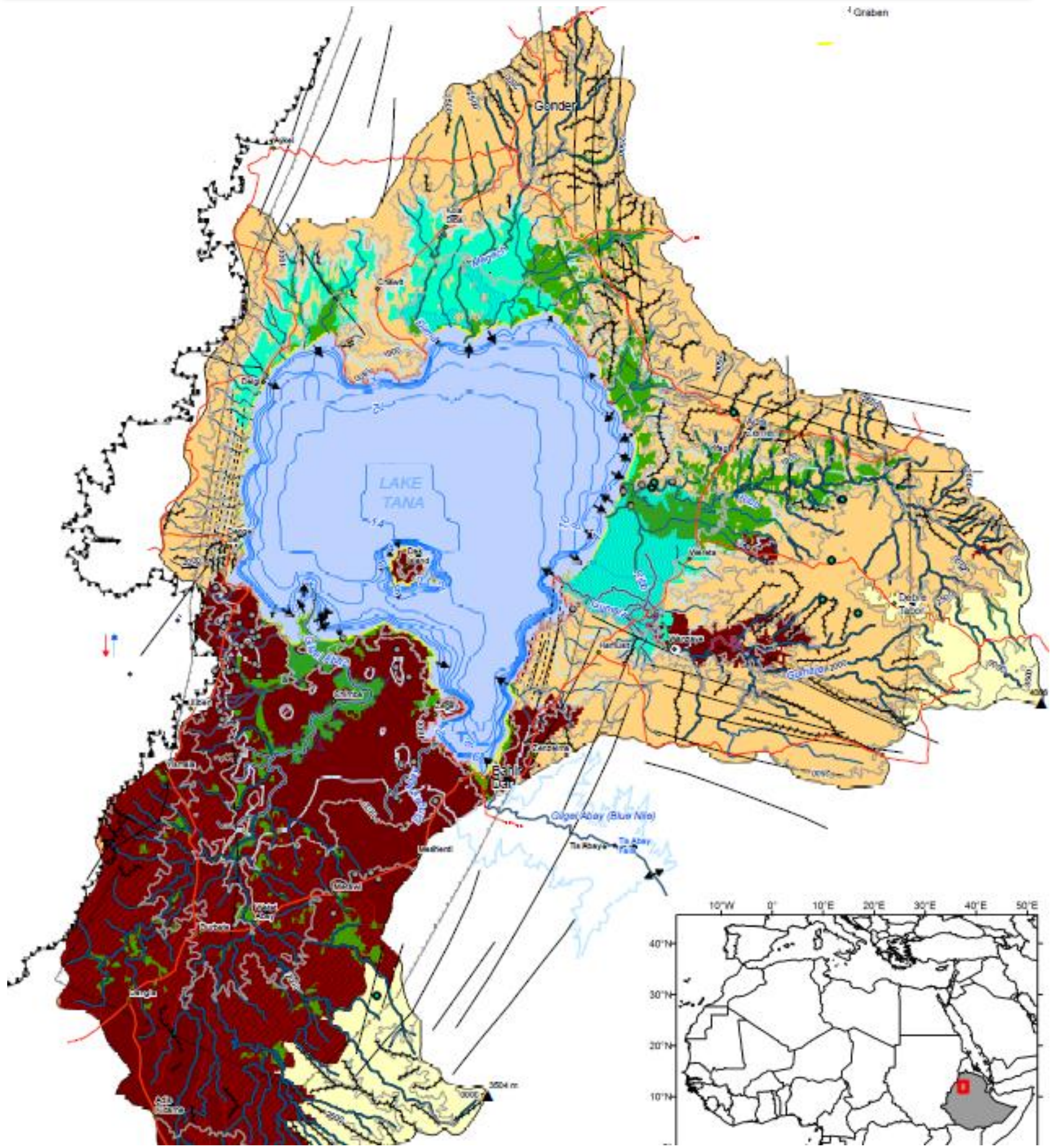
Satellite Images & Arial Photographs

- thematic map production by using GIS (for computing the plan area, surface area, and volume of the reservoir)
- For investigations of existing dams, photographs taken during construction are an invaluable aid to assessing the geology and construction of the dam
- provides an indication of relationships between the regional geology and landforms, drainage, soils, vegetation and land-use

GEOMORPHOLOGICAL MAPPING

- can provide an indication of the distribution of subsurface materials, their structure and areas of possible mass movement, e.g. landslides.

Geomorphology of lake Tana basin (Ethiopia)



See for details http://www.tandfonline.com/doi/abs/10.1080/17445647.2013.801000

GEOTECHNICAL MAPPING

- involves the location and plotting on suitable scales of all data which assists in understanding the geotechnical conditions at that site

- **Regional mapping**
  - able to provide the regional geological understanding required for a dam project

- **Geotechnical mapping at and near the sites**

The maps show the following

- ground surface contours;
- geomorphic features, e.g. slope changes, areas of hummocky ground;
- geological surface features, e.g. areas of rock outcrops, boulders and soil;
- features of *in situ* rock, e.g. rock types and their boundaries, the nature, location and orientation of important geological defects such as sheared or crushed zones;
- groundwater features, e.g. springs, seepage, areas of swamp and vegetation indicating moist or wet ground;
- the location of tracks, roads, test pits and trenches, with summary logs of the soils and rocks exposed;
- the position of drill holes and geophysical traverse lines;
- the proposed works in outline including the full supply level of the proposed storage (regional geology)

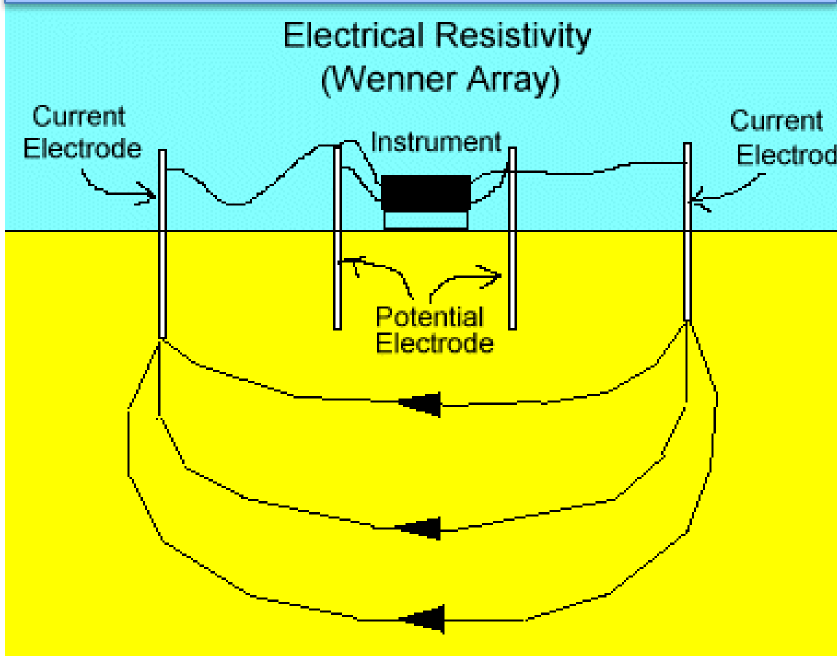
GEOPHYSICAL METHODS

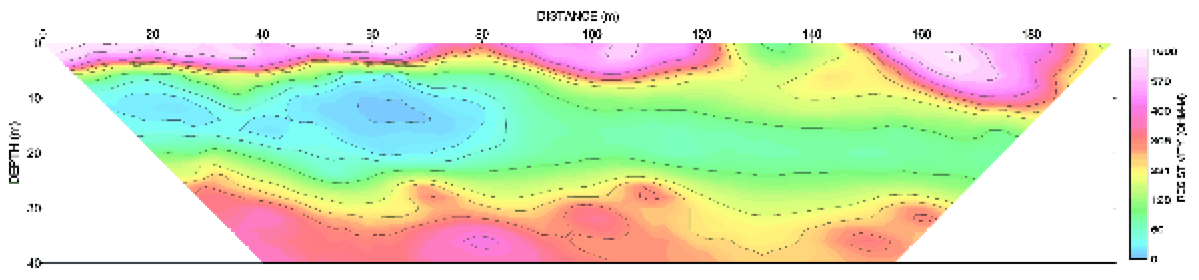
The *advantages* of the use of geophysical methods include:

- They are non-invasive and can be carried out from the surface or from existing boreholes;
- They can provide information on site conditions between data points e.g. boreholes;
- They may be able to identify local areas of concern which have no surface expression e.g. cavities;
- The surveys can usually be performed quickly and cover a relatively large area;
- Recent development of computer analysis and presentation of results (tomography) has assisted interpretation

Example
Resistivity Profiling or Imaging

- Based on generation of an artificial electric field in the earth by introduction of current through metal electrodes





Areas of investigation where the *correct use* of geophysical methods have provided valuable information include:

- Delineation of boundaries between the **underlying *in situ* rock and transported materials** such as alluvium, colluvium, glacial debris and landslide debris;
- Delineation of boundaries between **residual soil, weathered rock and fresh rock**;
- Delineation of boundaries between **sandy and clayey soils**;
- Location of **anomalous foundation features** e.g. igneous dykes, cavities, deeply weathered zones, fault zones, buried river channels;
- Assessment **of rippability, depth of foundation excavation, depth of cutoff excavation**,
- liquefaction potential;
- Location in existing structures of **seepage paths, low density zones, cavities**

Reading Assignment ---Different geophysical methods

TEST PITS AND TRENCHES

- are effective in providing information on subsurface conditions in dam foundations or existing dams, for the following reasons:

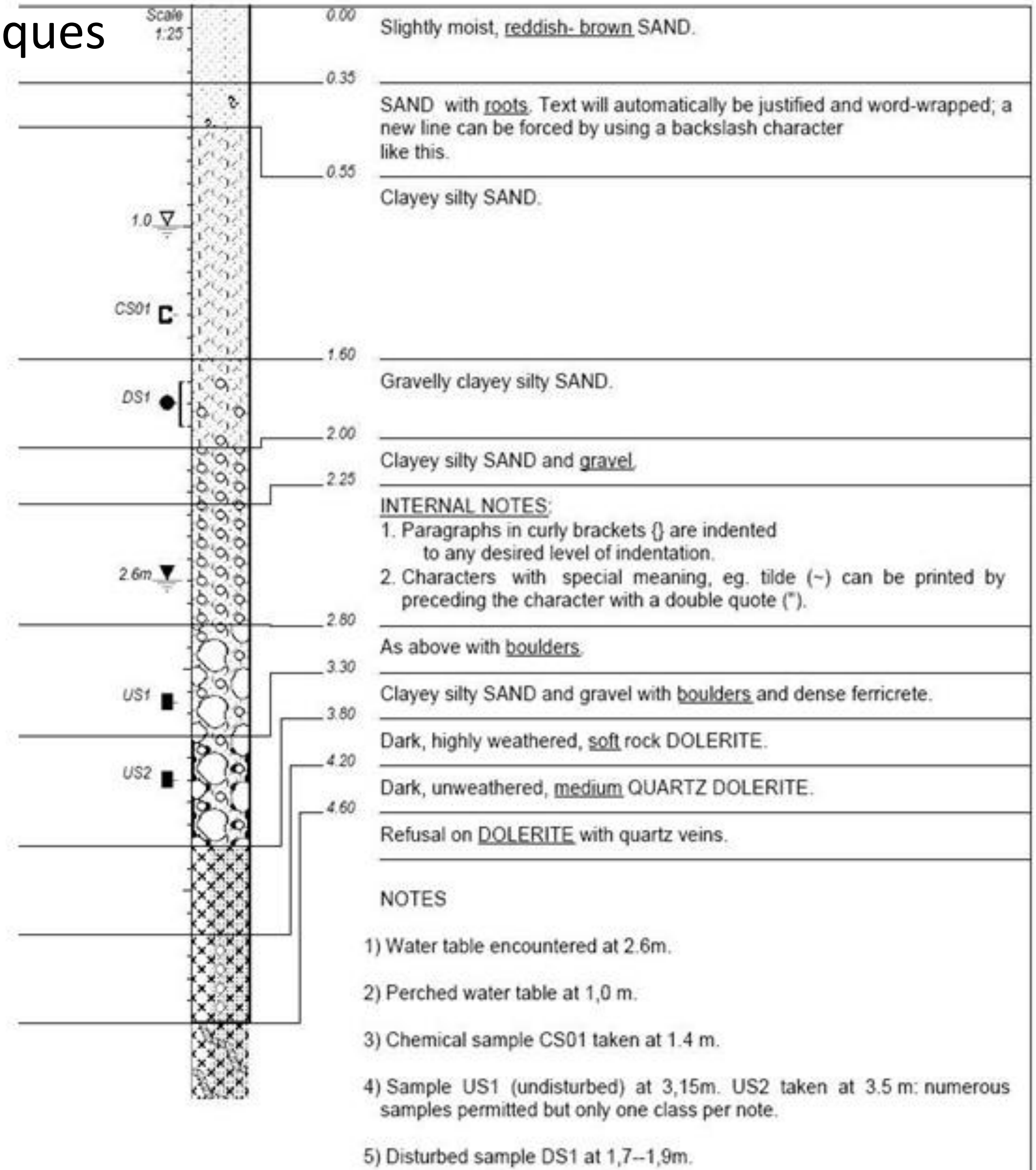
- They are relatively cheap and quick;
- The subsurface profile is clearly visible and can be logged and photographed;
- Material types, the nature and shape of their boundaries and structure can be observed and recorded in three dimensions;
- The absence or presence of groundwater is indicated and the sources of inflows can usually be observed and their flow rates recorded;
- Undisturbed samples can be collected;
- *In situ* tests can be carried out;
- The resistance to excavation provides some indication of excavation conditions likely to be met during construction.

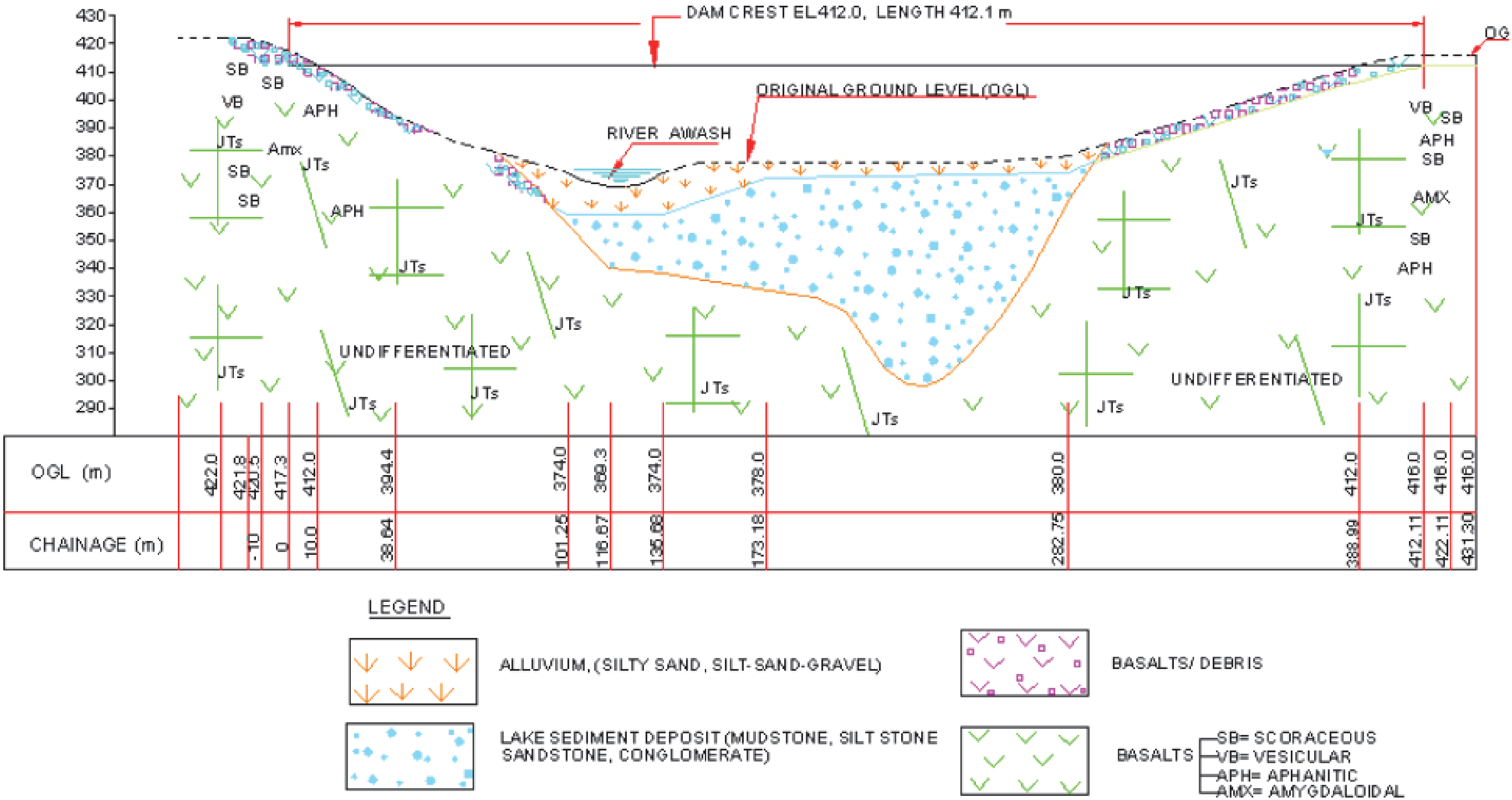
DRILL/BORE HOLES

The main objectives of drilling are to **extend the knowledge obtained from surface mapping, test pits and trenches** below the depth limitations of these methods and to:

- provide control for the interpretation of any geophysical investigations;
- provide samples from these greater depths;
- provide access for test equipment e.g. for measurement of water levels, pore pressures and permeability etc.

Borehole logging techniques





DAM CREST EL412.0, LENGTH 412.1 m
OGL
ORIGINAL GROUND LEVEL(OGL)
RIVER AWASH
UNDIFFERENTIATED
UNDIFFERENTIATED
SB
VB
APH
JTs
Amx
AMX
OGL (m)
CHAINAGE (m)
422.0
421.8
420.5
417.3
412.0
394.4
374.0
369.3
374.0
378.0
380.0
412.0
416.0
416.0
416.0
-10
0
10.0
38.64
101.25
116.67
135.68
173.18
282.75
388.99
412.11
422.11
431.30
LEGEND
ALLUVIUM, (SILTY SAND, SILT-SAND-GRAVEL)
LAKE SEDIMENT DEPOSIT (MUDSTONE, SILT STONE SANDSTONE, CONGLOMERATE)
BASALTS/ DEBRIS
BASALTS
SB= SCORACEOUS
VB= VESICULAR
APH= APHANITIC
AMX= AMYGDALOIDAL

Important points to consider when conducting subsurface EXPLORATION of Foundations of dams

Where?

- ✓ along the centerline of the dam
- ✓ at the proposed service and auxiliary spillway locations
- ✓ other critical areas

Depth?

- ✓ sufficient to locate and determine the extent and properties of all soil and rock strata (performance of dams, reservoir, appurtenant structures)
- ✓ Geologic bulletins, soil survey maps, groundwater resources bulletins, etc., may aid the designer in determining the scope of the exploration program needed and interpreting the results of the program.

- ✓ small low-hazard dams:
  - at least three explorations should be made along the centerline of the dam (one in the deepest part of the depression across which the dam will be built and one on each side)
- At least one exploration should be made at the proposed auxiliary spillway location.
- For small low-hazard dams, to be built on a foundation known from the geology of the area to be essentially incompressible and impervious to a great depth, the minimum depth of explorations should be 1.5 m unless bedrock is encountered above this depth.
- In other cases the minimum depth of explorations should be 3 m, with one or more borings extending to a depth equal to the proposed height of the dam.
- If it is proposed to excavate in the reservoir area, the possibility of exposing pervious foundation layers should be investigated by explorations or a review of the geology of the area.

- ✓ If rock is encountered in explorations, acceptable procedures, such as coring, test pits, or geologic information, should be used to verify whether or not it is bedrock.

- ✓ Sufficient subsurface explorations should be made to verify the suitability of encountered rock for use as a foundation and/or construction material.

- ✓ Testing of the rock materials shall ascertain its strength, compressibility, and resistance to degradation, and its ability to safely withstand the loads expected to be imposed upon it by the proposed project.

- ✓ Soils encountered in explorations should be described accurately and preferably classified in accordance with the Unified Soil Classification System.

CATEGORIES AND CAUSES FOR FAILURE OF EARTH DAMS

SEEPAGE FAILURE

- Piping through dam body
- Piping through foundation
- Sloughing of d/s toe
- High pore pressure

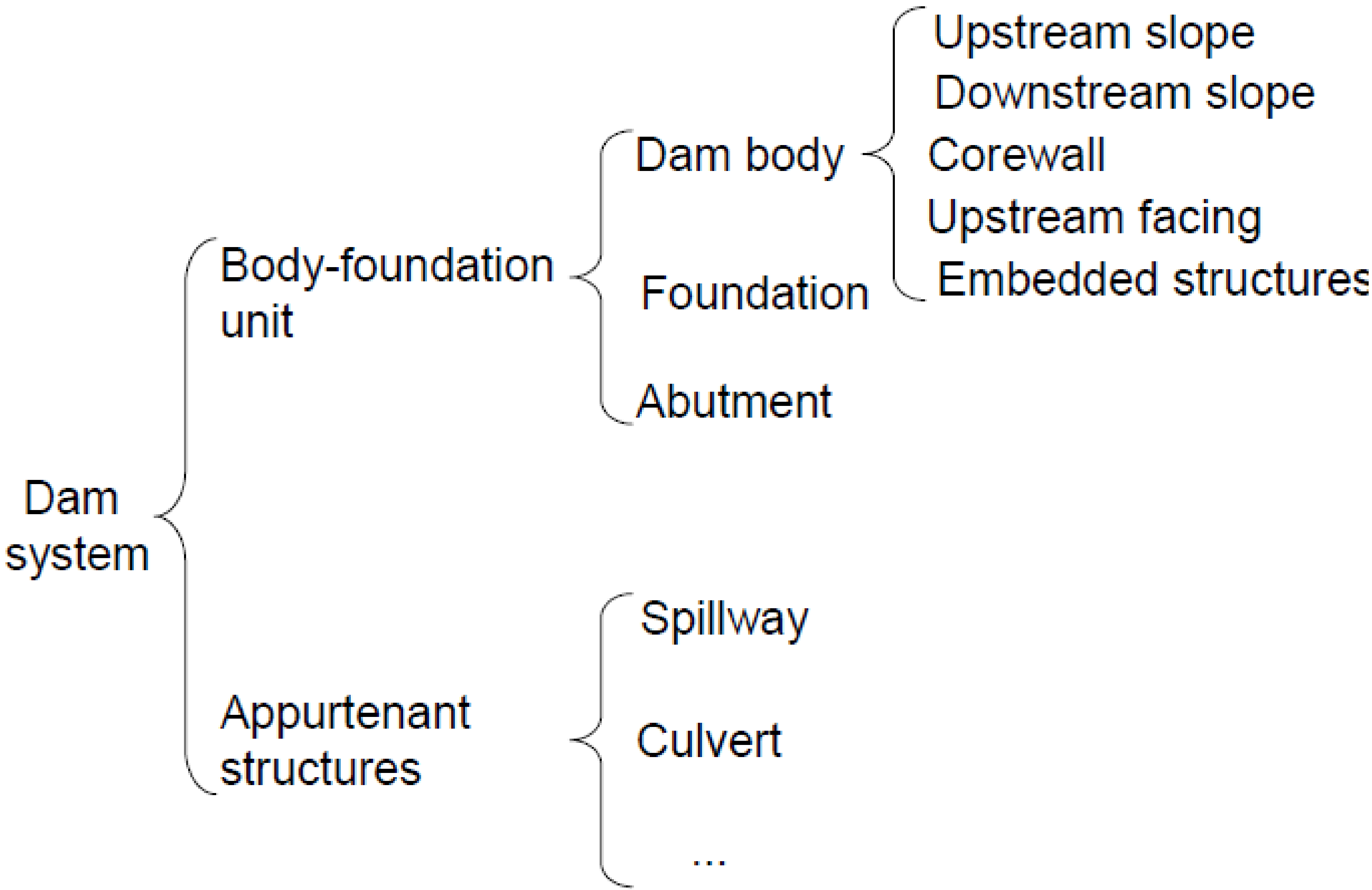
STRUCTURAL FAILURE

- **Slip of embankment**
- **Wave action**
- **Foundation slides**
- **Faulty construction**
- **Defective materials**
- **Improper maintenance**
- **Earthquakes**
- **Liquefaction of foundations**
- **Shear slides**
- **Failure of slope protection**
- **Burrowing animals**
- **Faulty operation**

CAUSES OF FAILURE

Foundation Failure (mainly piping)	40%
Inadequate spillway (overtopping)	23%
Poor construction	12%
Uneven settlement	10%
High pore pressure	5%
Acts of war	3%
Embankment slips	2%
Defective materials	2%
Incorrect operation	2%
Earthquake	1%
Total	100%

Potential locations at risk in a dam system



FOUNDATION DESIGN/PREPARATION

Foundation: includes both the valley floor and the abutments.

Essential requirements:

- **Provide stable support for the embankment under all conditions of saturation and loading, and**
- **Provide sufficient resistance to seepage to prevent excessive loss of water.**

" No structure is better than its foundation"

- Foundation parts:

  General Foundation: is the foundation beneath the bulk of the embankment.

  Cutoff Foundation: is the foundation under the earthfill core of an earth and rockfill dam, or the plinth of a concrete face rockfill dam.

Safe design of dam

Requires:

- Ample knowledge of the weak geologic features present at site (position, orientation and properties)
- Adequate analysis of their potential effect on dam performance (using soil mechanics, rock mechanics and structural analysis)
- Adequate design with adequate foundation treatment (properly constructed)

Three main classes of foundation according to their predominant characteristics:

I. **Foundations of rock**

II. **Foundations of coarse-grained material (sand and gravel)**

III. **Foundations of fine-grained material (silt and clay)**

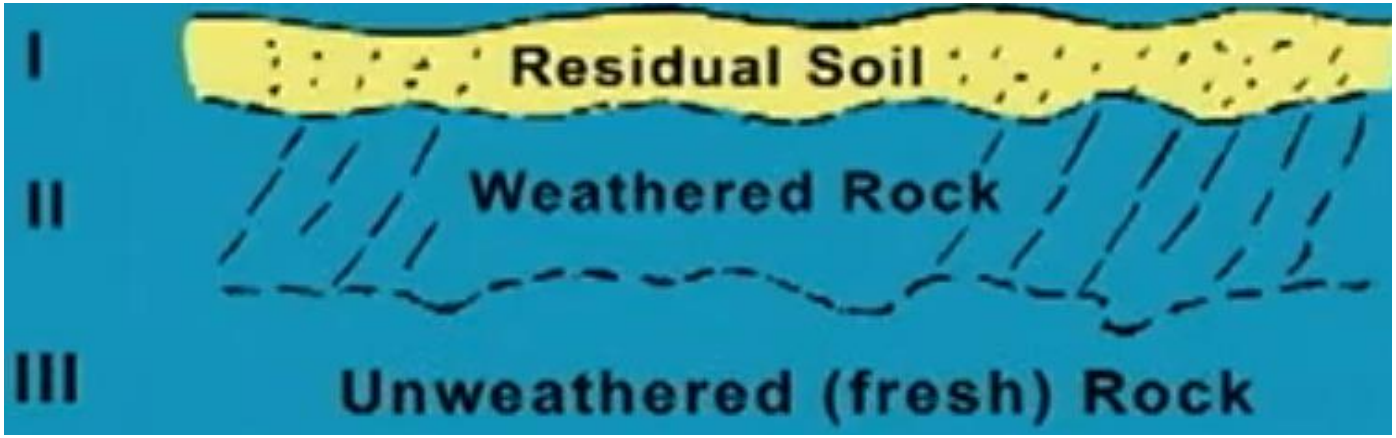
Rock Foundations

- Generally considered to be the more competent type of foundation and usually do not present any problem for small dams

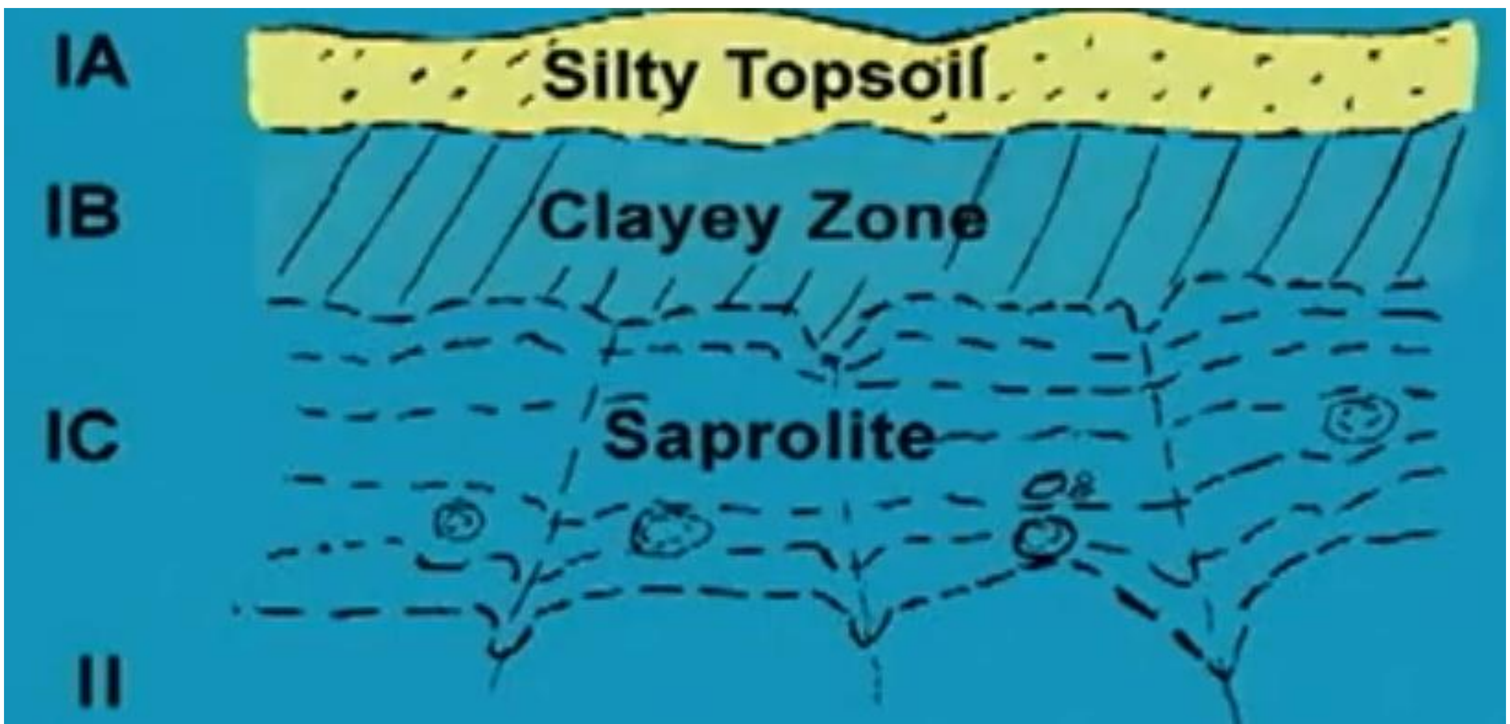
Various types of weak geological features/defects that can be encountered are:

1. Weathered rock profile
2. Faults
3. Thin Shear Zones
4. Shattered/Highly jointed rock- Master joints
5. Folds
6. Buried Channels
7. Jointing pattern of the rock mass
8. Caverns/Cavities or Karstic limestone
9. Springs etc.

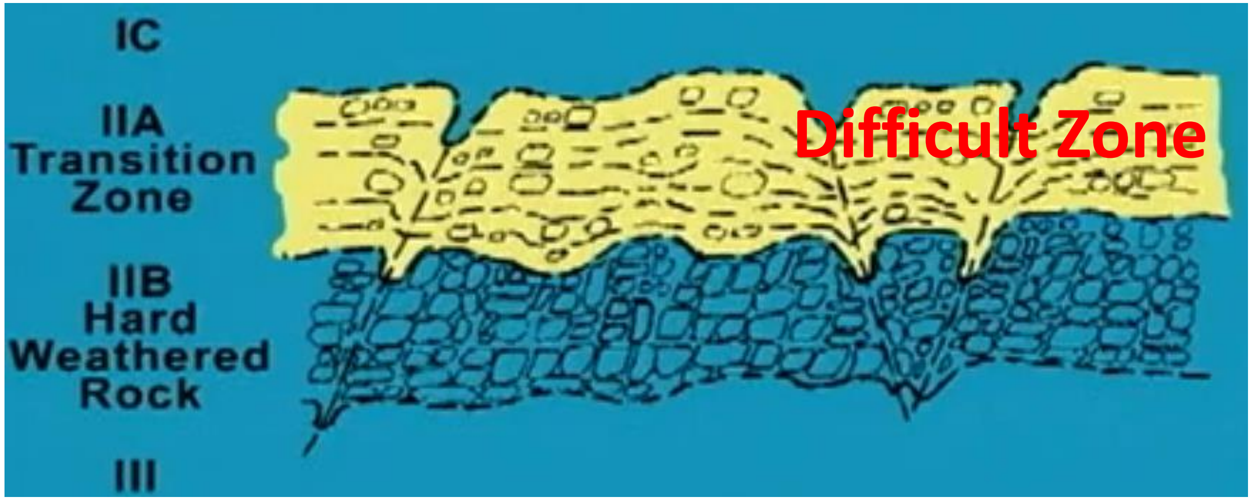
Generalized profile of weathering

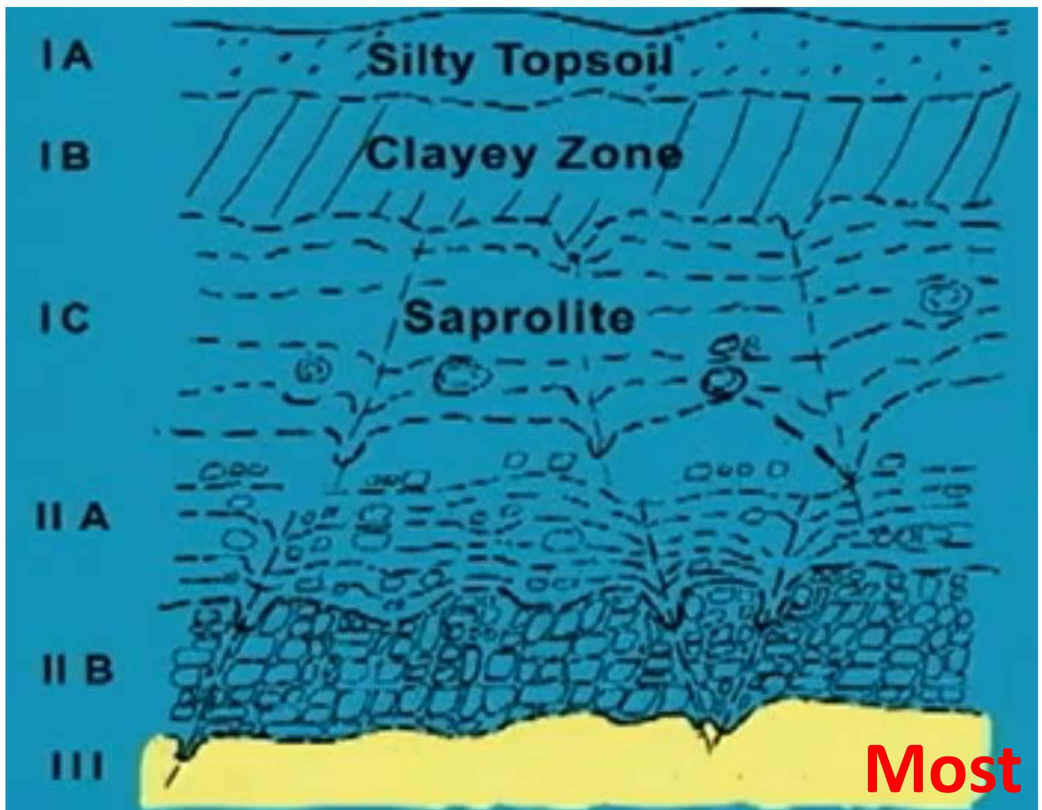


Subdivisions of Zone I

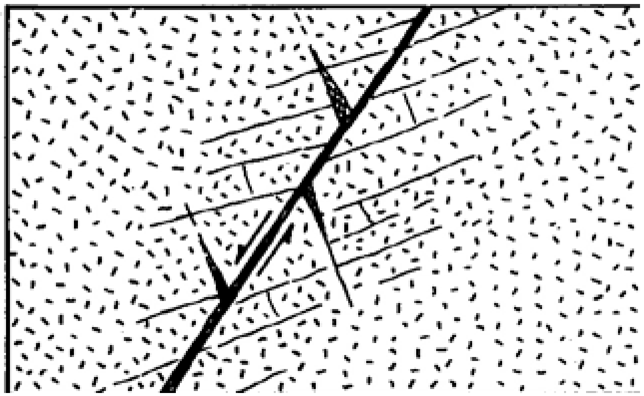


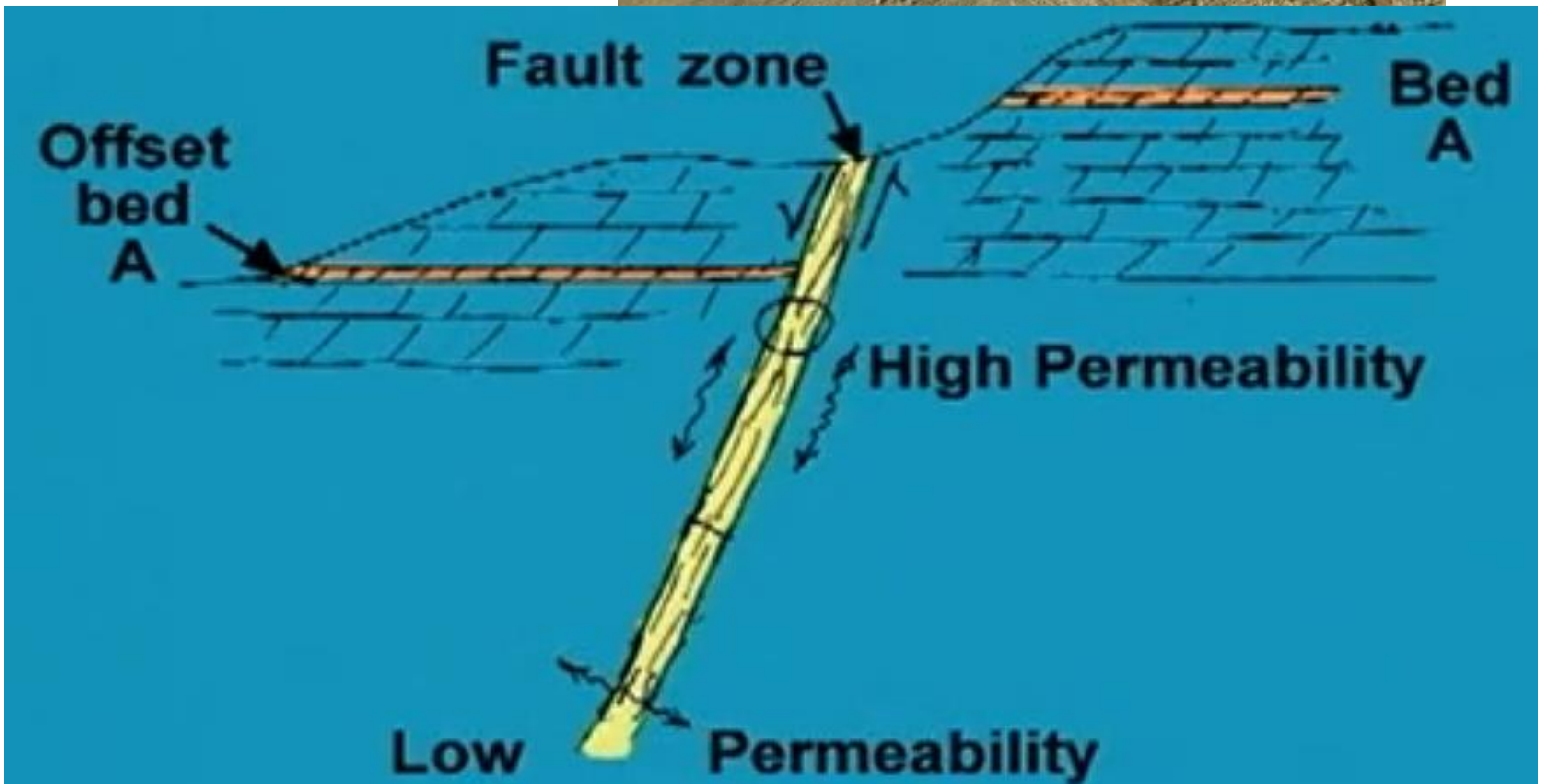
Subdivisions of Zone II





Fault

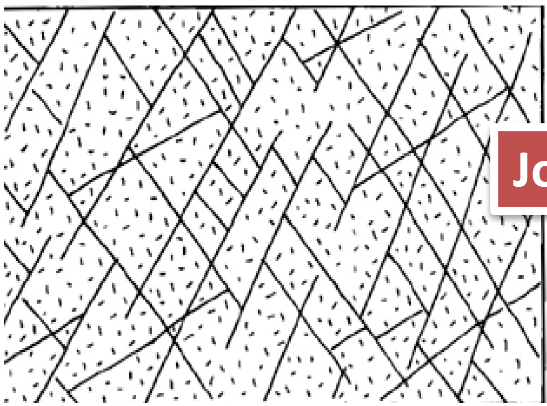




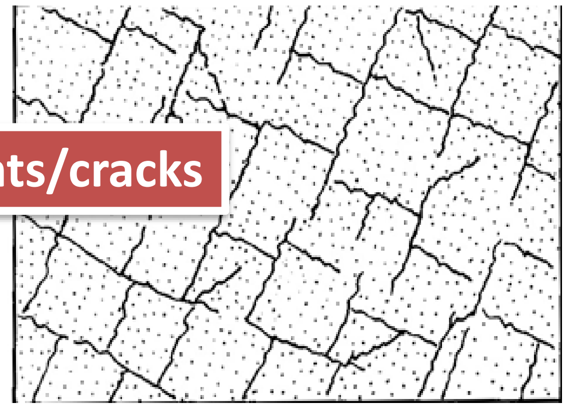
Fault zone
Offset bed A
Bed A
High Permeability
Low
Permeability

Joints/cracks, folds and thin shear bedding

- Tensional “gash joints” and tight compressed “shear joints” have developed along a fault



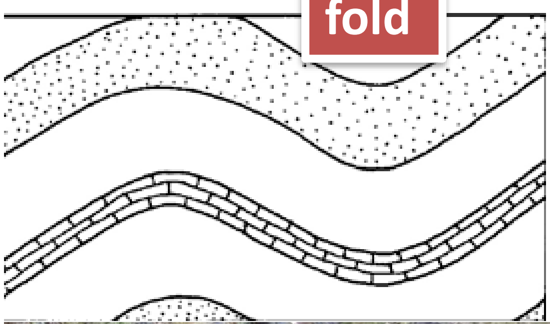
Joints/cracks



Rough surfaced tension joints

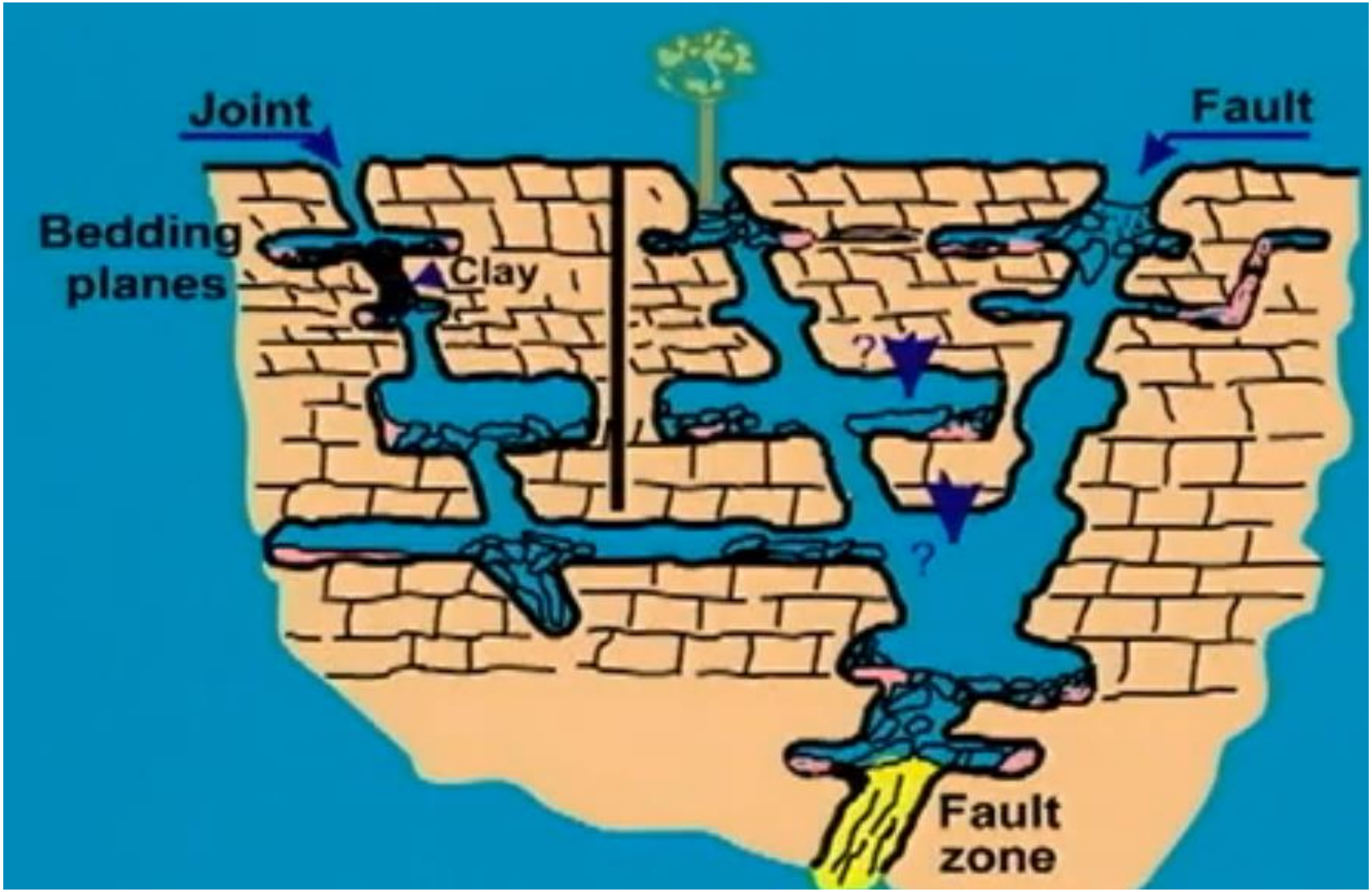
Smooth surfaced, intersecting shear joints

fold



Bedding

Caverns/Cavities or Karstic limestone



Methods of Treating Rock Foundations

- Removal and replacement of upper fractured rock
- Grouting
- Impervious upstream blanket

Factors affecting degree of foundation treatment:

- type of dam

- height of dam

- topography of the dam site

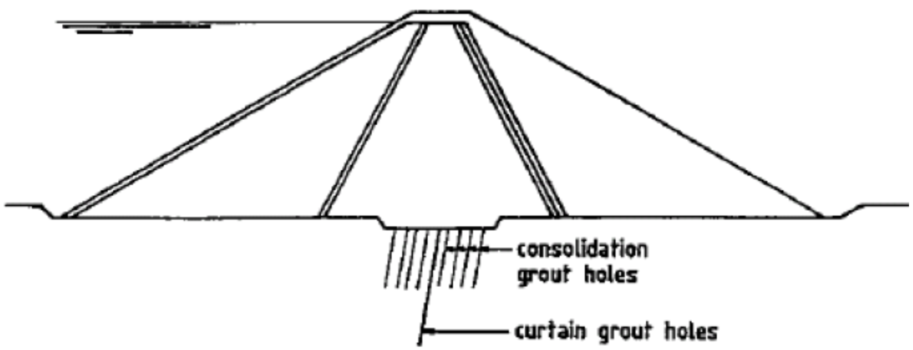
- erodibility, strength, permeability, compressibility of the foundation material

- groundwater inflows to excavations

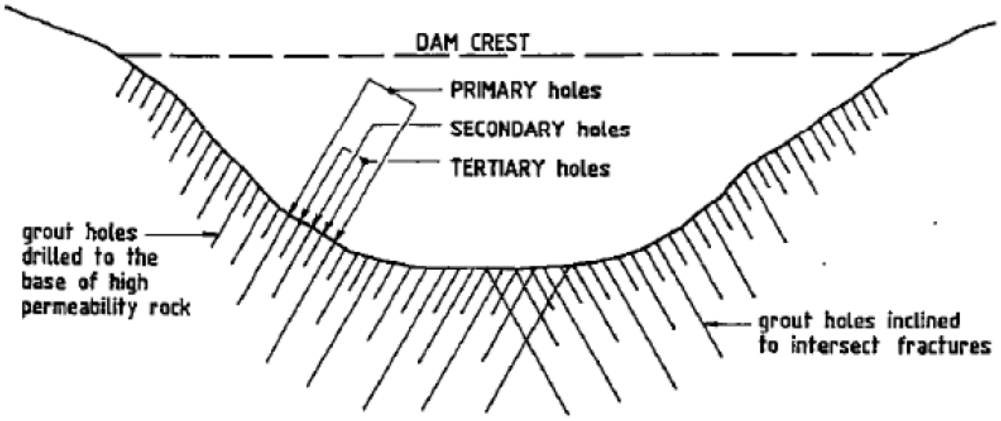
- climate and river flows

Foundation Grouting

- The foundation of dams more than 15 m high built on rock, are treated by grouting.
- Grouting consists of drilling a line or lines of holes from the cutoff level of the dam into the dam foundation, and forcing cement slurry or chemicals under pressure into the rock defects



SECTION THROUGH DAM



SECTION ACROSS RIVER

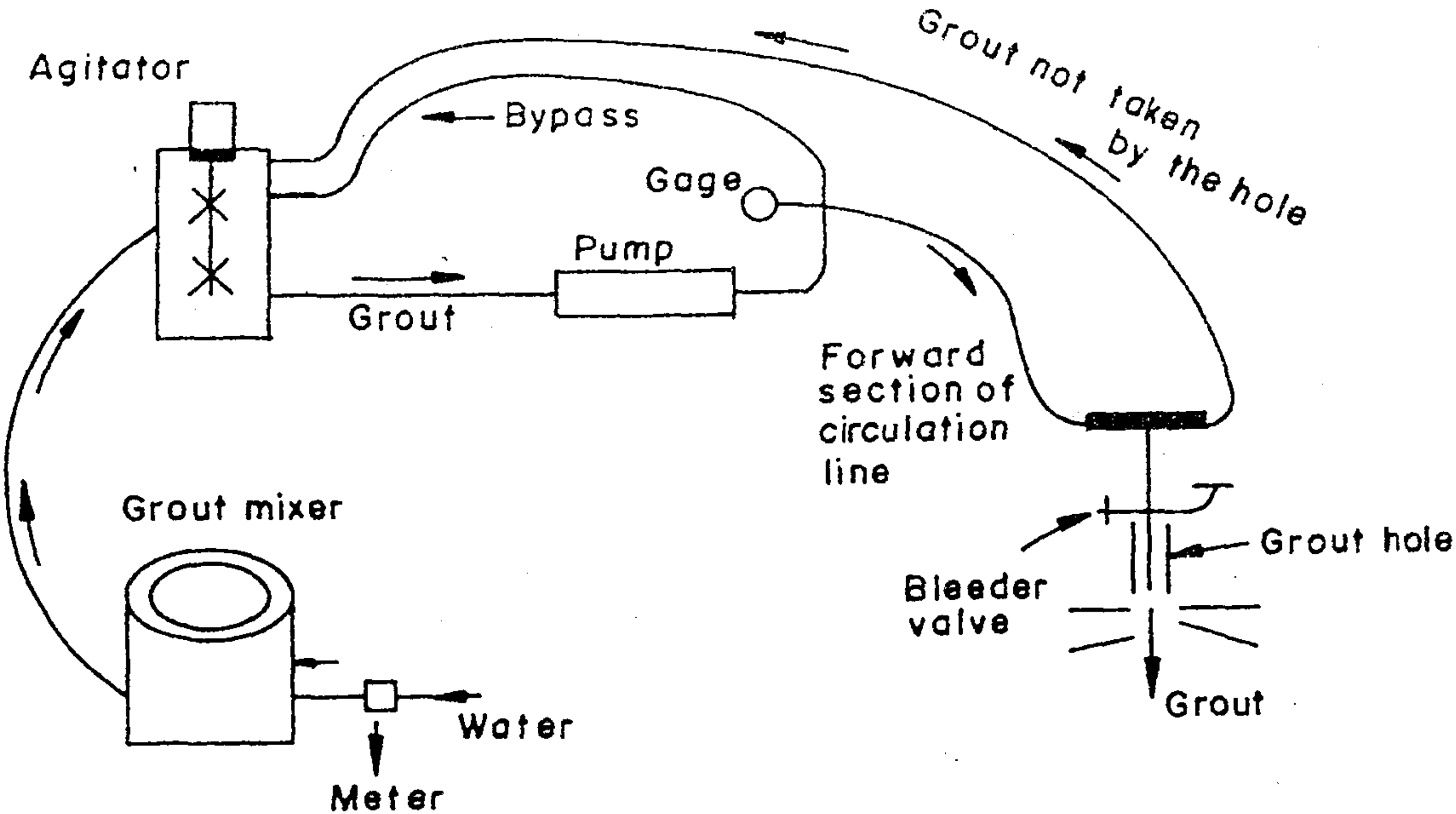
The grouting is carried out to:

- Reduce leakage through the dam foundation, i.e. through the defects;
- Reduce seepage erosion potential;
- Reduce uplift pressures (under concrete gravity dams when used in conjunction with drain holes);
- Reduce settlements in the foundation (for concrete gravity, buttress and arch dams).

Investigation methods

- Drilling and direct inspection to accurately locate and determine local conditions;
- Taking coring samples for laboratory tests;
- Drilling with drilling data recording to locate fissured zones, voids and the interface between structure and surrounding ground;
- Borehole logging
- Non-destructive geophysical investigations (seismic resistivity);
- Water testing (constant head or falling head tests conducted in borehole;
- Underground flow & temperature measurements;
- Pumping test to assessment of initial hydraulic conditions

Equipment for grouting



Typical equipment layout for rock grouting

Foundation Grouting (ctd.)

WATER TESTING IN GROUT HOLES (Lugeon or packer test)

- Enable decisions about the grouting
- The Lugeon unit of permeability is the most popular and relevant unit for grouting purposes
- **A Lugeon is a unit devised to quantify the water permeability of bedrock and the hydraulic conductivity resulting from fractures (1 *Lugeon* = approx. 10^{-7} m/s)**

1 lugeon unit = 1 litre of water taken per meter of test length, per minute, at 10 bars pressure (150 psi approx)

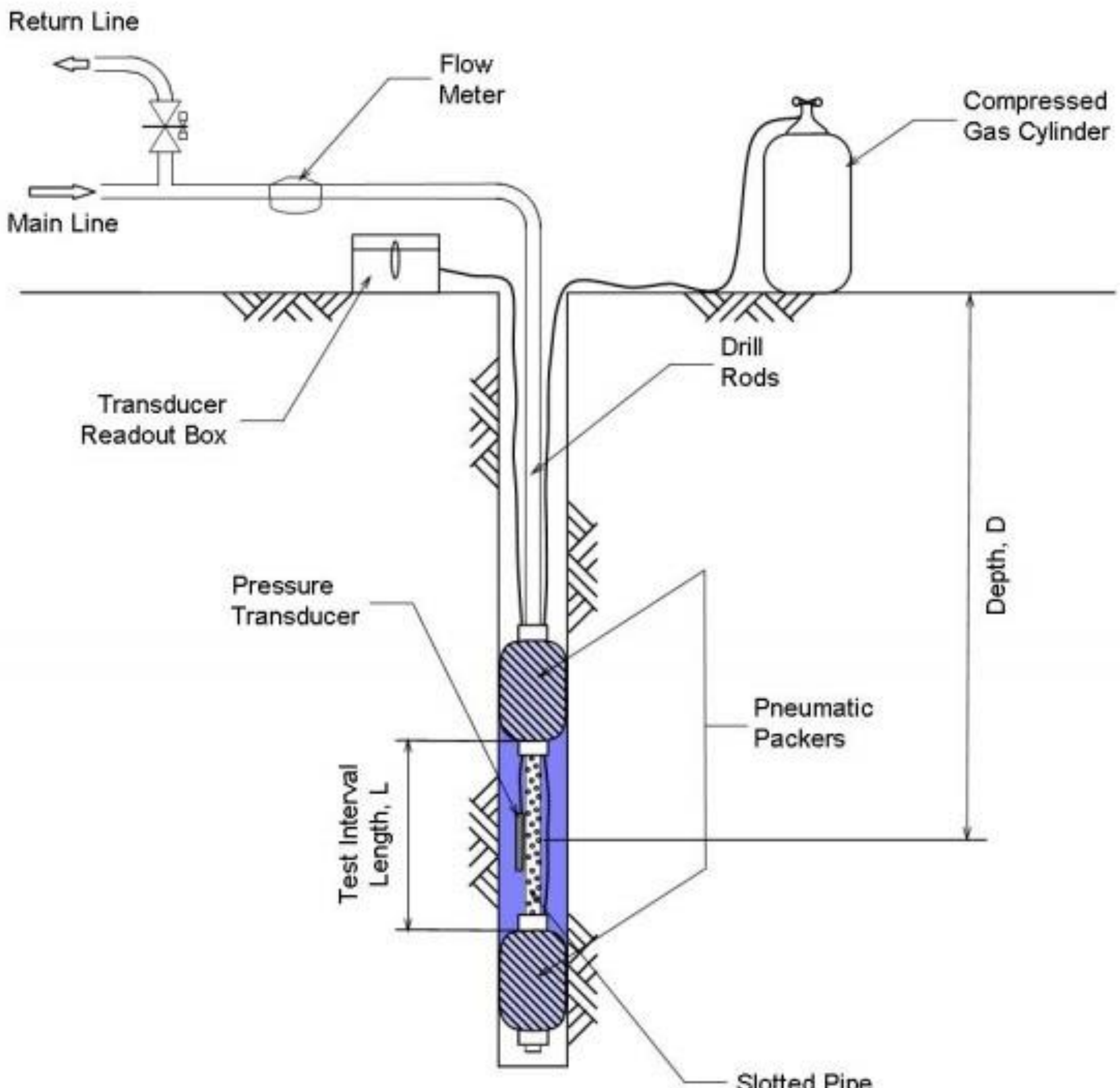
To give a sense of proportion for the unit

- 1 lugeon unit is the type of permeability where grouting is hardly necessary.
- 10 lugeons warrants grouting for most seepage reduction jobs.
- 100 lugeons is the type of permeability met in heavily jointed sites with relatively open joints or in sparsely cracked foundations where joints are very wide open.

Tests at Other Pressures

Lugeon units = litres/metre/minute x 10 (bars) /actual pressure (bars)

Lugeon or packer test



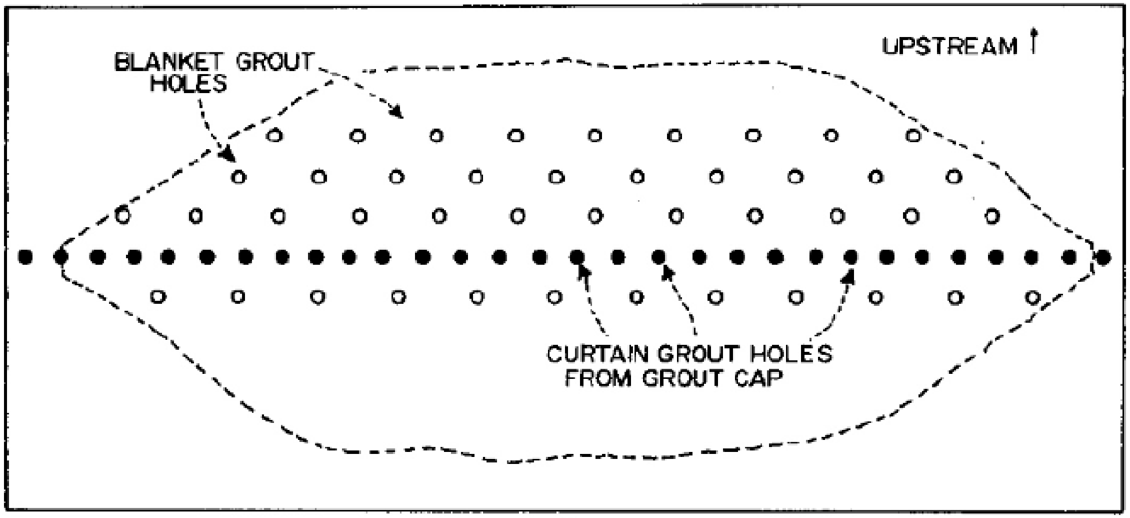
Foundation Grouting (ctd.)

- Foundation Grouting takes two forms:

-Curtain Grouting.

-Consolidation Grouting (Blanket grouting)

- Curtain Grouting: is designed to create a narrow barrier (or curtain) through an area of high permeability. It usually consists of a single row of grout holes which are drilled and grouted to the base of the impermeable rock, or to such depths that acceptable hydraulic gradient are achieved.



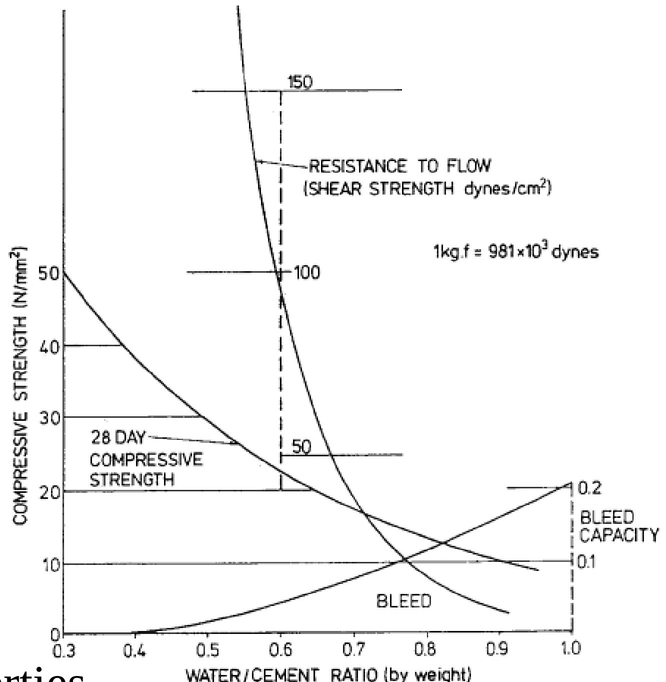
- Consolidation or Blanket Grouting: for embankment dams, it is designed to give intensive grouting of the upper layer of more fractured rock in the vicinity of the dam core. It is usually restricted to the upper 5 to 15 m.

Foundation Grouting (ctd.)

Grouting Scheme Design

i. Grouting material

- Most foundation grouting uses **cement** grout: Portland cement mixed with water in a high speed mixer to a water-cement ratio (mass water/mass cement) of between 0.5 and 5 to form a slurry
- **Chemicals** tend to be more expensive so are only used where cement grout would not be successful
- Single grout mix throughout



Effect of water content on grout properties

- An important requirement for the selection of a grout is that its particles be substantially smaller than the voids to be filled.
- This is determined by the groutability ratio, N , expressed by the equation

$$N = \frac{D_{15}}{D_{85}}$$

where D15 is the 15 percent finer grain size of the medium to be grouted and
D 85 is the 85 percent finer grain size of the grout

- N generally should be greater than 25 but in some cases may be as low as 15, depending upon physical properties of the grout materials

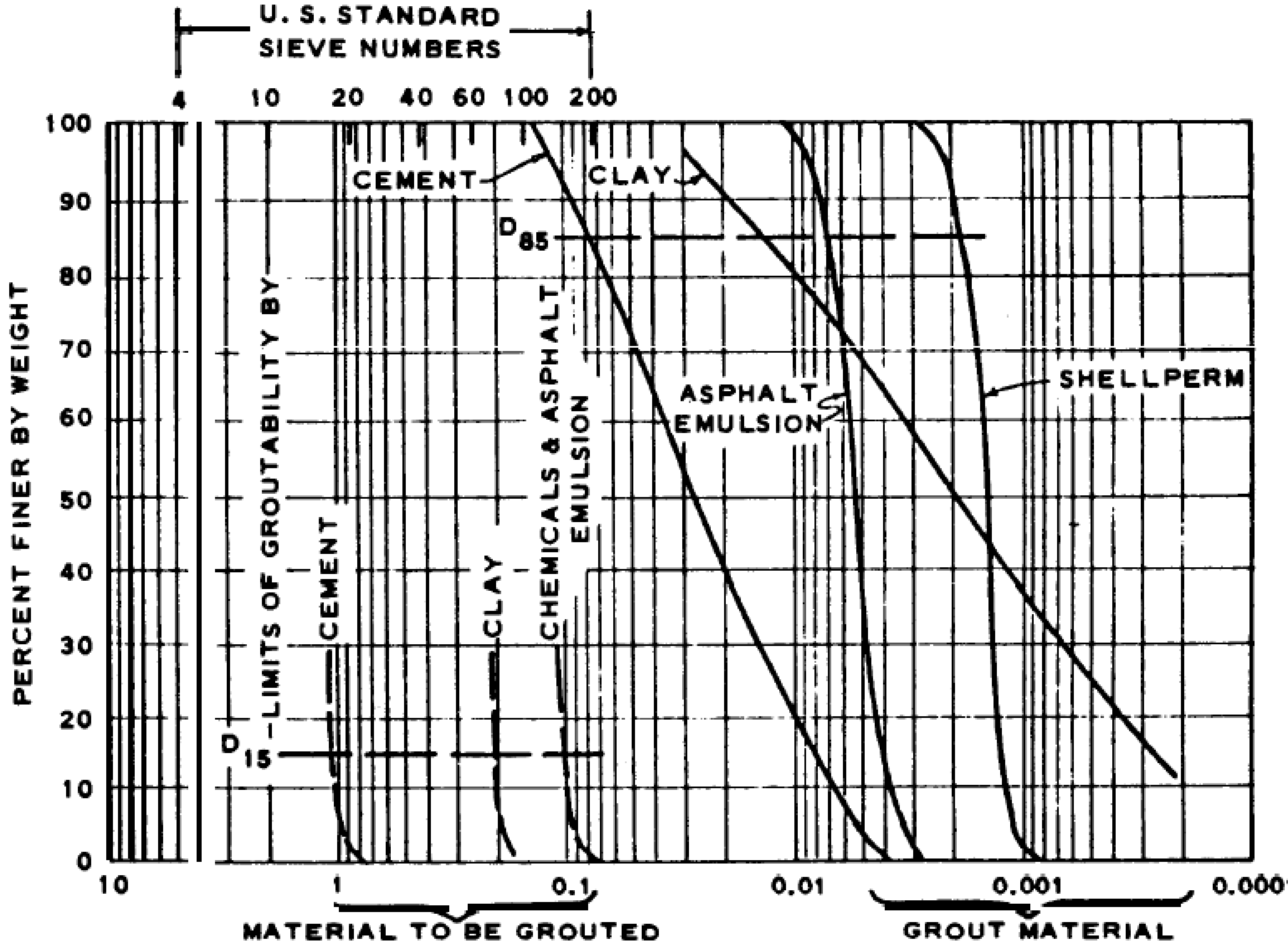


Figure : a graphic interpretation of the above equation. It shows (a) typical grainsize curves for Portland cement, Boston blue clay, ordinary asphalt emulsion, and special Shellperm asphalt emulsion, and (b) the lower limits (D15) of sand groutable by the above-described grout materials

Recommended water cement ratios:

- **Starting Mix:**
  - **2:1 most sites**
  - **3:1 for rock <5 Lugeons**
  - **1:1 for rock >30 Lugeons**
  - **0.8:1 for very high losses**
- **Thicken the mix::**
  - **to deal with severe leaks;**
  - **after 1.5 hours on the one mix with continued take (except for 1:1 and thicker mixes);**
  - **if hole is taking grout fast, e.g. > 500 liters in 15 minutes.**

Foundation Grouting (ctd.)

ii. Depth:

- According to ICOLD, a typical grout curtain varies in depth from 0.35 to 0.75H. (where H is the height of the reservoir above the top of the grout curtain in a specific location).

- In addition to this recommendation, grouting should be carried up to the rock mass of relatively low permeability

Foundation Grouting (ctd.)

iii. Grouting Pattern and Grout Hole Spacing:

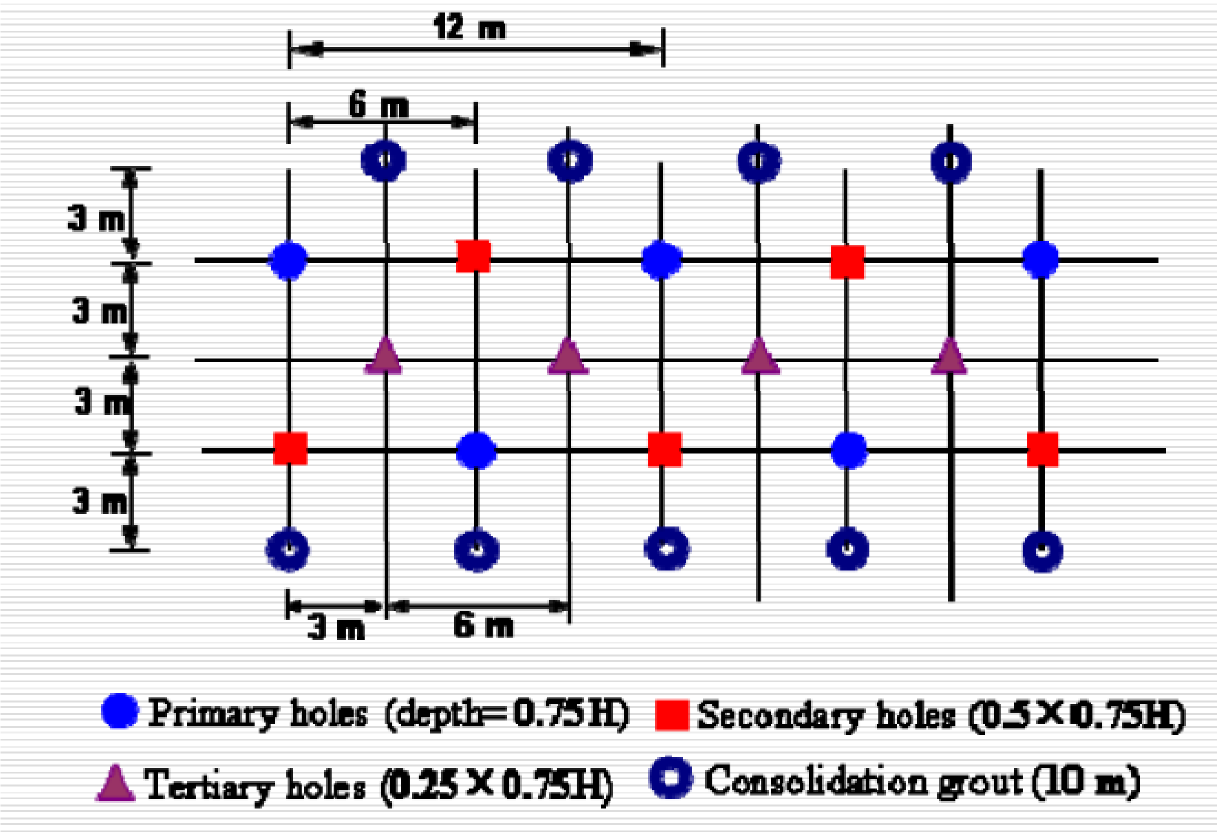
-Grout moves more rapidly into larger joints, frequently leaving the smaller ones almost unfilled. Consequently, grouting along a single row often can not be depended on to ensure an effective cutoff of seepage

- To ensure a highly efficient cutoff, a multiple row approach should be used. A three-row pattern seems to give generally good results (Janson, 1988).

- common practice to drill all grout holes perpendicular to the average slope of the ground surface

Foundation Grouting (ctd.)

- The **spacing** of the initial (primary) holes in a grout curtain ordinarily is based on an assessment or assumption that grout injected in any one of them is unlikely to penetrate to the nearest one on either side.
- In general, a primary spacing on the order of 6 to 12 m in each curtain row is selected. Grouting specifications sometimes provide for drilling widely spaced, cored primary holes for exploration and commonly provide for a maximum spacing of grout holes of about 3m.

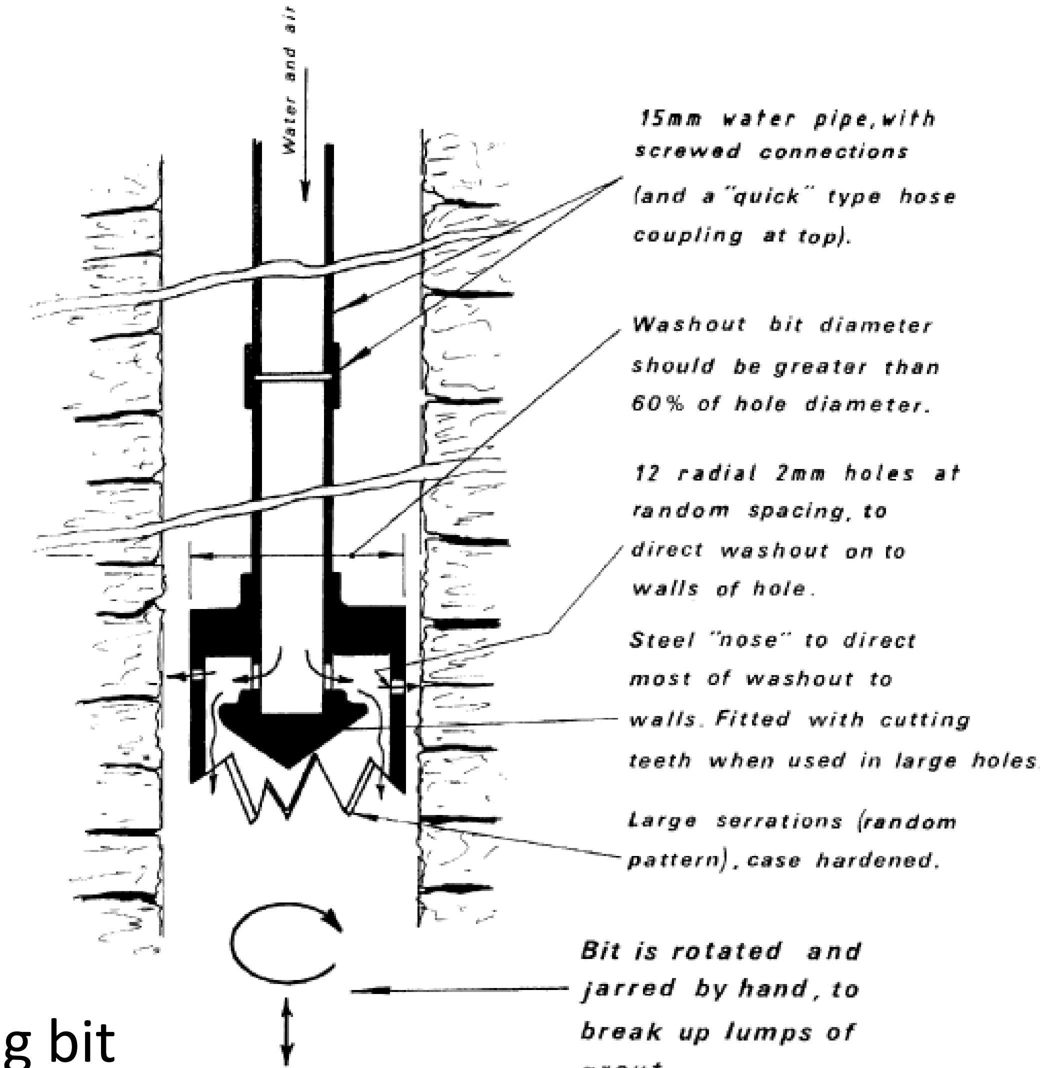


The principles of "closure"

- **successive halving of hole spacing from primary to secondary to tertiary holes etc.**

Foundation Grouting (ctd.)

Washing of the grout hole before grouting is essential to remove cuttings which have clogged the fractures

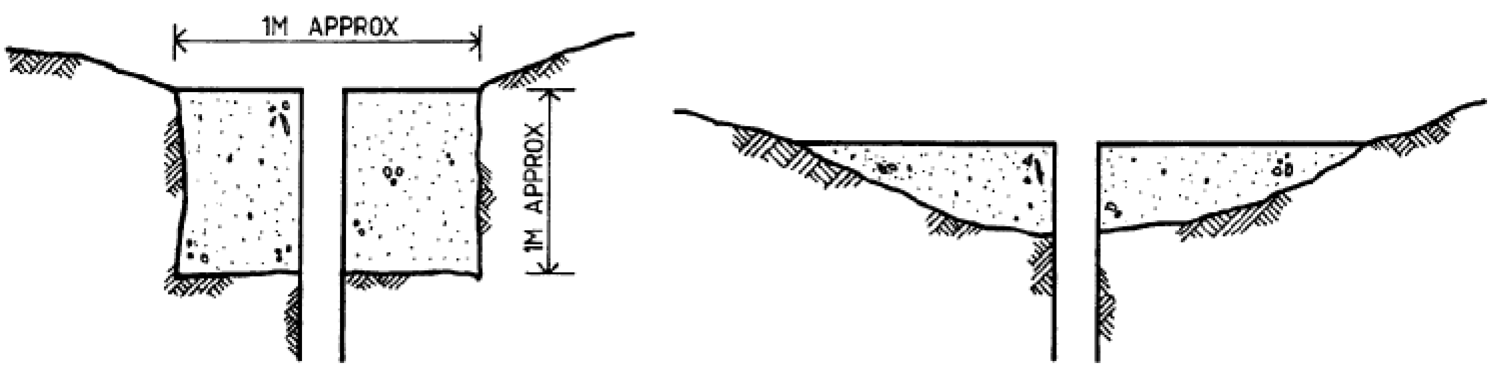


Grout hole washing bit

Grout Caps and Cutoff Walls

The grout cap or cutoff wall will serve two purposes

- It will form a positive cutoff in a shallow zone
- It will provide a firm anchorage for grout nipples or grout packers or both
- prevents excessive leakage to the surface and generally facilitates grouting.
- It also prevents damage to the cutoff surface by construction



(a) IDEAL SHAPE (b) POOR SHAPE

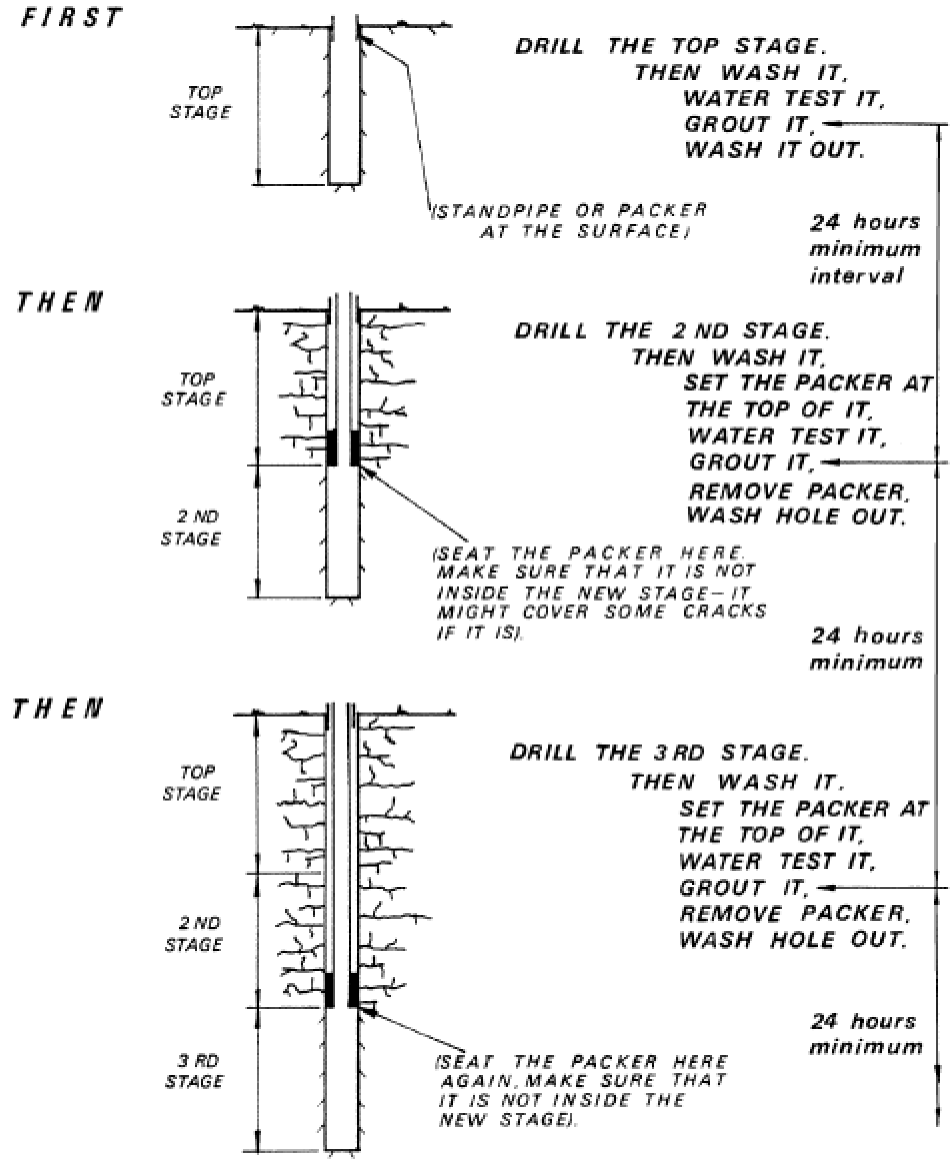
Foundation Grouting (ctd.)

iv. Staging of grouting

- Commonly grout stages will be 5m to 8m but may be increased in length lower in the foundation

Stage	Depth Range (m)
1	0 to 8
2	8 to 16
3	16 to 30
4	30 to 50

After Houlsby (1977)



V. Distance grout will penetrate

The distance to which the grout will penetrate is dependent on the **fracture width, grout pressure and viscosity and the time taken in grouting.** If grouting continues for sufficient time, the limit of penetration is determined by the yield point stress. Lombardi (1985) showed that

$$R_{max} = \frac{P_{max} a}{C}$$

where R_{max} = maximum radius of penetration (m); a = half width of the fracture (m); C = yield point stress (kPa); P_{max} = grouting pressure (kPa).

	Fracture spacing		
Lugeon value	1 m	0.50 m	0.25 m
100	20	12	4
50	12	3	2
20	3	1.5	1
10	2	1	NP
5	1	NP	NP
1	NP	NP	

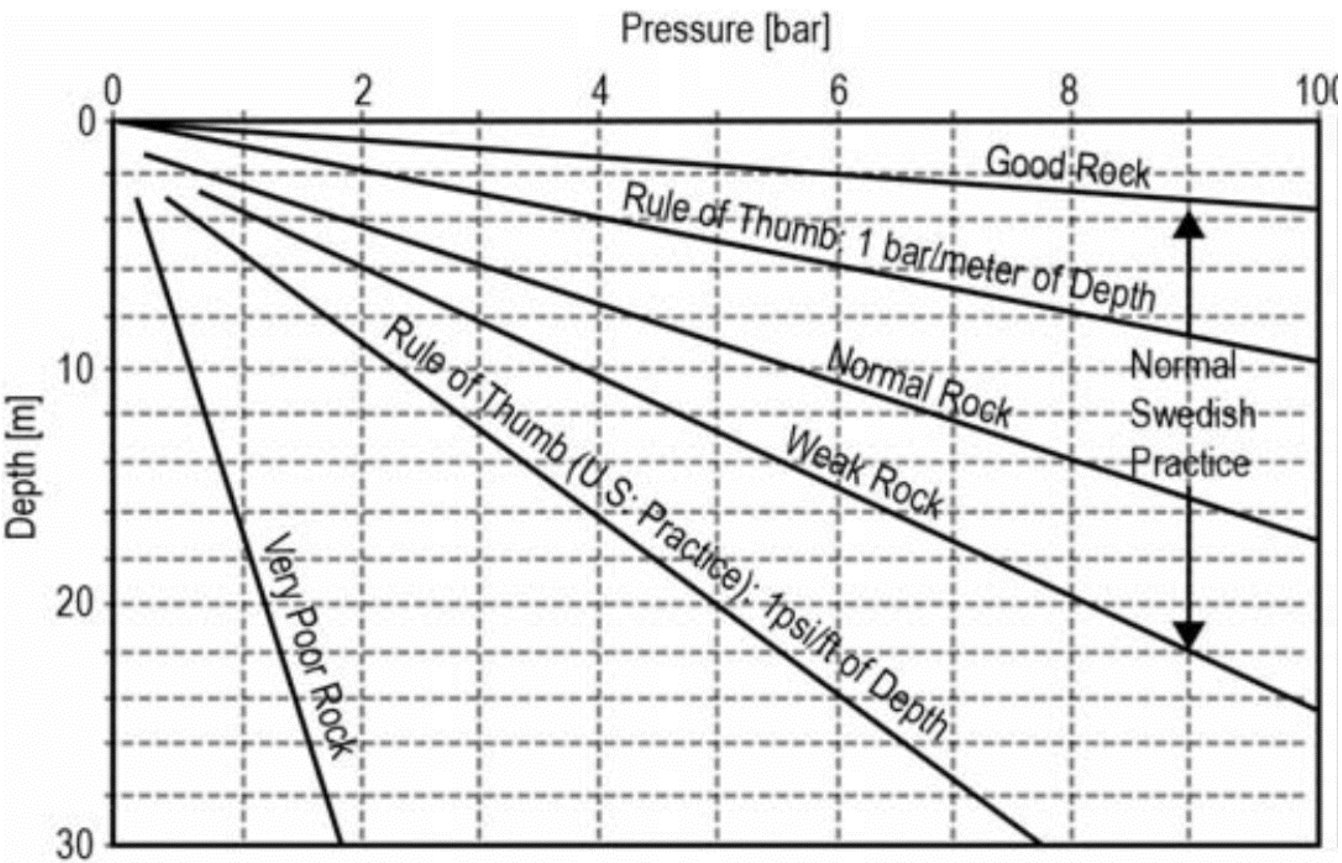
Approximate penetration from the borehole of cement grout in fractures (m)

vi. Grout pressures

- the maximum pressures at the base of the stage being grouted are given by:

$$P_B = \alpha d$$

where P_B = pressure at base of hole in kPa; α = factor depending on rock conditions; $\approx$70 for "sound" rock; $\approx$50 for "average" rock; $\approx$25 to 35 for "weak" rock; d = depth of bottom of stage below ground surface in metres.



Grouting pressure according to practice in Sweden and the USA (Weaver 1991).

Foundation Grouting (ctd.)

Example : Tendaho Dam

Salient features

Type of Dam--------Zoned earth fill dam

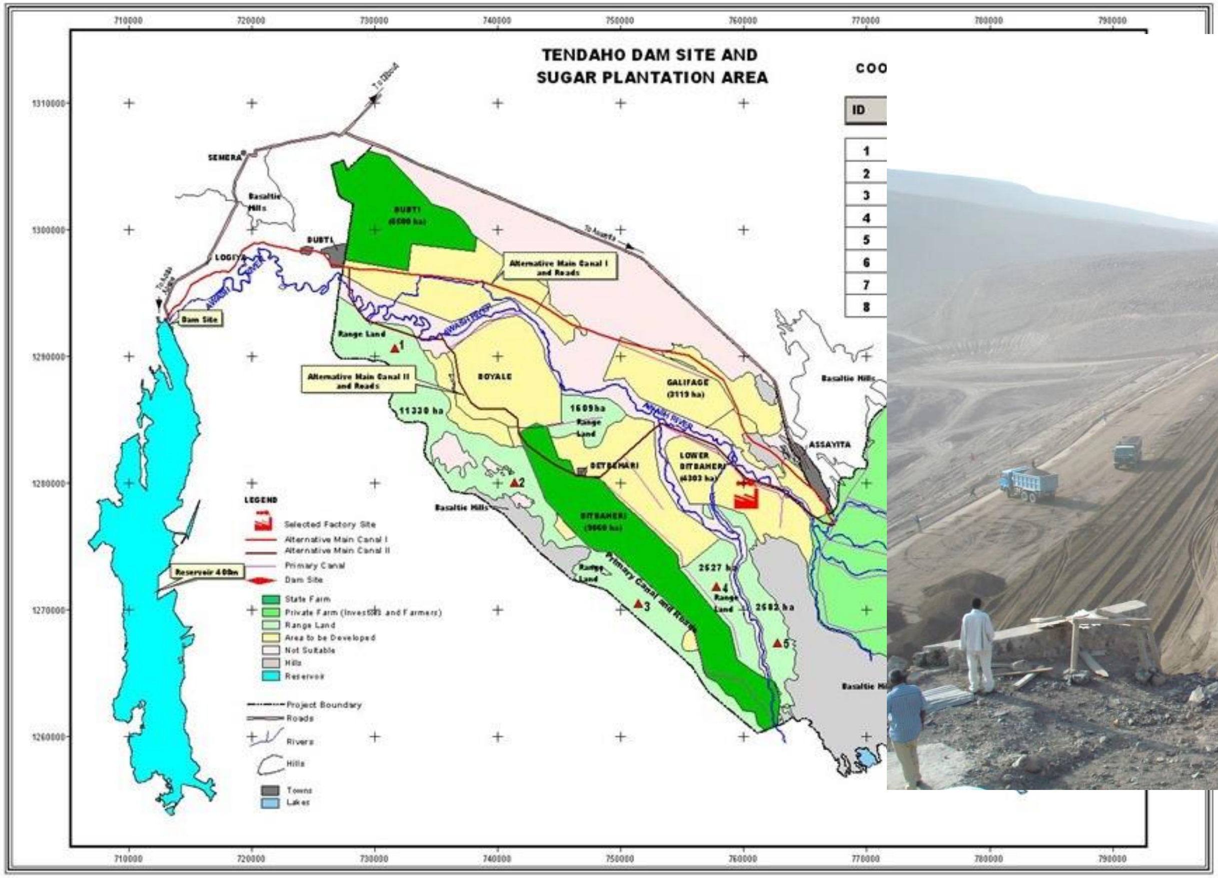
Crest Length -------421m

Max.height at the deepest section----53m

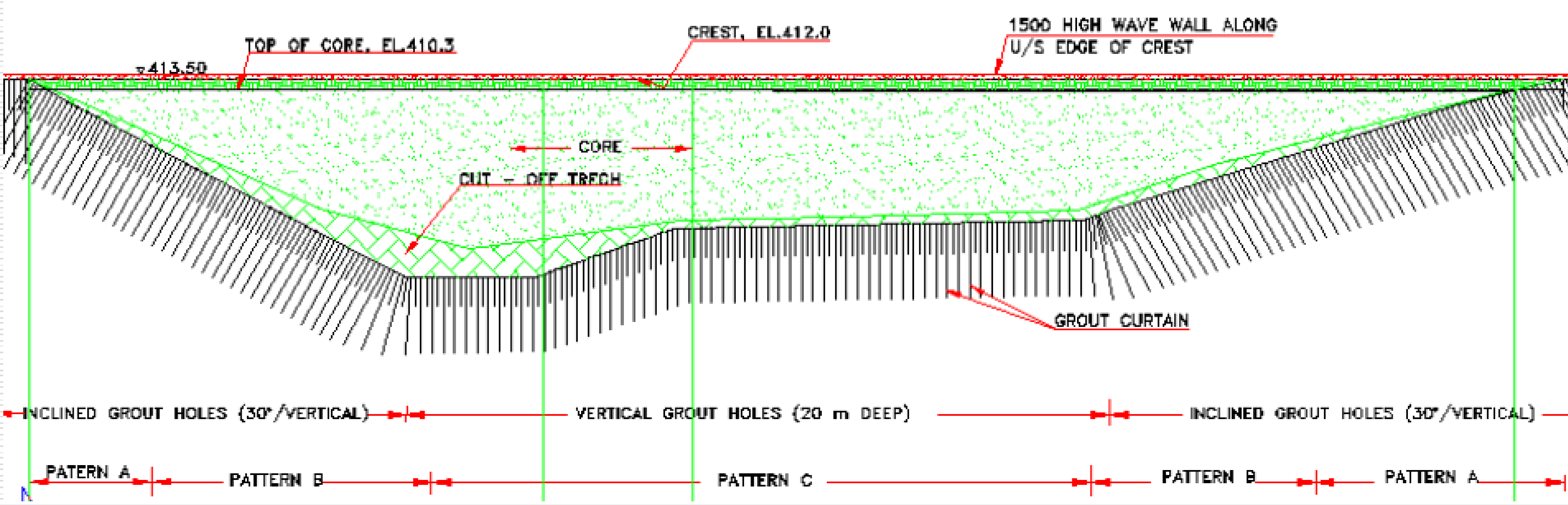
Volume of fill material----1.37mil.

Reservoir capacity --------1.86bm3

Reservoir area ---17,000ha.

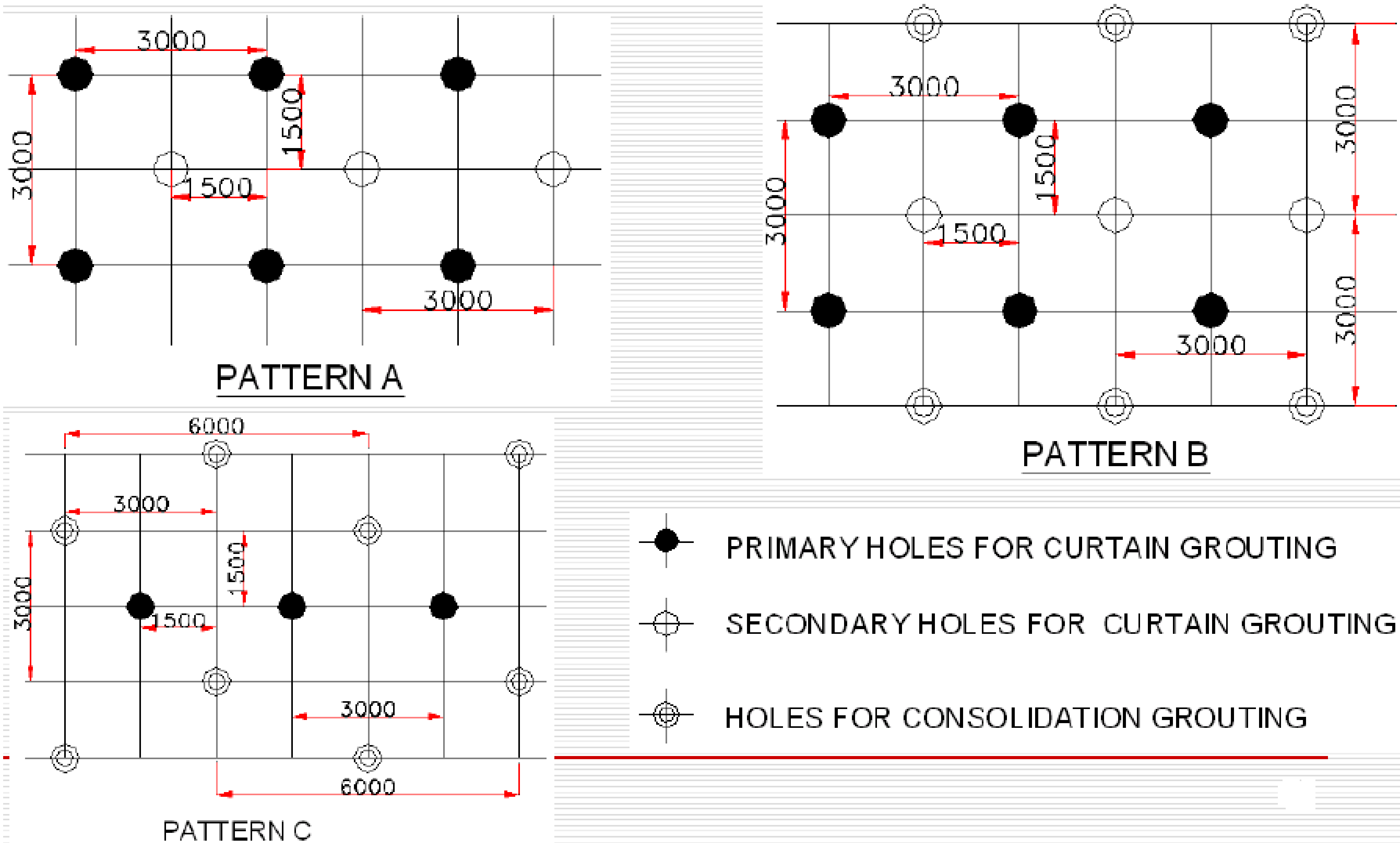


Foundation Grouting (ctd.)



Proposed Grouting profile for Tendaho dam

Foundation Grouting (ctd.)



Leakage in Tendaho Dam

This is leakage through the right abutment after the raining season in 2010. The sever leakage wasn't through the Dam body but the country rock

Task force report coordinated by Water Works Design & Supervision

Leaking water from the right abutment has a temperature of 40ºC, and from the left abutment a temperature of 33ºC; while the reservoir water is 31ºC.

Cut off

The cut off is required,

- To reduce loss of stored water through foundations and abutments
- To prevent sub-surface erosion by piping.

Various types of cut off

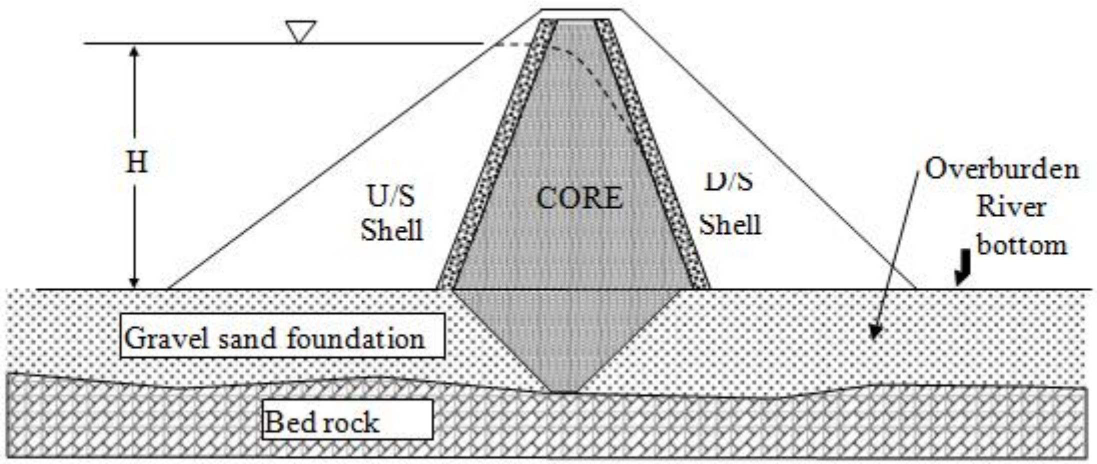
- Cut-off trench
- Sheet piling
- Cement-bound or jet-grouted curtain cutoff
- Slurry trench
- Alluvial grouting
- Concrete cutoff wall

Cut-off Trench

- can be accomplished by excavating a trench and backfilling with compacted earth which is in effect part of the embankment core
- In most cases the economic depth is likely to be less than 10 meters. Beyond this depth slurry trench and other wall cutoffs are likely to be more economic

Types of cut off Trenches

i. **Positive Cut-off Trench**
ii. **Partial cut-off trench**



Positive Cut-off Trench

Partial cut off trench is effective in stratified foundation by intersecting more impervious layer in the foundation and by increasing the vertical path of seepage

Guidelines for design of cut off trench

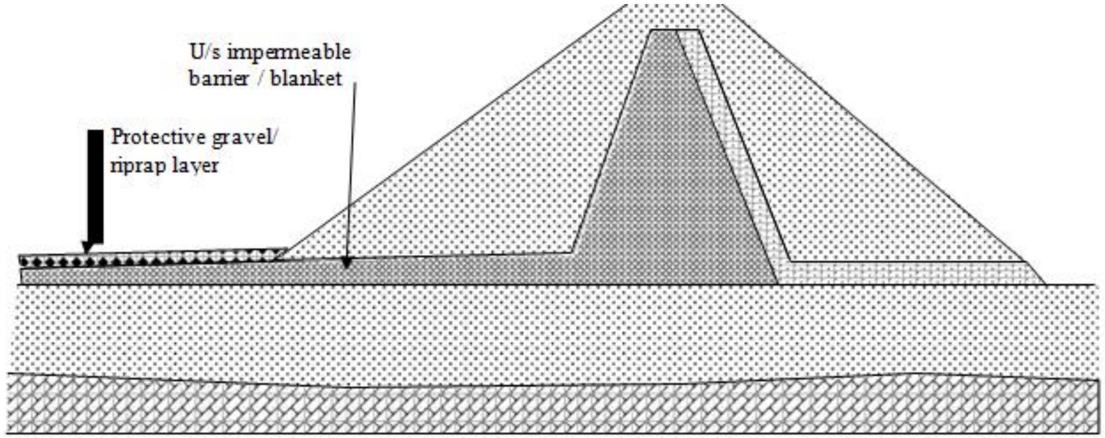
- **Position :** The cut off shall be located such that its center line should be within the base of impervious core. The positive cut off should be keyed at least to a depth of 0.4 meter into continuous impervious sub stratum or inerodable rock formation.
- A minimum **bottom width** of 4.0 meter is recommended.
- **Side slopes** of at least 1:1 or flatter may be provided in case of over burden while 1/2:1 and 1/4:1 may be provided in soft rock and hard rock respectively.
- The **back fill material** for cut off trench shall have same properties as those specified for impervious core.
- The **cut off in the flanks** on either side should normally extend up to the top of impervious core.
- **Cut off trench on weathered rock :** If cut off trench is terminated in rock formation which is weathered or have cracks, joints and crevices and if percolation test exhibit a lugeon value of more than 10 , then rock foundation below the bed of cut off trench should be **grouted**.

Sheet Piling Cutoffs

Steel sheet pile may be driven into **soft alluvium**

- Depth to bed rock
- Used in combination with partial cutoff to seal lower horizons
- limited to use in foundations of silt, sand, and fine gravel (not convenient for cobbles or boulders)
- Not suitable for cobbles/boulders as these formations cause misalignment/ open joints, interlock liable to tear-off, making an ineffective barrier.
- Twin steel sections may be used with interior filled with cement grout
- Not completely water tight
- 80-90% effective if good work
- Poor workmanship, efficiency less than 50%

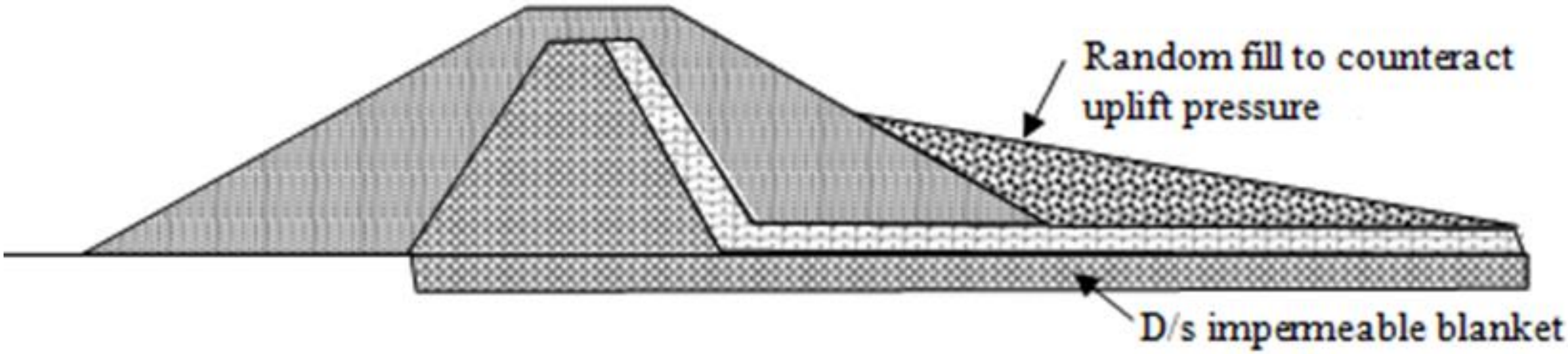
Horizontal U/s Impervious Blanket



- constructed of impervious material extending u/s of the dam face and connected with impervious core of the dam embankment
- used in conjunction with partial cutoff
- Blanket is generally used for a stream channel or valley floor of **sand and gravel**
- Blanket **thickness** 10% of dam height (minimum 10 feet) at dam face to minimum 3 ft at outer end
- Blanket protected from erosion by 2-3 ft thick riprap over gravel bedding
- **Length** of blanket governed by desired reduction in seepage flow
- Blanket may not eliminate piping in naturally stratified soils as high pressures may exist in one or more strata at d/s toe of the dam

Horizontal d/s impervious blanket

- impermeable horizontal blanket may also be provided at d/s of dam to lengthen seepage path and reduce seepage



- must be designed to resist uplift pressure. This is done by providing berm of random fill material to add weight over the impermeable layer

Sand Gravel Foundations

- Gravel/sand foundation has enough bearing/shear strength to support small to medium earthfill and rockfill dams
- However, these foundations are very conducive to seepage and need suitable treatment for seepage and uplift pressure control

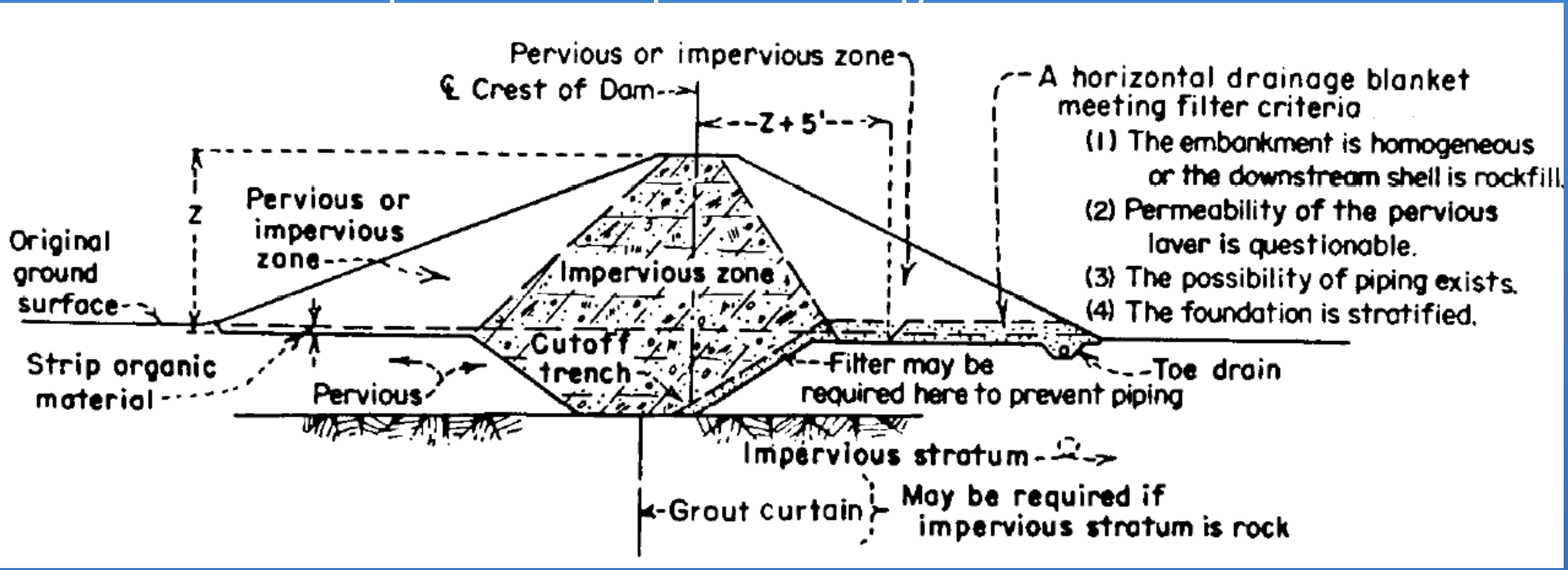
The design criteria require control of seepage flow through the foundation and abutments

The pervious foundation may be either exposed or covered at the surface.

Case I: Exposed Foundation

a. Shallow Foundation (a depth approximately equal to or less than the height of the dam, but should not exceed about 50 feet).

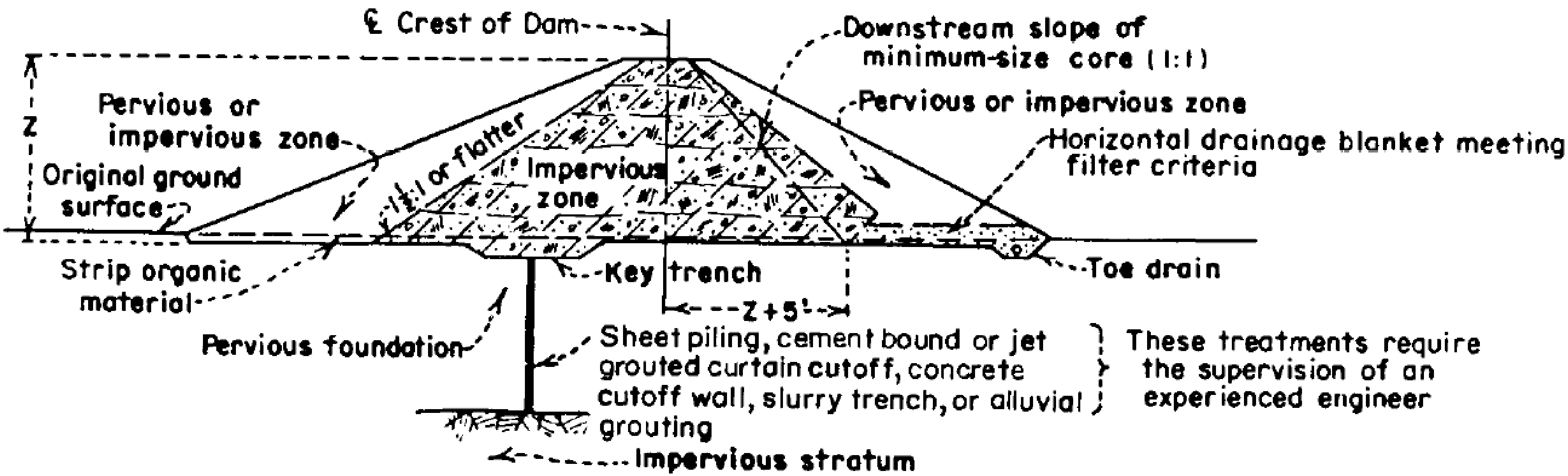
- Provide a positive (complete) cutoff to bedrock.
- Grouting of bedrock, if needed.
- Horizontal drainage blanket not necessary if shallow pervious foundation can act as filter and provide adequate drainage.

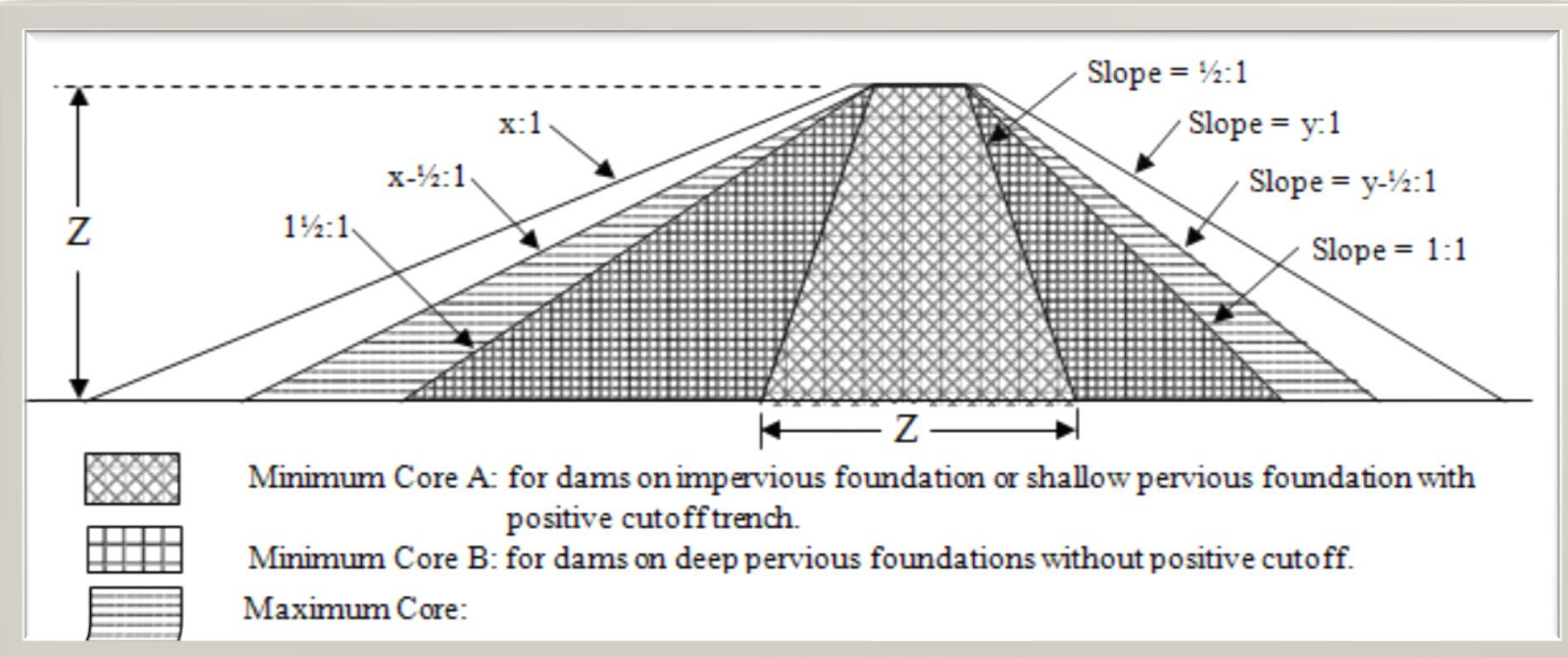


- If rockfill at d/s portion of dam, provide DB from d/s slope to the impervious zone/core.
- If positive cutoff not practical due to lack of materials, short construction season, wet climate, high dewatering cost, then other methods of cutoff be used.

b) Intermediate Depth Foundation (the distance to the impervious layer is too great for a cutoff trench, but can be economically reached by another type of positive cutoff)

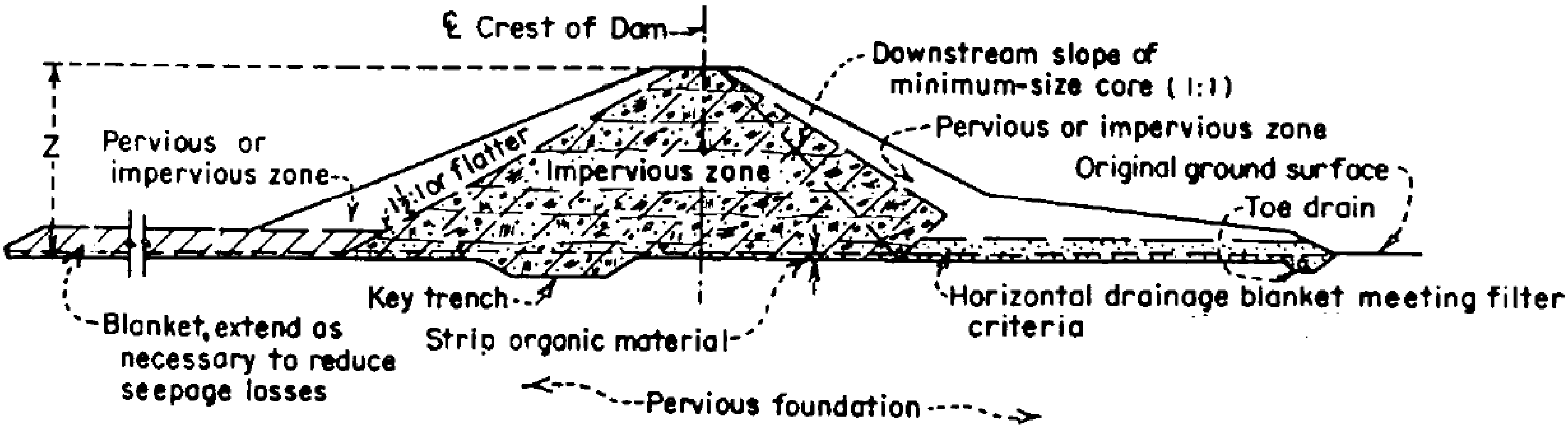
- Positive cutoff trench may be less economical
- Provide other methods of cutoff (sheet pile, slurry trench etc).
- Provide minimum impervious zone/core B (1½:1 u/s slope and 1:1 d/s slope)
- Provide drainage blanket of filter grade if i) overlying zone is impervious or ii) overlying zone is rockfill, iii) piping potential is present
- Provide key trench




Slope = ½:1
x:1
Slope = y:1
x-½:1
Slope = y-½:1
Z
1½:1
Slope = 1:1
Z
Minimum Core A: for dams on impervious foundation or shallow pervious foundation with positive cutoff trench.
Minimum Core B: for dams on deep pervious foundations without positive cutoff.
Maximum Core:

C: Deep Depth Foundation

- Foundation too deep for a positive cutoff
- Provide u/s impermeable blanket in continuation of impermeable core.
- Minimum core B
- Provide key trench
- At d/s of embankment, provide adequate thickness of previous or impervious (random fill) materials (berm) to improve stability against high uplift pressures.



- Provide toe drains
- For foundations of high K, which cause extensive seepage, ponding and sand boils, then provide drainage trenches, pressure relief wells, extension of d/s toe of dam or blanket on d/s area.
- For deep stratified layers, provide partial cutoff and u/s blanket
- Some seepage inadvertent

Case 2: Covered Pervious Foundations

- the type of treatment depends on the thickness and imperviousness of the layer covering the pervious zone and on the permeability of the underlying pervious layer
  - If the overlaying layer is equal to or less than a few feet thick (**say 3 ft**), ignore this and consider conditions for case 1(exposed pervious foundation)
  - **If the overlaying layer is greater than 3 feet and less than the hydraulic head**
    - Assess the suitability of the upstream **covering(the thickness, continuity, impervious qualities, and upstream distance of the natural deposit**) and decide to use it or ignore
    - Provisions should be made to relieve uplift pressures at the downstream toe and to remove the seepage

When the dam embankment is homogeneous or when the downstream zone has questionable permeability, a horizontal drainage blanket is constructed

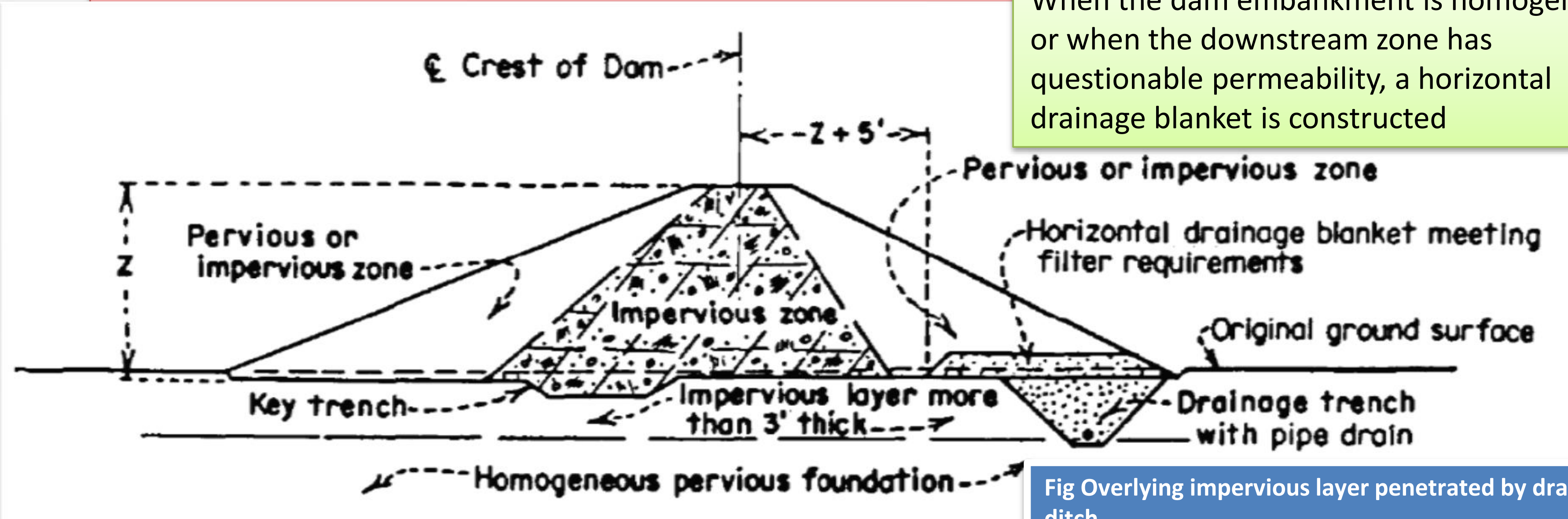


Fig Overlying impervious layer penetrated by drainage ditch

Silt and Clay Foundations

Characteristics

- Foundation of fine grained soil (silt, clay) are sufficiently impermeable and thus no danger of under seepage and piping
- Main problem is stability against consolidation and shear failure due to low bearing/shear strength
- Characteristics depend on location of water table, and compactness of soil
- Weak soils need to be treated for improving strength

Relatively Dry Foundations

- These soils exhibit large strength at its present dryness
- The **relative density** of the material indicates the potential/danger of soil on compression
- Many soils will undergo quick and sudden volume reduction on wetting/saturating on reservoir filling
- Dense soils which will undergo small compaction on loading and wetting may be used as foundation for dams.
- Pre-wetting of soil before loading improves its strength on loading.
- Large compaction and could cause serious rupture/weak section for dam core materials and consequent dam failure
- Drainage must be assured by an underlying pervious layer or by a vertical drainage.

Treatments

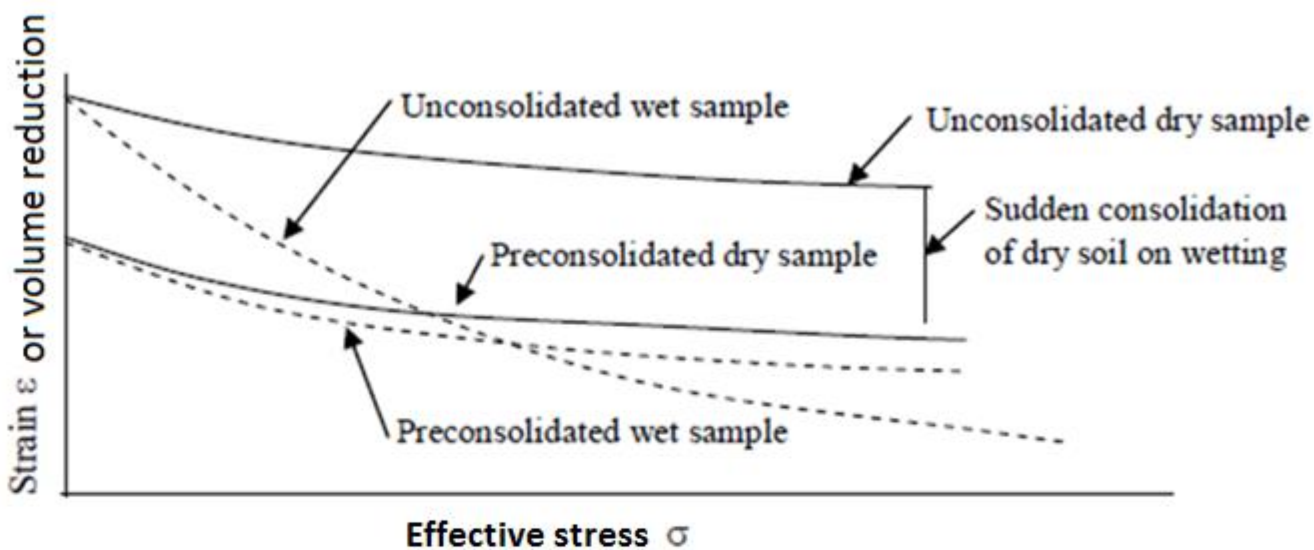
1) Pre-consolidation
2) Densification of cohesionless soils (Vibration)
3) Dynamic compaction

The shear strength can be increased by

I. Removing the soil of low shear strength

II. Providing **drainage of foundation** to permit settlement on drainage and increase of strength during construction.
- Vertical drains may be provided to facilitate consolidation.
- practical for low embankments

III. Reducing the magnitude of the average shear stress along the potential surface of sliding by **flattening the slopes of the embankment**



Design of embankment dams to withstand earthquakes

EFFECT OF EARTHQUAKE ON EMBANKMENT DAMS

Earthquakes can affect embankment dams by causing any of the following:

- Settlement and cracking of the embankment
- Instability of the upstream and downstream slopes of the dam;
- Reduction of freeboard due to settlement or instability which may, in the worst case, result in overtopping of the dam;
- Differential movement between the embankment, abutments and spillway structures leading to cracks;
- Internal erosion and piping which may develop in cracks;
- Liquefaction or loss of shear strength due to increase in pore pressures induced by the earthquake in the embankment and its foundations;
- Overtopping of the dam by waves due to earthquake induced landslides into the reservoir from the valley sides;
- Damage to outlet works passing through the embankment leading to leakage and potential piping erosion of the embankment.

Four main issues to consider

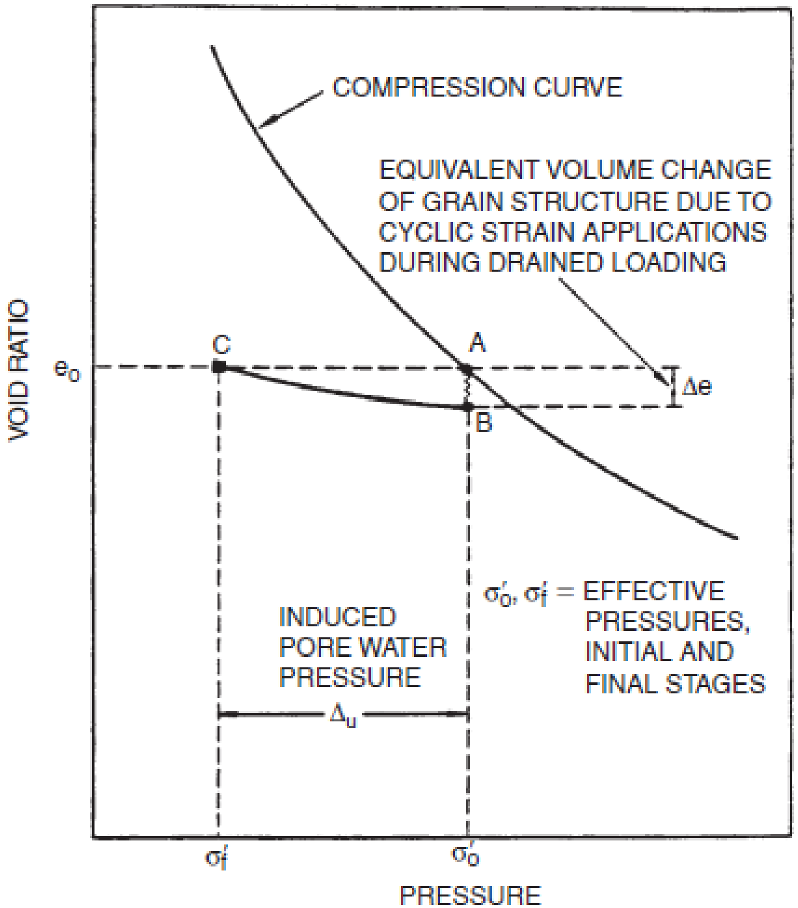
- The general (or "defensive") design of the dam
  - Provision of filters
  - Provision of zones with good drainage capacity (e.g. free draining rockfill);
- The stability of the embankment during and immediately after the earthquake;
- Deformations induced by the earthquake (settlement, cracking) and dam freeboard;
- The potential for liquefaction of saturated sandy and silty soils and some gravels with a sand and silt matrix in the foundation, and possibly in the embankment, and how this affects stability and deformations during and immediately after the earthquake.

LIQUEFACTION OF DAM EMBANKMENTS AND FOUNDATIONS

"Liquefaction"

All phenomena giving rise to a reduction in shearing resistance and stiffness, and development of large strains as a result of increase in pore pressure under cyclic or monotonic (static) loading of contractive soils

- Occurs when the effective stress of a soil is reduced to essentially zero, which corresponds to a complete loss of shear strength

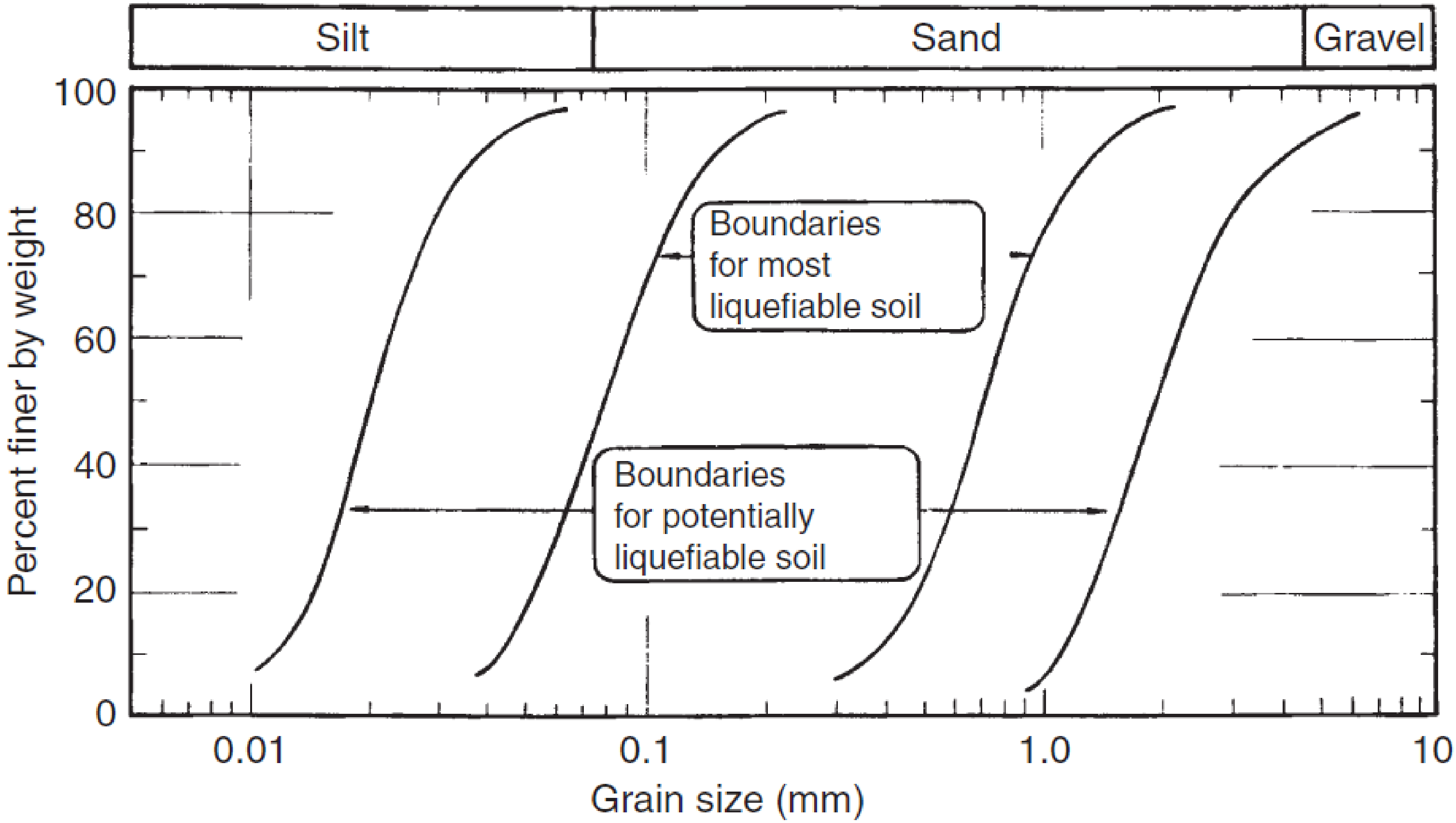


- Soil starting at A and subject to cyclic loading, which would otherwise have ended at B, will in fact have stresses represented by C where total stress σo is taken by $\sigma f'$ and $\Delta u'$. The pore pressures must increase to maintain equilibrium in this undrained condition.
- The pore pressures build up gradually with the number of cycles of loading and only if the pore pressures build up to equal the total stress, does the “initial liquefaction” (effective stress 0) condition occur.

Schematic illustration of mechanism of pore pressure generation during cyclic loading

Soils susceptible to liquefaction

- Saturated sands, silty sands, and gravelly sands are susceptible to liquefaction.



Limits in the particle size gradation curves separating liquefiable and unliquefiable soils

The resistance of the cohesionless soil to liquefaction will depend on **the density of the soil, confining stresses, soil structure (fabric, age and [cementation](#)), the magnitude and duration of the cyclic loading, and the extent to which shear stress reversal occurs**

SEISMIC STABILITY ANALYSIS OF EMBANKMENTS

The methods of analysis currently used in practice to evaluate seismic stability of embankment dams vary widely, ranging from simple limit equilibrium type analyses to highly sophisticated numerical modeling techniques. These include:

- Pseudo-static analysis
- Simplified methods of deformation analysis
- Numerical modelling techniques
  - total stress
  - effective stress.

Pseudo-static analysis

- was the standard method of stability assessment for embankment dams under earthquake loading.
- The approach involved a conventional limit equilibrium stability analysis, incorporating a horizontal inertia force to represent the effects of earthquake loading.
- The inertia force was often expressed as a product of a seismic coefficient “k” and the weight of the sliding mass W

Numerical methods

- Numerical modelling techniques such as **the finite element method** were first applied to the dynamic analysis of embankment dams by Clough and Chopra (1966).
- In this approach the dam is idealized as a t**wo-dimensional plane strain or plane-stress finite element system**, the reservoir being regarded as a continuum. The foundation zone is generally idealized as a finite element system equivalent to a viscoelastic half-space.
- The dynamic numerical codes used in practice may be divided into two main categories:
    - total stress codes, and
    - effective stress codes (Zienkiewicz et al., 1986; Finn, 1993).

Simplified methods of deformation analysis for dams where liquefaction and significant strain weakening do not occur

- There are a number of approaches for estimating the deformations of a dam which may occur during an earthquake. These include:

  (a) Empirical methods based on recorded deformations, dam geometry and earthquake loading e.g. Swaisgood (1998), Pells and Fell (2002, 2003) extended this to include an empirical method to assess whether cracking would occur;

  (b) Integration of the displacements which occur when the earthquake loading exceeds the available strength e.g. Newmark (1965) and developments of that approach using **simplified numerical analyses programs** such as SHAKE;

  (c) Developments of the Newmark (1965) approach to allow for dynamic response of the embankment e.g. Makdisi and Seed (1978).

- Here we will discuss only the first approach and you may further read the next approaches from the text book.
- Swaisgood (1998) recommended the following equations to **predict settlement:**

$$CS = SEF \times RF$$

- where CS is the vertical crest settlement expressed as a percentage of the dam height plus the alluvium thickness. SEF is the seismic energy factor and RF is the resonance factor.
- These factors are calculated from:

$$SEF = e^{(0.72M+6.28\,PGA-9.1)}$$

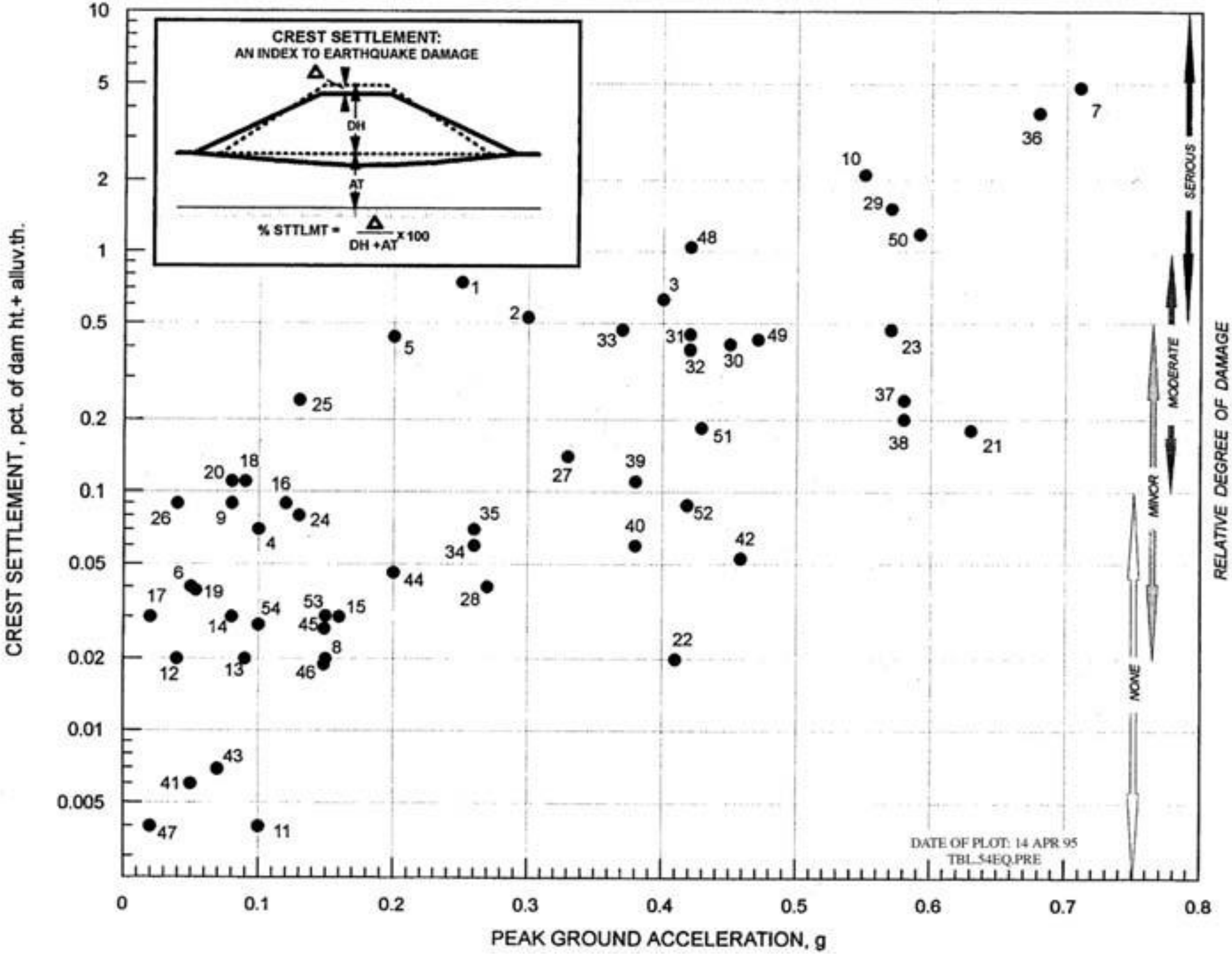
- in which M is the magnitude of the earthquake, and PGA is the peak horizontal ground acceleration at the dam site as a fraction of the acceleration due to gravity

- RF = 2.0 $D^{0.35}$ for earthfill dams

  = 8.0 $D^{0.35}$ for hydraulic fill dams

  = 0.12 $D^{0.61}$ for rockfill embankments

  – in which D is the distance between seismic energy source and dam, in kilometres


CREST SETTLEMENT:
AN INDEX TO EARTHQUAKE DAMAGE
DH
AT
% STTLMT = Δ / (DH + AT) x 100
CREST SETTLEMENT, pct. of dam ht.+ alluv.th.
PEAK GROUND ACCELERATION, g
RELATIVE DEGREE OF DAMAGE
SERIOUS
MODERATE
MINOR
NONE
DATE OF PLOT: 14 APR 95
TBL_54EQ.PRE

Pells and Fell empirical method for estimating settlement, damage and cracking

- Pells and Fell (2002, 2003) gathered data from 305 dams, 95 of which reported cracking, and classified these for damage according to the system shown in Table next slide.
- Figures shown in the following slides (Fell et. 2005) show plots of damage contours versus earthquake magnitude and peak ground acceleration for earthfill and earth and rockfill dams

Table 12.4. Damage classification system for embankment dams under earthquake loading (Pells and Fell, 2002, 2003).

Damage class			
Number	Description	Maximum longitudinal crack width[1] mm	Maximum relative crest settlement[2] (%)
0	No or slight	<10 mm	<0.03
1	Minor	10–30	0.03–0.2
2	Moderate	30–80	0.2–0.5
3	Major	80–150	0.5–1.5
4	Severe	150–500	1.5–5
5	Collapse	>500	>5

[1] Maximum crack width is taken as the maximum width, in millimetres, of any longitudinal cracking that occurs.

[2] Maximum relative crest settlement is expressed as a percentage of the maximum dam height (from general foundation to dam crest).

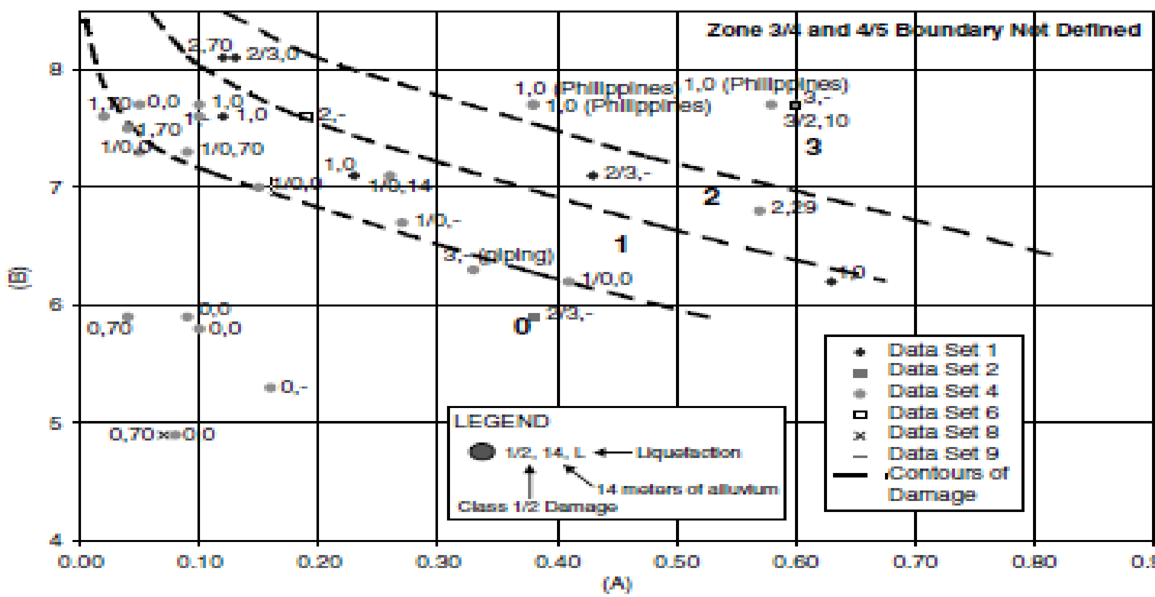


Figure 12.34. Earthfill Dams – contours of damage class versus earthquake magnitude and peak ground acceleration. (A) Earthquake magnitude. (B) Foundation peak ground acceleration (as a fraction of acceleration due to gravity). Contours drawn without consideration for cases that had liquefaction (Pells and Fell, 2003).

DEFENSIVE DESIGN PRINCIPLES FOR EMBANKMENT DAMS

- Apply logical, commonsense measures to the design of the dam, to take account of the cracking, settlement and displacements which may occur as the result of an earthquake.
- As important (probably more so) as attempting to calculate accurately the stability during earthquake or the likely deformations.

The most important measures which can be taken are:

- Provide ample freeboard to allow for settlement
- Use well designed and constructed filters downstream of the earthfill core to control erosion if the core (or face) being cracked in the earthquake. Filters should be taken up to the dam crest level, so they will be effective in the event of large crest settlements, which are likely to be associated with transverse cracking. For larger dams, full width filters (2.5 m to 3 m) might be adopted instead of narrower (1.5m say) filters placed by spreader boxes.
- Provide ample drainage zones to allow for discharge of flow through possible cracks in the core
- Avoid, densify, drain (to be non-saturated) or remove potentially liquefiable materials in the foundation or in the embankment

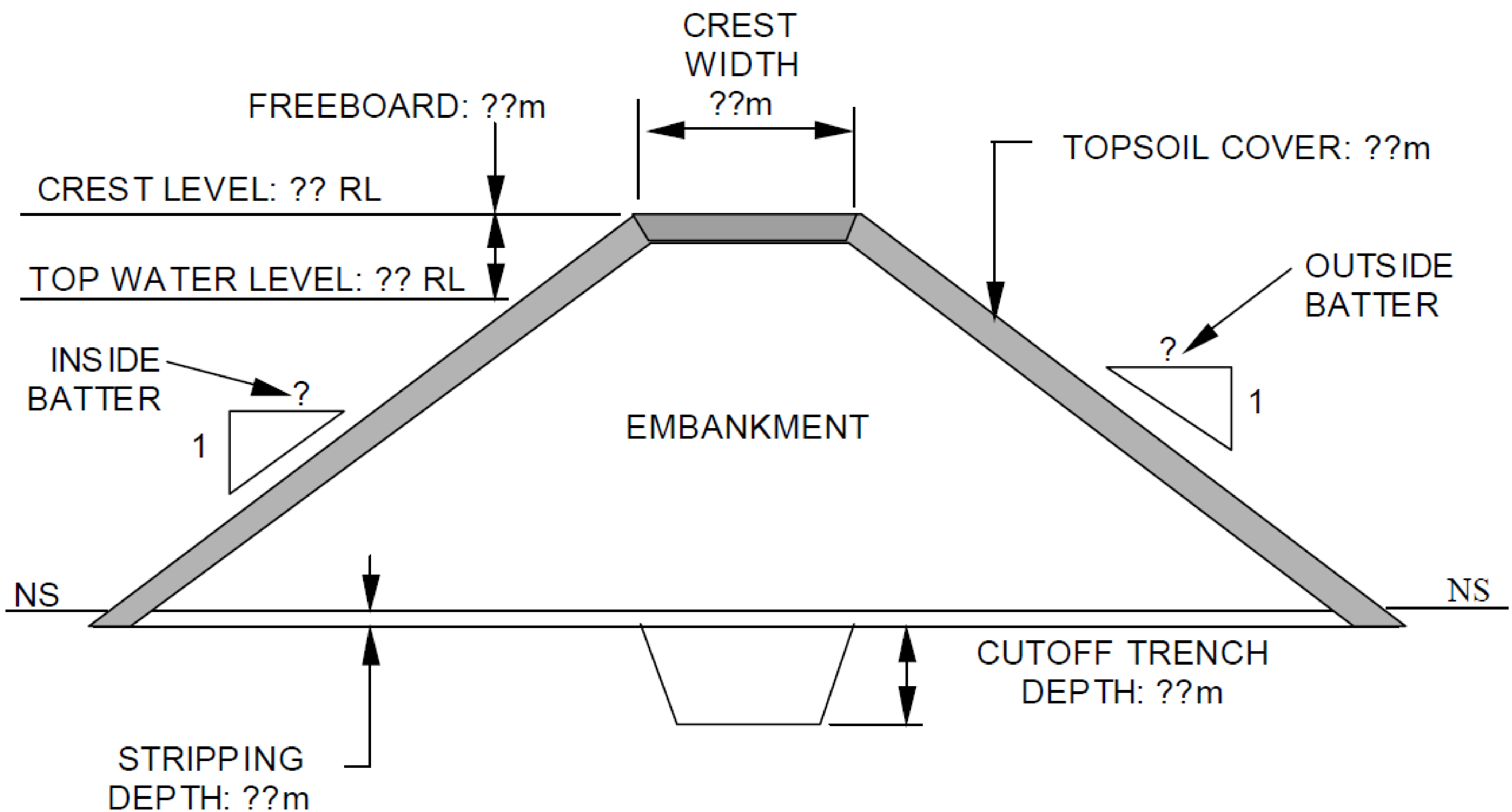
- The foundation under the core should be shaped to avoid sharp changes in profile across the valley. These are likely to make the core more susceptible to cracking due to differential settlement under earthquake (and normal) loading.
- Use a well-graded filter zone upstream of the core to act as a crack stopper, possibly only to be applied in the upper part of the dam
- Locate the core to minimize the degree of saturation of materials (e.g. use sloping upstream core).
- Stabilize slopes around the reservoir rim (and appurtenant structures such as spillways) to prevent slides into the reservoir or onto the structures

Embankment section design

Elements of Dam Design

- The preliminary dam section design will mainly comprises dam height, dam zoning, crest size and fixing the side slopes of the dam.
- For fixing the height of the dam it is necessary to determine the normal storage level and of course the free board.
- The normal storage level or also called maximum storage level or normal pool level is basically fixed after detailed analysis of the hydrologic study.
- The dam zoning are done in terms of imperviousness requirement and slope stability demands.
- The dam crest is a function of several factor such as stability, seepage length and road requirements.
- The side slopes are basically determined by stability requirements in terms of the material used for construction and also the height of the dam.

Embankment section design



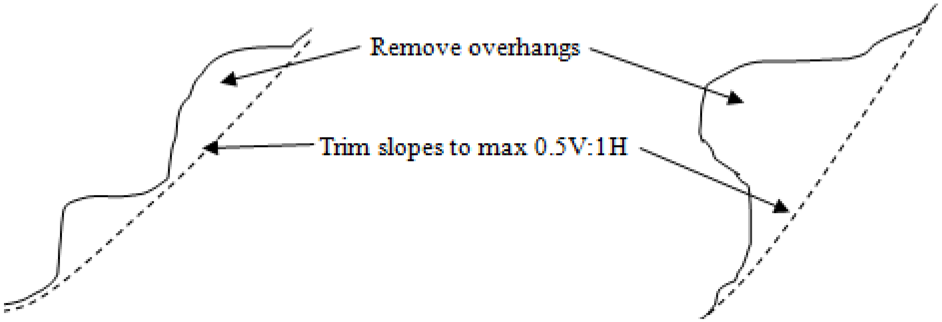
General Design Criteria Of Earthfill Dams

- Stability
- Control of Seepage
- Overtopping and Free Board
- Maximum Flood Evacuation
- Upstream Slope Protection
- Outlet and Ancillary Works

i. Dam Height Determination

a. Axis Alignment

- Axes of embankment dams are usually made straight or of the most economical alignment fitting the topography and foundation conditions
- Thin abutment ridges should be avoided as much as possible.



b. Freeboard Computation

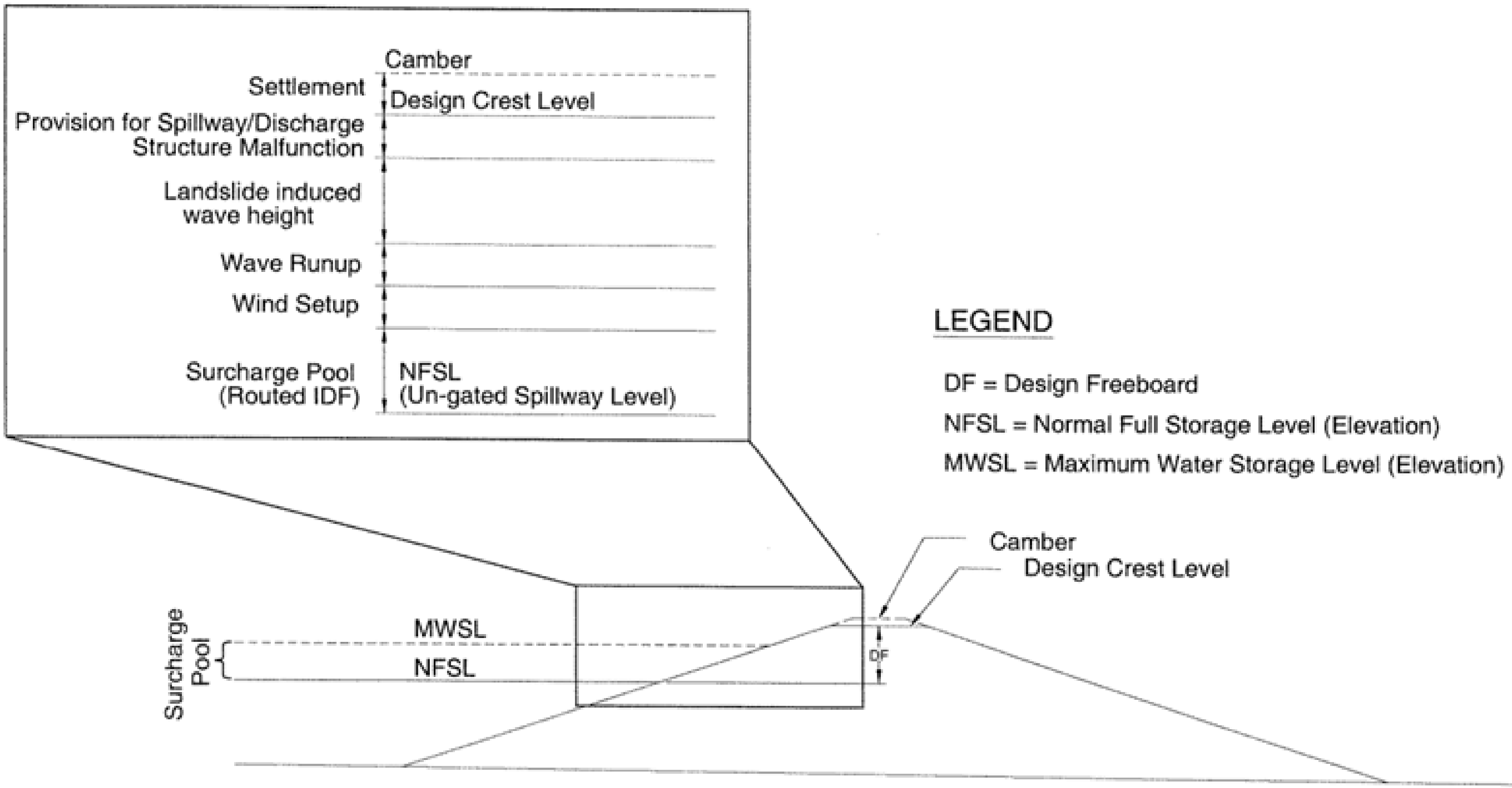
Freeboard: is the vertical distance between the crest of the embankment (without camber) and the reservoir water surface.

Normal Freeboard: vertical distance between the crest of the embankment and the Normal Water Level.

Minimum Freeboard: vertical distance between the crest of the embankment and the Maximum Water Level.

Freeboard Design Considerations

- ✓ Wind-generated wave action, wind setup, and wave runup;
- ✓ Earthquake and/or landslide-generated waves and runup;
- ✓ Post-construction settlement of embankment dams and foundations;
- ✓ Provision for malfunction of spillways (especially gated structures) and outlet works, and
- ✓ Site-specific uncertainties including flood hydrology



First Approximations for Freeboard Requirements

Freeboard Requirements for Preliminary Studies of Small Dams for rock faced slope (USBR, 1987, 1992)

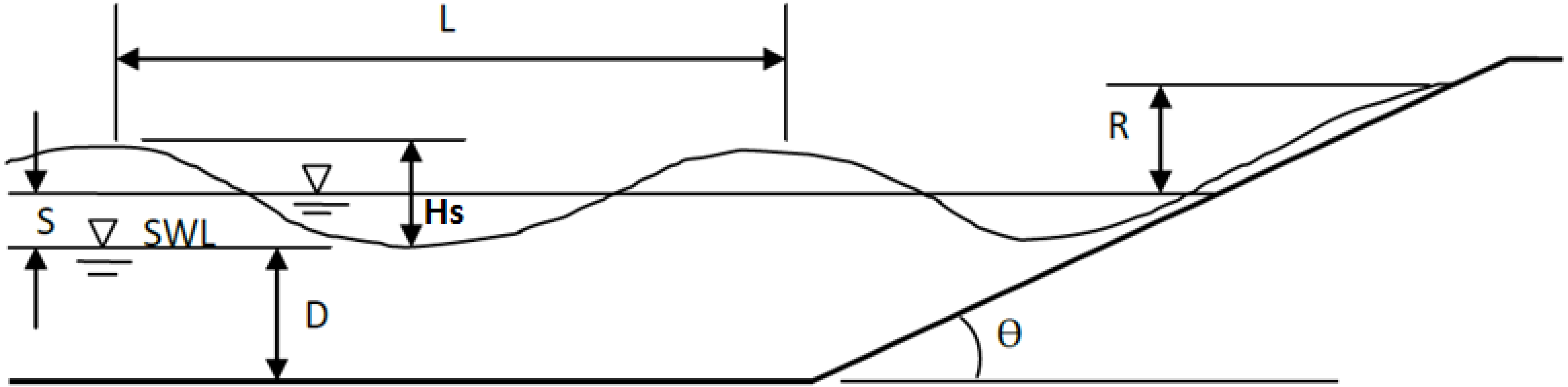
Longest fetch (km)	Normal Freeboard (m)	Freeboard – MFL(*) (m)
< 1.6	1.2	0.9
1.6	1.5	1.2
4.0	1.8	1.5
8.0	2.4	1.8
16.0	3.0	2.1

(*) MFL = Maximum flood level.

i. USBR Detailed Studies (Wave based calculations- Minimum free board)

The computations are based on significant wave height

Definition of Terms for Design Wave Parameters



$$L = 5.12\,T^2$$ Where L is the deep water wave length in feet and T in seconds

Significant wave height (Hs, in feet) and **wave period (T)** are first computed from the design wind and effective fetch (Fe) using design charts, L is also in feet

Wave run-up (R) is defined as the vertical height above still-water level (SWL) to which water from an incident wave will run up the face of the dam

$$\text{R(in feet)} = \frac{H_s}{0.4 + \left(\frac{H_s}{L}\right)^{0.5} \cot\theta}$$

For embankment dams with smooth upstream faces, the computed run-up is increased by a factor of 1.5.

Wind setup (S in feet) is computed as follows

$$S = \frac{U_f^{\,2} F}{1400\,D}$$

Source: US Army Corps of Engineers (USACE 1989)

Where Uf is the design wind velocity over water (mile/hour), and D = average water depth along the fetch (feet) and F is the fetch in feet (normally equals 2 Fe.)

Freeboard (minimum) required = R + S

Wind Velocity Relationship – Water to Land (USBR, 1992)

Effective Fetch (Fe) (km)	**Wind Velocity Ratio (over water/over land)**
0,8	1.08
1,6	1.13
3,2	1.21
4,8	1.26
6,4	1.28
≥ 8	1.30

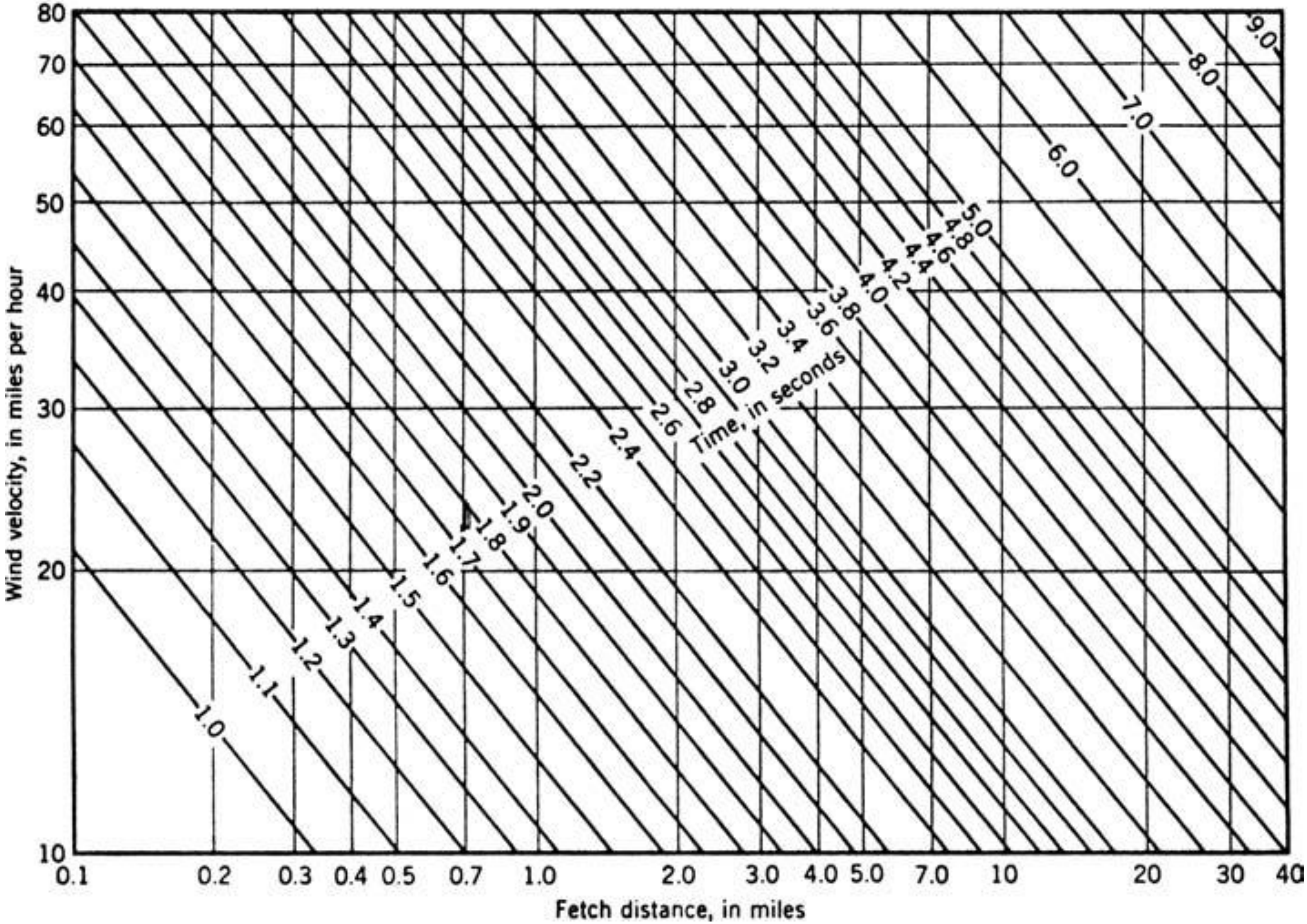


Fig for Wave periods (T)

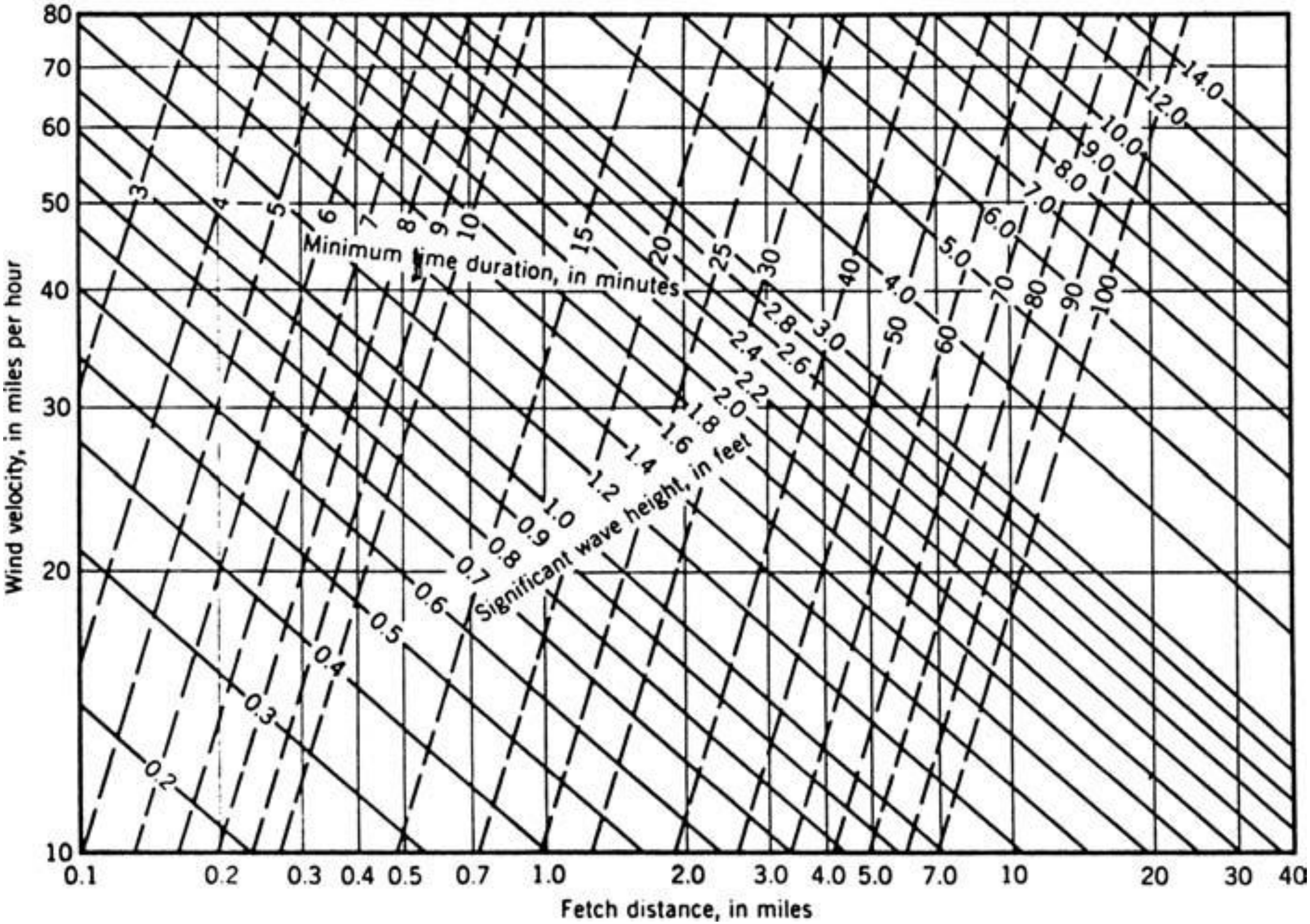
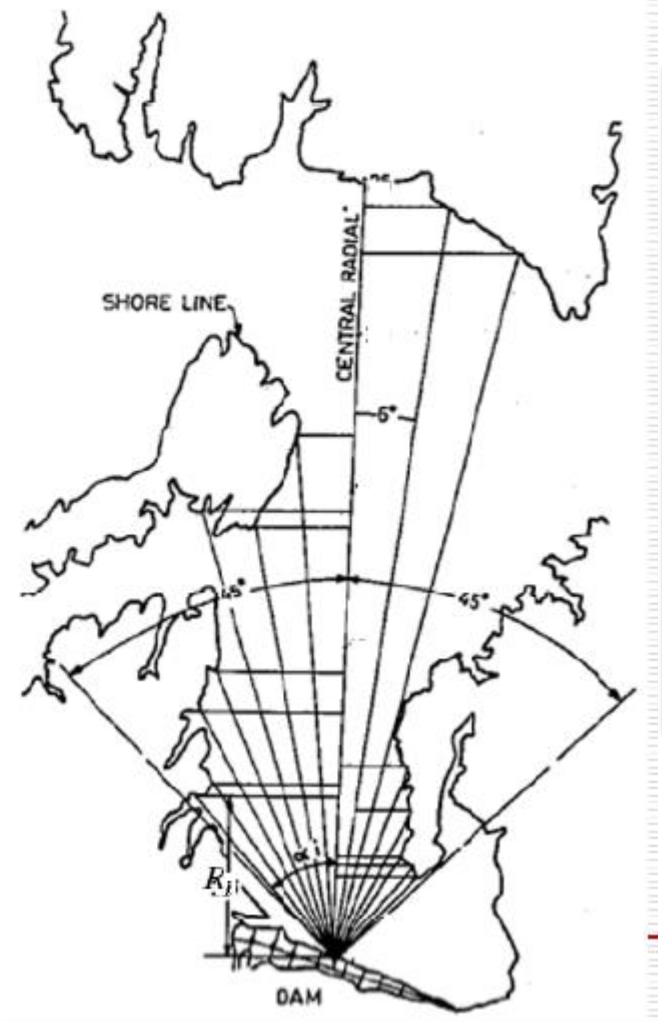


Fig Wave heights and minimum duration wind (Saville et al., 1962; USBR, 1981). Note 1 mile per hour= 1.6 km per hour

The **effective fetch** at a given station can be computed by **Saville's Method**

$$F_e = \frac{\sum X_i \cos^2 \alpha_i}{\sum \cos \alpha_i}$$

in which αi= angle (in degree) between the central radial and radial I and Xi = length of projection of radial i on the central radial



- Fetch lengths are usually drawn as straight lines from the center of the point of interest, here from the dam face to the opposite bank
- A trial and error approach should be used to select the critical position on the dam and direction of the central radial to give the maximum effective fetch.
- The radials spanning 45° on each side of the central radial should be used to compute the effective fetch.

ii. The Molitor Stevenson formula

Fb = Hs + R + S

For F < 32 km

Where Fb = Free board (m)

Hs = wave height = $0.032(\sqrt{V.F}) + 0.76 - 0.24F^{1/4}$

V = Design wind velocity (km/hr)

F = Fetch length at maximum reservoir level (km)

R = wave Run up = 50 % Hs

S= wind set up = $\dfrac{V^2.F.Cos^2\alpha}{62000D}$

α = Angle between the wind direction and the fetch

D = Average Reservoir depth in meters over the fetch distance

For F > 32 km $Hs = 0.032\sqrt{V.F}$

Example

An ogee spillway of crest length 12m has been provided to pass a routed flood of $31.4m^3/s$ for an earth dam. The dam is located in an area where the wind velocity data shows that the maximum recorded velocity is 80km/h, and from the topographic surveys and reservoir planning studies, the following data are available.

- Minimum river bed level is 2522m asl
- Full reservoir water level is 2541.5m asl
- Fetch length at maximum reservoir level is 0.8km

Calculate the free board for the earth dam using the Molitor Stevenson formula.

Crest Design

In designing the dam crest of a small earthfill dam the following items should be considered

- ✓ Width
- ✓ Drainage
- ✓ Camber
- ✓ Surfacing
- ✓ Safety requirements

Crest width W

The width W of the crest is governed by

- ➢ height of dam,
- ➢ importance of structure,
- ➢ width of highway,
- ➢ construction procedure,
- ➢ access required either during construction or as a permanent feature

The crest width is, as a rule, determined empirically and largely by precedent experience.

1. According to the Bureau of Reclamation

$w = H/5 + 10$

where: w = width of crest in feet

H = height of dam, in feet, above the streambed

Crest width recommended for small dams – Bureau of Reclamation

Dam Height (m)	Crest Width (m)
5	4
10	5
15	6

2. Japanese code

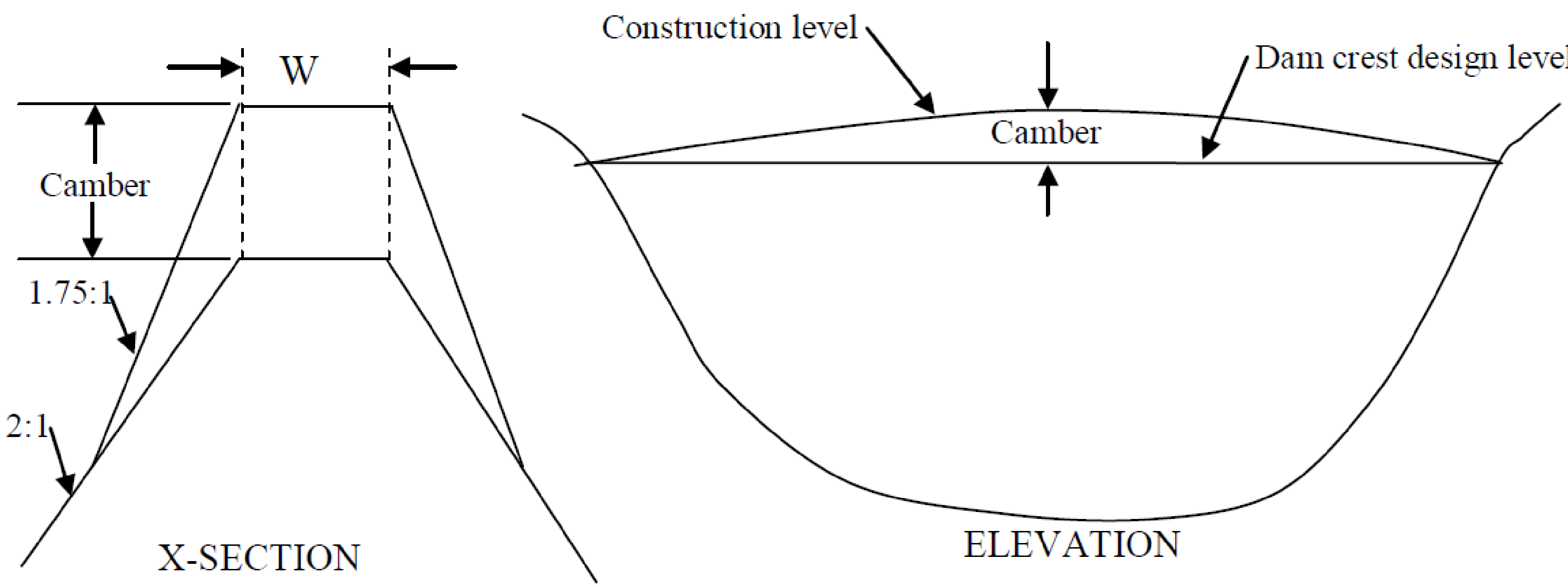
$W\ (m) = 3.6\ H^{1/3} - 3$

3. empirical formula (Lewis, 2002)

Crest width (m) = $H^{0.5} + 1$

Camber

- The crest elevation is increased towards center of the dam by an amount equal to future consolidation of dam foundation and embankment after completion of the construction.
- Selection of amount of camber is somewhat arbitrary
- The camber is provided by increasing the u/s and d/s slopes near the crest of the dam
- For non-compressible foundations, camber of about 1% of dam height is provided.



Surface Drainage

Surface drainage of crust be provided by a crown of at least 3”, or by sloping towards the upstream floor. For wider crest 2% slope is adequate.

Surfacing

- Crest surface should be protected against damage from wave splash, rainfall, wind, frost and traffic wear
- A layer of fine rock or gravely material of 4 inches minimum thickness be provided
- If a highway is carried across the dam, then crest width and surfacing must conform to highway codes.

Safety Requirements

- providing metallic or concrete guard rails on both shoulders of the crest

Embankment Slopes and zoning

Embankment slopes are designed to ensure

- strength,
- stability and economy of construction: Flat slopes, more cost, more stability/strength; Steeper, lower costs, stability or strength.

Embankment slopes

- may be continuous or discontinuous/berms
- may have a single slope over whole height, or multiple slopes
- The u/s and d/s slopes of the embankment and core are selected from general guidelines, experiences in the light of foundation materials and materials available for construction.
- The seepage analysis and stability of the selected dam section is carried out and dam section may be acceptable if factor of safety for the dam under different construction and operation conditions are found satisfactory
- Stability of the shape is analyzed under static loads as well as under seismic conditions.

The slope depends on

- materials available,
- foundation condition,
- dam height
- Coarser free draining materials allow steeper slopes, and finer materials require flatter slope
- on the type of the dam and on the nature of materials for construction.

Homogeneous

Materials				No rapid draw down		Rapid drawdown	
				u/s	*d/s*	u/s	*d/s*
GW	GP	SW	SP	Materials not suitable -too pervious			
GC	GM	SC	SM	2½:1	2:1	3:1	2:1
CL	ML			3:1	2½:1	3½:1	2½:1
CH	MH			3/6:1	2½:1	4:1	2½:1

Zoned embankment

- All zoning schemes are based on the estimated quantities of required excavation and of borrow area materials available.
- zoning scheme may divide the dam into two or more zones, depending on the range of variation in the character and gradation of the materials available for construction.
- In general, the permeability of each zone should increase toward the outer slopes

Recommended slopes for small zoned earthfill dams on stable foundations

Type	Shell material	Core material	No rapid drawdown		Rapid drawdown	
			U/s	D/s	U/s	D/s
Min core A	Rock, GW, GP, SW, SP, gravely	GC, GM, SC, SM, CL, ML, CH, MH	2:1	2:1		
Max core	Rock, GW, GP, SW, SP gravely	GC, GM	2:1	2:1	2½:1	2¼:1
		SC, SM	2¼:1	2¼:1	2½:1	2¼:1
		CL, ML	2½:1	2½:1	3:1	2½:1
		CH, MH	3:1	3:1	3½:1	3:1

Shell materials

- Properties of shell materials according to Unified Classification

S.N	Soil Classification Group	Proctor compaction		Permeability ,k 10^{-6} cm/sec	Compressibility, %		Shearing strength
		Max Dry Density (t/m^3)	Optimum Moisture Content (%)		At 1.4 kg/cm^2	At 3.5 kg/cm^2	Tanϕ
1.	GM	>1.82	<14.5	>0.3	< 1.2	<3.0	>0.67
2.	GP	>1.76	<12.4	(0.031 – 0.09)	< 0.8	(.)	>0.74
3.	GM	>1.82	<14.5	>0.3	< 1.2	<3.0	>0.67
4.	GM	>1.82	<14.5	>0.3	< 1.2	<3.0	>0.67
5.	GM	>1.82	<14.5	>0.3	< 1.2	<3.0	>0.67
6.	GP	>1.76	<12.4	(0.031 – 0.09)	< 0.8	(.)	>0.74
7.	GP	>1.76	<12.4	(0.031 – 0.09)	< 0.8	(.)	>0.74
8.	GM	>1.82	<14.5	>0.3	< 1.2	<3.0	>0.67
9.	GP	>1.76	<12.4	(0.031 – 0.09)	< 0.8	(.)	>0.74
10.	GP	>1.76	<12.4	(0.031 – 0.09)	< 0.8	(.)	>0.74

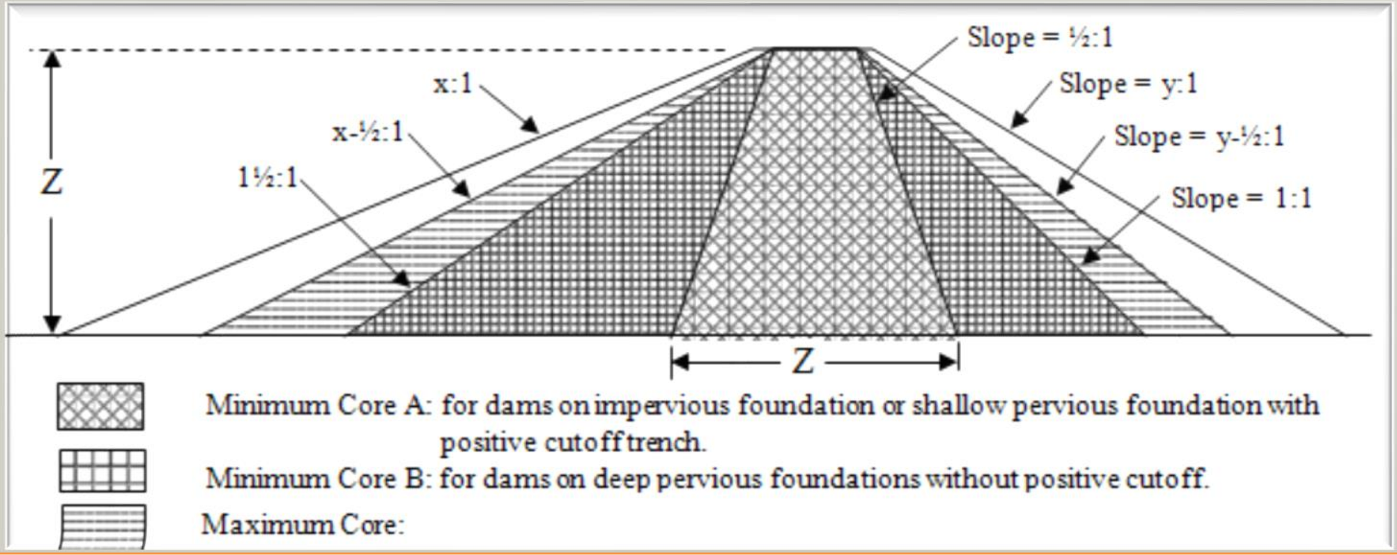
Design of Impervious Core

The core of a dam is built within the embankment to form the impermeable barrier, the shell and other part of the dam being provided to ensure stability.

-The core is usually designed by use of natural materials such as clay, sandy clay, clayey sand, clayey gravel, silty sand, etc.

Engineering use chart (Wagner, 1957)					
SOIL GROUP SYMBOL	PERMEABILITY WHEN COMPACTED	SHEAR STRENGTH WHEN COMPACTED AND SATURATED	COMPRESSIBILITY WHEN COMPACTED AND SATURATED	WORKABILITY AS CONSTRUCTION MATERIAL	AS CORE MATERIAL
GC	IMPERVIOUS	GOOD TO FAIR	VERY LOW	GOOD	1
SC	IMPERVIOUS	GOOD TO FAIR	LOW	GOOD	2
CL	IMPERVIOUS	FAIR	MEDIUM	GOOD TO FAIR	3
CH	IMPERVIOUS	POOR	HIGH	POOR	7

Size range of impervious core for zoned embankment



Impervious Core

- Pervious or impervious foundation with positive cut off - provide minimum core A (top width 10ft, base = Z, symmetric)
- Exposed pervious foundations or covered pervious foundation (cover < 3 ft). No positive cutoff-minimum core B
- For core greater than maximum core, outer shells become ineffective in stabilizing the dam and embankment may be considered as homogenous for stability analysis.
- Core smaller than minimum core – dam as diaphragm type.
- Impervious cover over foundation more than 3feet- select between core A and core B depending on extent and effectiveness of the core.
- Top of the core kept 3-5 ft below crest to safeguard against weathering

Impervious Core Thickness

There is no definite rule for determining the safe thickness of the core. It is governed by different factors such as:

-Tolerable seepage loss.

-Minimum width for proper construction.

-Type of material available.

-Design of the proposed filter layer

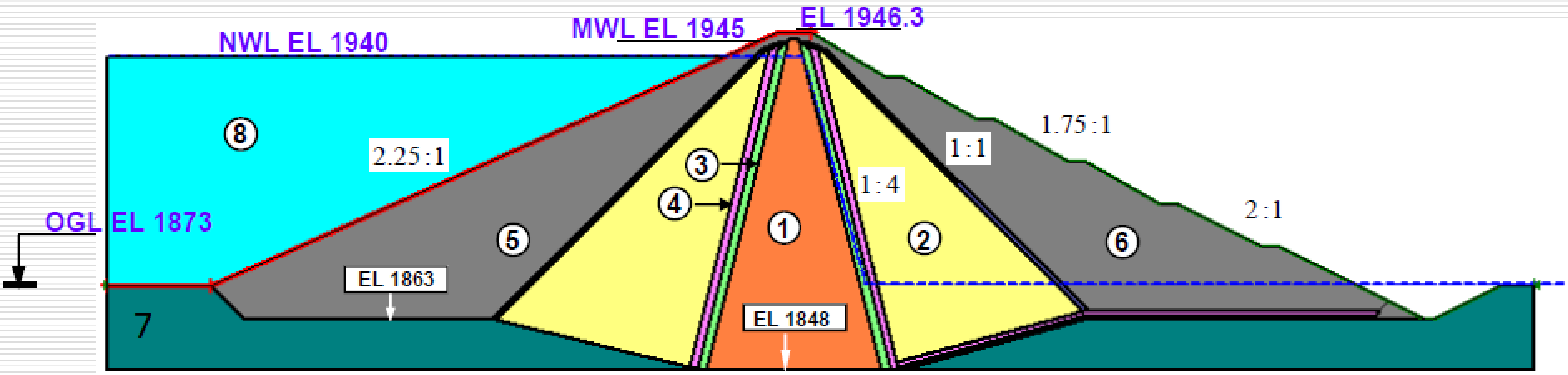
- Sherard et al. (1963) suggested the following criteria:

1) **Cores with a width of 30% to 50% of the water head have proved satisfactory for any soil type and dam height**
2) **Cores with a width of 15% to 20% of the water head are considered thin but with adequately designed and constructed filter layers, they are satisfactory under most circumstances**
3) **Cores with widths of much less than 10% of the water head have not been used widely.**

Thomas (1976) suggested that a common value for the core thickness is about one-half of the dam height at that particular section – with a minimum above reservoir flood level

Ribb Zoned Earth-Rockfill Dam (Large Dam)

- **Capacity** : **234 million m^3**
- **Irrigation** : **20,000 ha**



Zone 1: Impervious core

Zone 2: Shell + compacted alluvium

Zone 3: Fine filter

Zone 4: Coarse filter

Zone 5: Transition zone

Zone 6: Rockfill

Zone 7: Alluvium foundation

Zone 8: Water

- Resulting in minimum core thickness of more than 50% of water head.

Control of Seepage, Internal erosion & piping for embankment dams

Design Objective:

- the objective is to ensure that the dam has adequate measures to control seepage and potential internal erosion so that:

A) Pore pressures in the dam and foundation are such that there is an adequate margin of safety against slope stability.

B) Internal erosion, which might progress to form a pipe and breach the dam, will not occur

These Objectives are achieved by zoning of the dam, provision of filters and high seepage discharge capacity zones and other embankment design and foundation treatment measures. The design features are

- **Impervious core**
- **Cutoff trench**
- **Chimney filter drain**
- **Grouting**
- **Shell zones**
- **Horizontal filter drain**
- **D/S Weighing berm**
- **Upstream blanket**

- Seepage occurs through all embankment dams & foundation.

-The permeability of most compacted earthfill core materials is

less than 1×10^{-8} to 1×10^{-9} m/s.

-By comparison, virtually all rock foundations have rock mass permeability greater than 1 Lugeon (approximately 1×10^{-7} m/s.) And most rocks have greater than 5 Lu.

-Foundations of alluvial or deeply weathered and lateritised rock, may have mass permeability as high as $10^{-3} – 10^{-5}$ m/s.

Most of the seepage is through the foundation, not embankment

Details of some measures for pore pressure and seepage flow control

Horizontal and vertical drains

- Seepage beneath a dam on a permeable soil (or permeable weathered rock) foundation should be allowed to exit in a controlled manner into a horizontal drain.
- The horizontal drain should have sufficient capacity

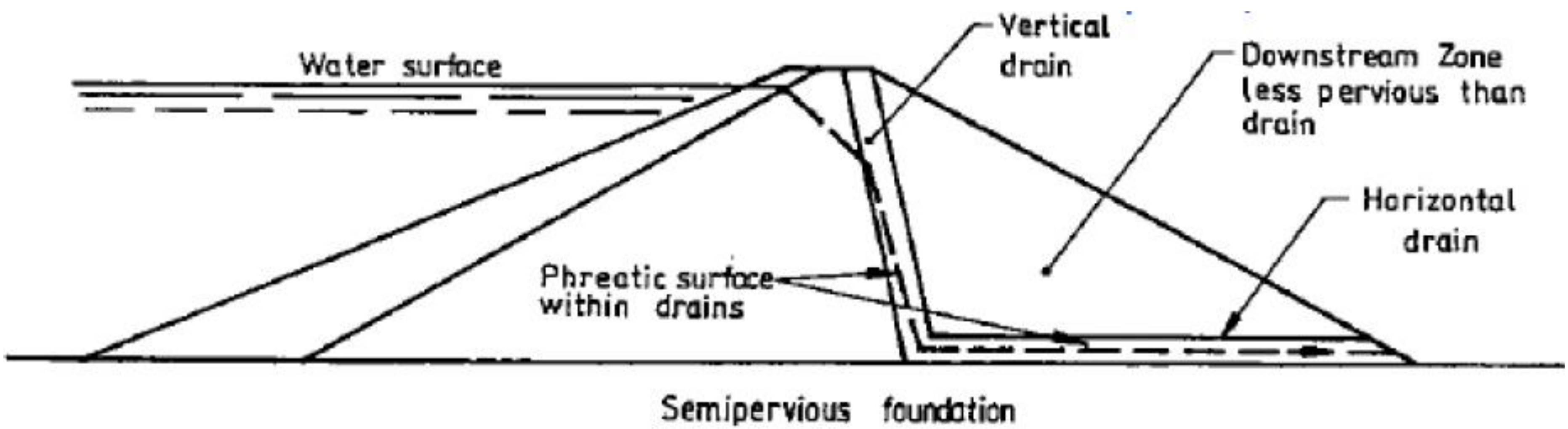
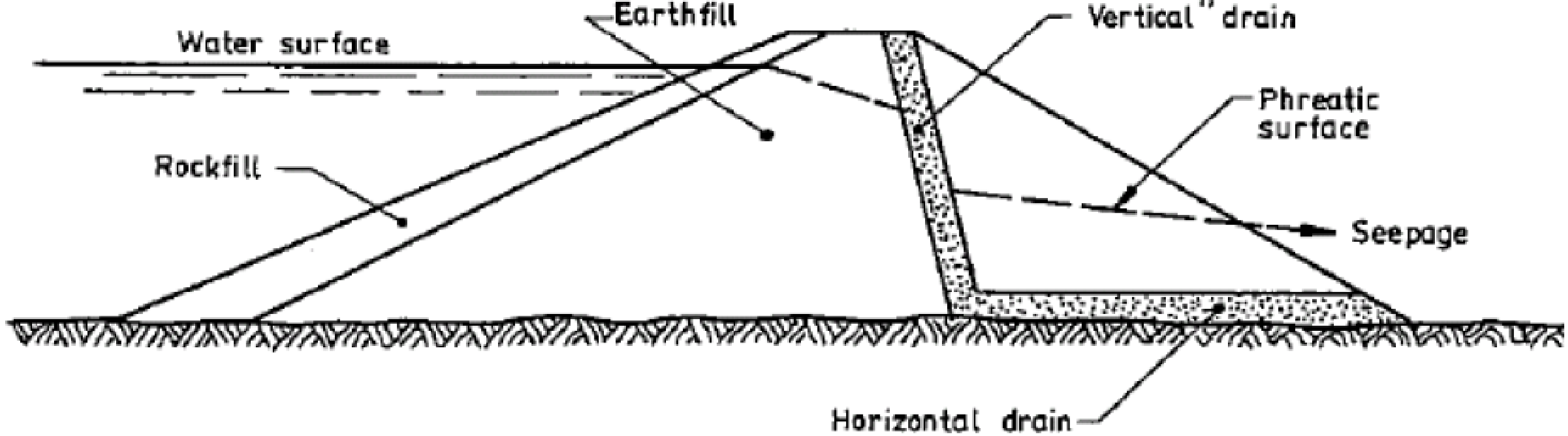


Fig Earth dam with internal drains of sufficient capacity



Earth dam with internal drains of inadequate capacity

- Cedergren (1972) gives a design method for estimating the discharge capacity of a horizontal drain based on:

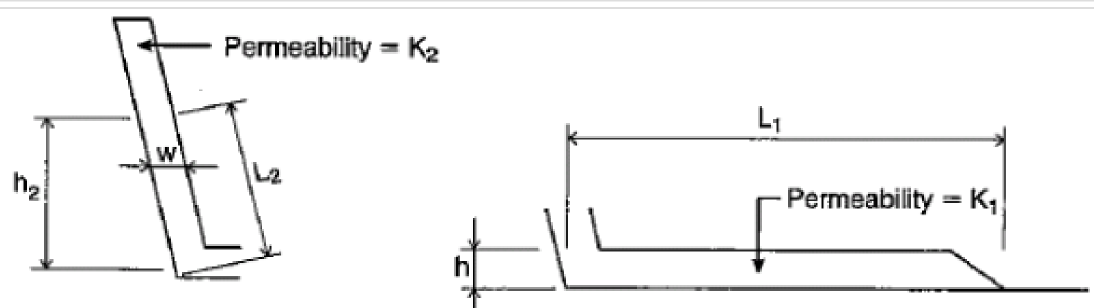
$$\boxed{q = \frac{k_1 h^2}{2L_1}}$$

k_1: permeability of drain material – m/s.

h: vertical thickness of drain – m.

L_1: length of the drain – m.

q: discharge capacity of drain per meter width of drain – $m^3/s/m$.



(a) Vertical drain (b) Horizontal drain

- The capacity of the vertical drain is seldom a critical issue, because the quantity of seepage through the earthfill is small and the vertical drain width is dictated by construction factors. However, its capacity should be checked by:

$$\boxed{q = \frac{k_2 h_2 w}{L_2}}$$

k_2: permeability of drain material – m/s.

h_2, L_2: are as shown in the above figure – m.

w: width of drain – m.

q: discharge capacity of drain per meter width of drain – $m^3/s/m$.

Filter Design and Construction

- Filters in embankment dams and their foundations are required to perform two basic functions:
  A)Prevent erosion of soil particles from the soil they are protecting.
  B)Allow drainage of seepage water.
- Filters are usually identified in terms of their particle size distribution.
- They are required to be sufficiently fine, relative to the particle size they are protecting (the "base soil"), to achieve function A, while being sufficiently coarse to achieve function B.

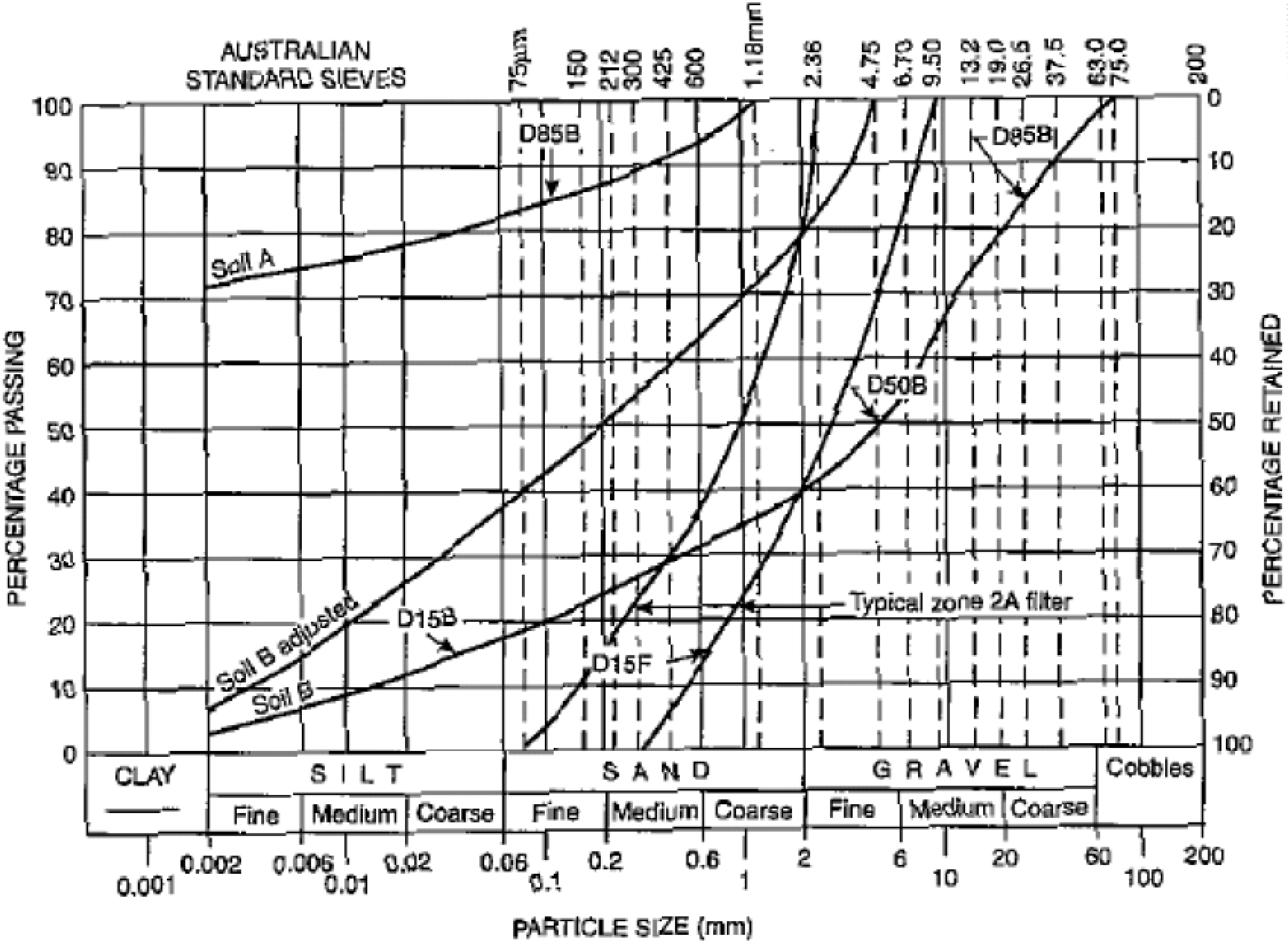
a) Filter Design Criteria

- Filter design practice have basically evolved from the concepts of Terzhagi (1926), who proposed that:

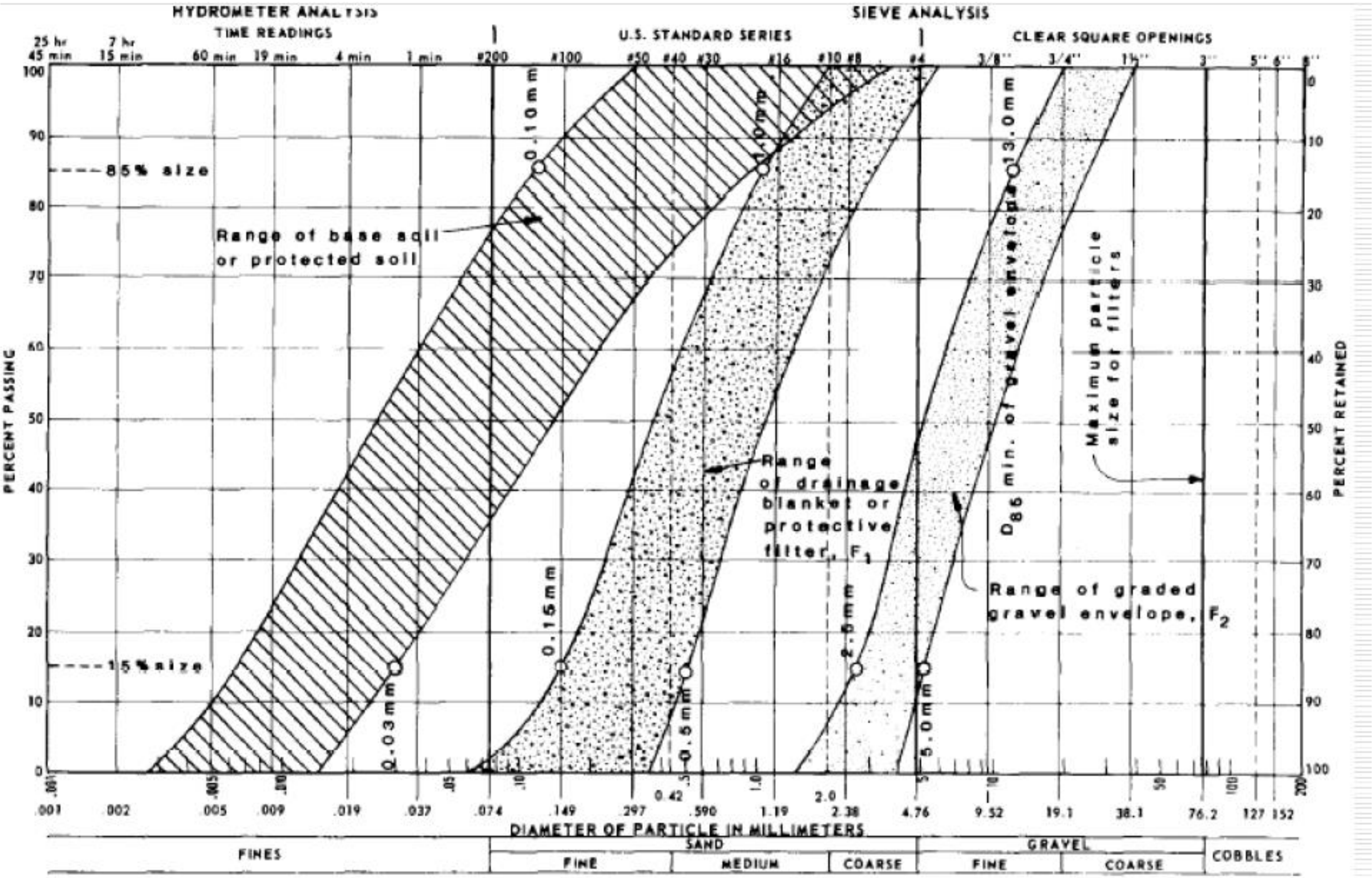
$$\boxed{D_{15F}/D_{85B} \leq 4}$$ to protect erosion.

$$\boxed{D_{15F}/D_{15B} \geq 4}$$ to ensure sufficient permeability.

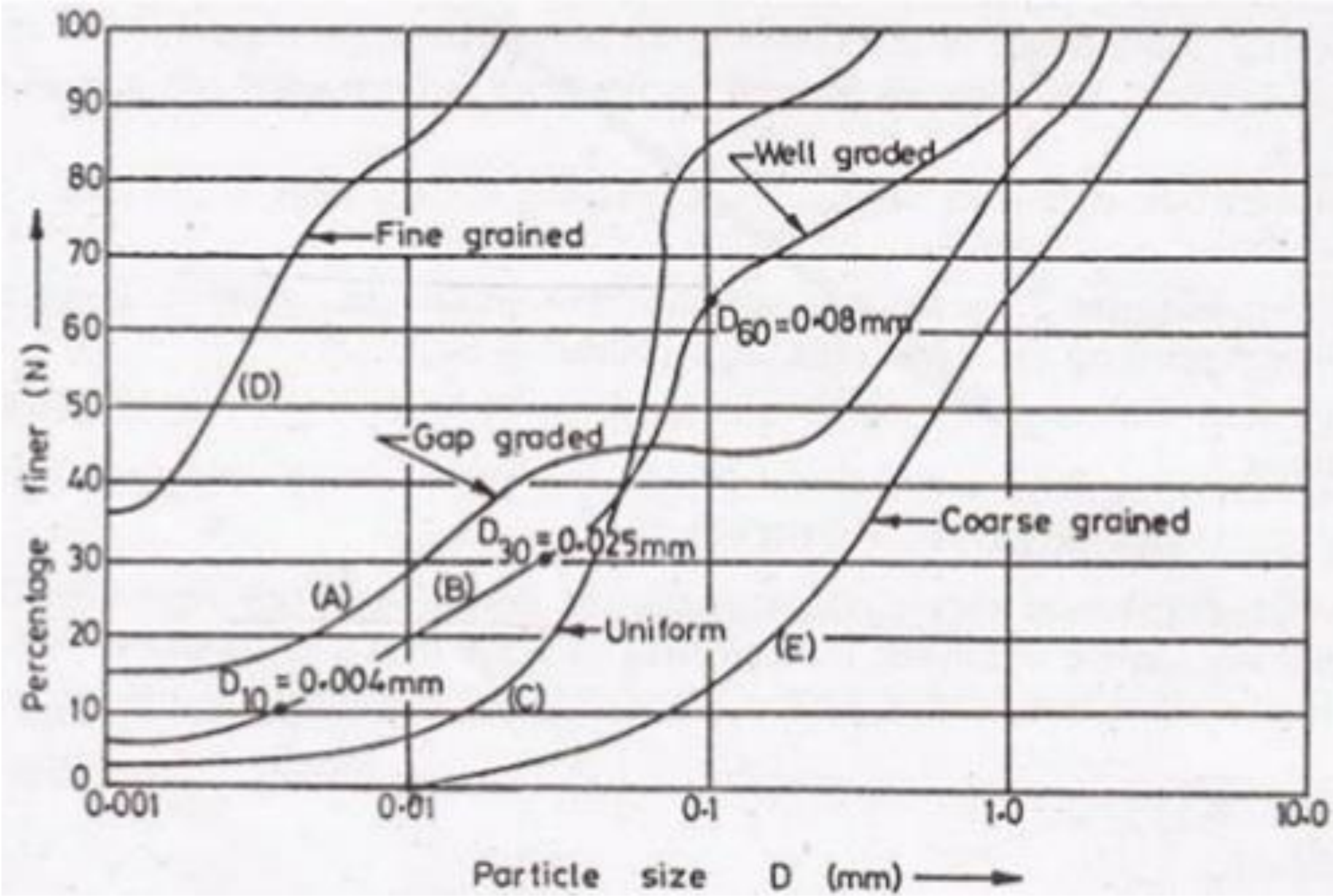
D_{15} is the size at which 15 percent of the total soil particles are smaller; the percentage is by weight as determined by mechanical analysis



Filter design notation and adjustment of the particle size distribution for gravelly base soils.



HYDROMETER ANALYSIS
TIME READINGS
25 hr 45 min
7 hr 15 min
60 min
19 min
4 min
1 min
SIEVE ANALYSIS
U.S. STANDARD SERIES
#200 #100 #50 #40 #30 #16 #10 #8 #4
CLEAR SQUARE OPENINGS
3/8" 3/4" 1½" 3" 5" 6" 8"
PERCENT PASSING
PERCENT RETAINED
85% size
15% size
Range of base soil or protected soil
0.10mm
1.0mm
0.03mm
0.15mm
0.5mm
2.5mm
5.0mm
Range of drainage blanket or protective filter, F_1
D_{85} min. of gravel envelope 13.0mm
Maximum particle size for filters
Range of graded gravel envelope, F_2
.001 .002 .005 .009 .019 .037 .074 .149 .297 0.42 .590 1.19 2.0 2.38 4.76 9.52 19.1 38.1 76.2 127 152
DIAMETER OF PARTICLE IN MILLIMETERS
FINES
SAND
FINE
MEDIUM
COARSE
GRAVEL
FINE
COARSE
COBBLES



100
90
80
70
60
50
40
30
20
10
0
Percentage finer (N)
0·001
0·01
0·1
1·0
10·0
Particle size D (mm)
Fine grained
(D)
Well graded
D60 = 0·08 mm
Gap graded
D30 = 0·025 mm
(A)
(B)
Uniform
(C)
D10 = 0·004 mm
(E)
Coarse grained

Several filter design methods are currently available that are based on extensive laboratory test results (eg. by USBR and USACE).

USBR Method

- The USBR method was widely used for design

The original USBR method (1977) is:

(i) a) D_{15F} / D_{15B} = 5 to 40 provided that;

b) the filter does not contain more than 5% fines passing 0.075 mm, and the fines should be cohesionless.

(ii) $D_{15F} / D_{85B} \leq 5$;

(iii) The grain size curve of the filter should be roughly parallel to that of the base soil.

(iv) Maximum size particles in filter = 75 mm to prevent segregation during placement.

(v) For base materials which include gravel particles, the base material D_{15B} and D_{85B}, etc. should be on the bases of the gradation of the soil finer than 4.75 mm.

Currently widely USBR method (1987) is:

(Recommended Method)

(i) a) $D_{15F} / D_{15B} \geq 5$ provided that;

b) the filter does not contain more than 5% fines passing 0.074 mm (no. 200 sieve).

(ii) $D_{15F} / D_{85B} \leq 5$;

(iii) Generally the filter should be uniformly graded to provide adequate permeability and to prevent segregation. Maximum size particles in filter = 3 inch (76.2 mm).

(iv) D_{85F}/ Maximum opening of pipe drain ≥ 2.

(v) For base materials which include gravel particles, the base material D_{15B} and D_{85B}, etc. should be on the bases of the gradation of the soil finer than sieve no. 4 (4.76 mm).

Filter design steps (based on Sherard's research, FEMA, and other studies)

Step 1: **Plot the gradation curve (grain-size distribution) of the base soil material**

- Use enough samples to define the range of grain sizes for the base soil or soils.

Step 2: **Proceed to step 4 if the base soil contains no gravel (material larger than No. 4 sieve).**

Step 3: **Prepare adjusted gradation curves for base soils that have particles larger than the No. 4 (4.75 mm) sieve.**

Step 4: **Place the base soil in a category determined by the percent passing the No. 200 (0.075 mm) sieve from the regraded gradation curve data according to table 26–1.**

Table 26–1 Regraded gradation curve data

Base soil category	% finer than No. 200 sieve (0.075 mm) (after regrading, where applicable)	Base soil description
1	> 85	Fine silt and clays
2	40 – 85	Sands, silts, clays, and silty & clayey sands
3	15 – 39	Silty & clayey sands and gravel
4	< 15	Sands and gravel

Step 5: To satisfy filtration requirements, determine the maximum allowable D15 size for the filter in accordance with the table 26–2.

Label the maximum D^{15} size **Control point 1**

Table 26–2 Filtering criteria — Maximum D_{15}

Base soil category	Filtering criteria
1	$\leq 9 \times d_{85}$ but not less than 0.2 mm
2	≤ 0.7 mm
3	$\leq \left(\frac{40-A}{40-15}\right)\left[\left(4 \times d_{85}\right)-0.7\text{mm}\right]+0.7\text{mm}$ A = % passing #200 sieve after regrading (If $4 \times d_{85}$ is less than 0.7 mm, use 0.7 mm)
4	$\leq 4 \times d_{85}$ of base soil after regrading

Step 6: **If permeability is a requirement, determine the minimum allowable D^{15} in accordance with table 26–3. Note: The permeability requirement is determined from the d15 size of the base soil gradation before regrading.**

Label the minimum D15 size **Control point 2**.

Table 26–3 Permeability criteria

Base soil category	Minimum D_{15}
All categories	$\geq 4 \times d_{15}$ of the base soil before regrading, but not less than 0.1 mm

Step 7: **The width of the allowable filter design band must be kept relatively narrow to prevent the use of possibly gap-graded filters. Adjust the maximum and minimum D^{15} sizes for the filter band determined in steps 5 and 6 so that the ratio is 5 or less at any given percentage passing of 60 or less. Criteria are summarized in table 26–4.**

Table 26–4 Other filter design criteria

Design element	Criteria
To prevent gap-graded filters	The width of the designed filter band should be such that the ratio of the maximum diameter to the minimum diameter at any given percent passing value ≤ 60% is ≤ 5.
Filter band limits	Coarse and fine limits of a filter band should each have a coefficient of uniformity of 6 or less.

Step 8: **The designed filter band must not have an extremely broad range of particle sizes to prevent the use of possibly gap-graded filters. Adjust the limits of the design filter band so that the coarse and fine sides have a coefficient of uniformity of 6 or less. The width of the filter band should be such that the ratio of maximum to minimum diameters is less than or equal to 5 for all percent passing values of 60 or less.**

$$CU = \frac{D_{60}}{D_{10}} \leq 6$$

Calculate a maximum D^{10} value equal to the maximum D^{15} size divided by 1.2. (This factor of 1.2 is based on the assumption that the slope of the line connecting D^{15} and D^{10} should be on a coefficient of uniformity of about 6)

- Calculate the maximum permissible D^{60} size by multiplying the maximum D^{10} value by 6. Label this **Control point 3**

- Determine the minimum allowable D^{60} size for the fine side of the band by dividing the determined maximum D^{60} size by 5. Label this **Control point 4**.

Step 9: Determine the minimum D^5 and maximum D^{100} sizes of the filter according to table 26–5.

Label as **Control points 5 and 6**, respectively

Table 26–5 Maximum and minimum particle size criteria*

Base soil category	Maximum D_{100}	Minimum D_5, mm
All categories	≤ 3 inches (75 mm)	0.075 mm (No. 200 sieve)

* The minus No. 40 (.425 mm) material for all filters must be nonplastic as determined in accordance with ASTM D4318.

Step 10: To minimize segregation during construction, the relationship between the maximum D^{90} and the minimum D^{10} of the filter is important.

Calculate a preliminary minimum D^{10} size by dividing the minimum D^{15} size by 1.2

Determine the maximum D^{90} using table 26–6. Label this as **Control point 7.**

Table 26–6 Segregation criteria

Base soil category	If D_{10} is :	Then maximum D_{90} is:
	(mm)	(mm)
All categories	< 0.5	20
	0.5 – 1.0	25
	1.0 – 2.0	30
	2.0 – 5.0	40
	5.0 – 10	50
	> 10	60

Table 26–7 Criteria for filters used adjacent to perforated collector pipe

Noncritical drains where surging or gradient reversal is not anticipated	The filter D_{85} must be greater than or equal to the perforation size
Critical drains where surging or gradient reversal is anticipated	The filter D_{15} must be greater than or equal to the perforation size.

Step 11: Connect Control points 4, 2, and 5 to form a partial design for the fine side of the filter band. Connect Control points 6, 7, 3, and 1 to form a design for the coarse side of the filter band. This results in a preliminary design for a filter band. Complete the design by extrapolating the coarse and fine curves to the 100 percent finer value.

Step 12: Design filters adjacent to perforated pipe to have a D^{85} size no smaller than shown in table 26–7.

Example for filter design

Given the following base soil data, determine filter gradation limits for the base soil.
The most important function of the filter is to act as a filter.

Sieve size	% passing
No 10	100
No 200	90
0.05mm	80
0.02mm	60
0.005mm	40
0.002mm	32

FINES	SANDS												GRAVELS						COBBLES		
SIEVE OPENING, (mm)	(0.075)	(0.105)	(0.149)	(0.250)	(0.297)	(0.42)	(0.59)	(0.84)	(1.19)	(2.0)	(2.38)	(4.76)	(9.525)	(12.7)	(19.05)	(25.4)	(38.1)	(50.8)	(76.2)	(152.4)	(304.8)
U.S. STANDARD SIEVE SIZE	#200	#140	#100	#60	#50	#40	#30	#20	#16	#10	#8	#4	3/8"	1/2"	3/4"	1"	1 1/2"	2"	3"	6"	12"

Solution

Step 1.
Plot the gradation curve of the base soil
Use excel for this.

Step 2
Check if base soil contains no gravel (>No 4 sieve size or 4.75mm size)
Given base soil has 100% finer than No 4 sieve size.
No need to re-grade (Proceed to step 4)

Step 4
Identify the category of the base soil based on the % passing No 200 (0.075mm) sieve and Table 26-1
The given soil has 90% finer than No 200 sieve size and from table 26-1
Soil is in category 1.

Step 5

Determine maximum allowable D15 size for the filter (Table 26-2)

Max D15 filter <=9D85 of base soil but not less than 0.2mm

<=9*0.06=0.54mm (not <0.2mm)

Take this as control point 1

Step 6

Determine minimum D15 of filter (permeability requirement and Table 26-3)

Minimum D15 of filter>=4*D15 base soil, bot not less than 0.1mm

The given base soil doesn't have a meaningful D15 size. The data show that the base soil has 32% finer than 0.002mm, the smallest particle size. Therefore, use the default value of 0.1mm for the minimum D15 of the filter.

(control point 2)

Therefore, preliminary minimum D15 filter= 0.1mm (control point 2)

Step 7

Preventing the use of possibly gap graded filter

i.e (Max. D15 size/ Min. D15 size) <=5 (Table 26-4)

In this case, 0.54/0.1= 5.4 (slightly greater than 5)

A slight adjustment is needed. The minimum D15 is the control because filtering is stated as the most important purpose (prevention of migration of fine materials)

Hence, adjust max D15 size.

Adjusted max D15 size of filter = 0.1*5= 0.5mm

This is the final control point 1.

Step 8

Uniformity coefficient criteria

$$Cu = \frac{D60}{D10} \leq 6$$

Determine max. D60 size of the filter. To do this, divide max.D15 size of filter by 1.2 (the factor 1.2 is based on the assumption that the slope of the line connecting D15 and D10 should be on a coefficient of uniformity of about 6.

Max. D10=Max.D15/1.2= 0.42mm

Max. D60 size = 0.42*6=2.5mm

This is control point 3.

Determine min. D60 size of the filter (table 26-4)

(Max D60/Min.D60)<=5 (The width of the designed filter band should be such that the ratio of the max. diameter to the min. diameter at any given % passing value <=60% is <=5)

Hence, min. D60= 2.5/5=0.5mm

This is control point 4.

Step 9

determine min. D5 and max. D100 sizes of the filter (Table 26-5)

Min. D5 value = 0.075mm

This is control point 5

Max. D100 size of filter = 3 inch or 75mm

This is control point 6

Step 10

Determine max. D90 of filter (Minimize segregation during construction (Table 26-6)

To do this, determine min. D10 size bydividing the minimum D15 size by 1.2.

Minimum D10 size= min. D15 size/1.2= 0.1/1.2= 0.083mm

From Table 26-6, since min. D10<0.5mm, Max D90 size= 20mm

This is control point 7

Step 11

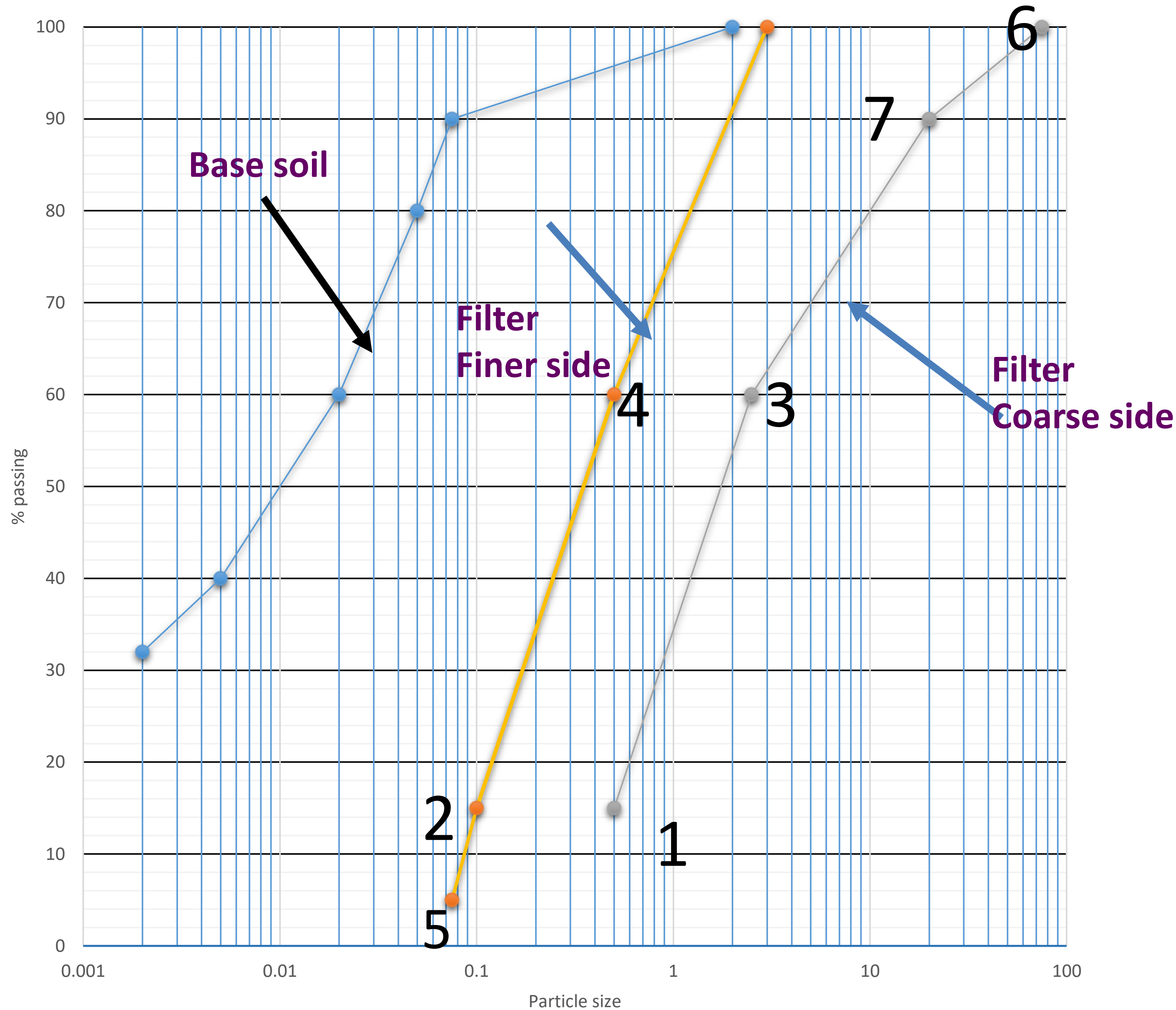
Connect control points 4,2 and 5 for the fine side of the filter band

Connect control points 6,7,3 and 1 for the coarse side of the filter band.

Step 12

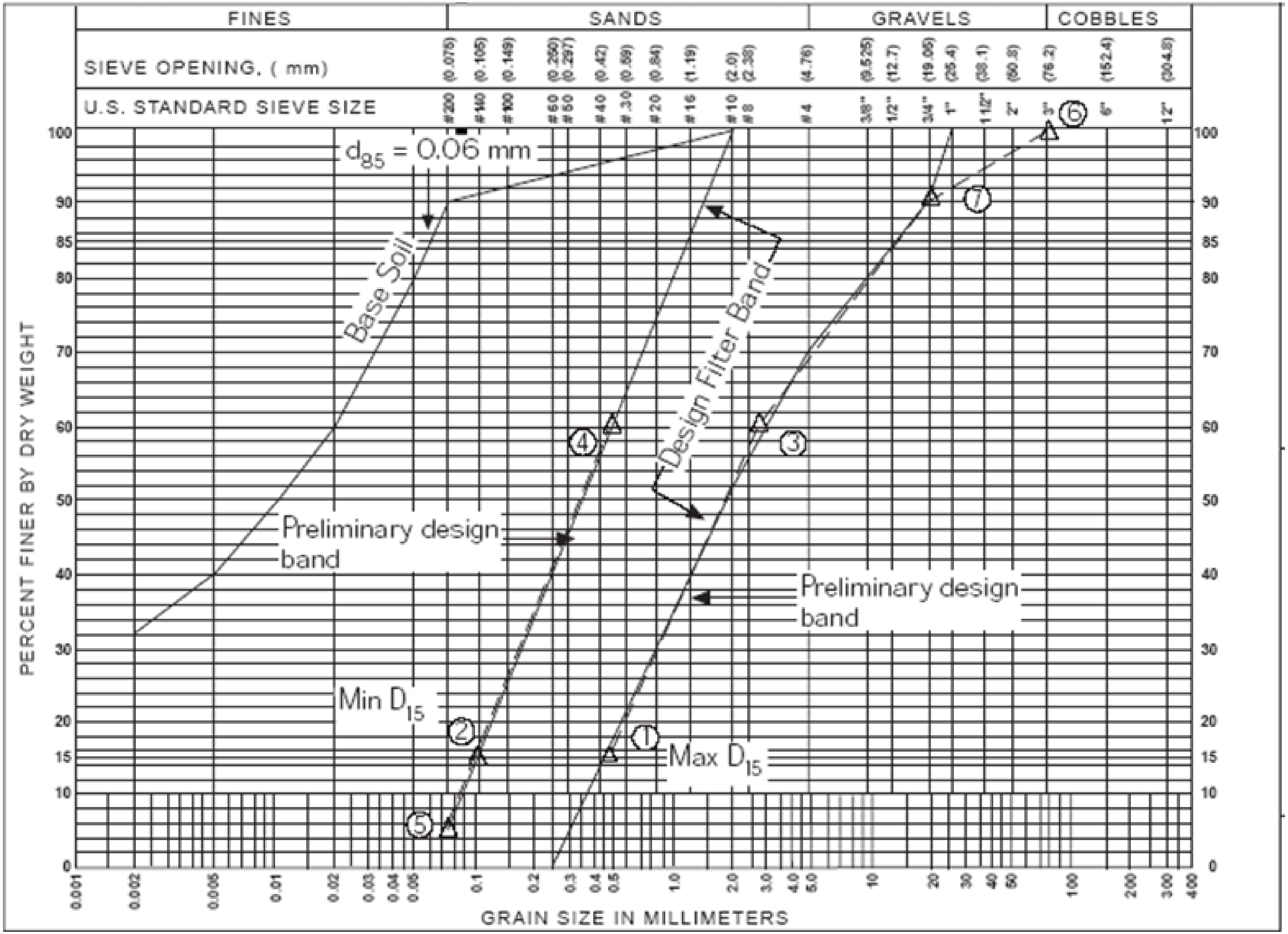
For this design the filter will not be used around a perforated pipe (this step is not applicable)

See the graph on the next slide



100
90
80
70
60
50
40
30
20
10
0
% passing
0.001
0.01
0.1
1
10
100
Particle size
Base soil
Filter
Finer side
Filter
Coarse side
1
2
3
4
5
6
7

FOR THE EXAMPLE



Slope Protection

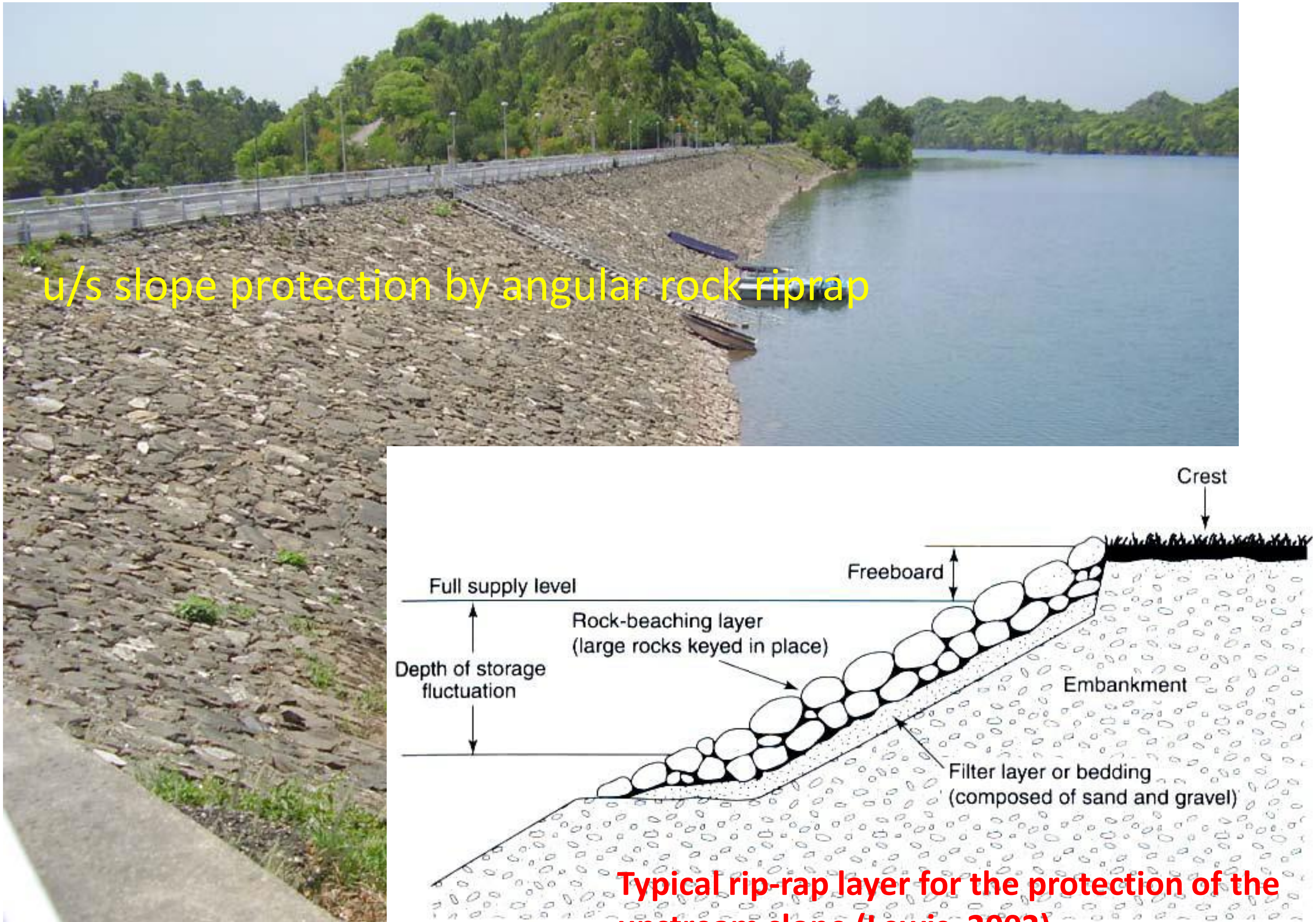
Upstream slope protection

- The upstream slopes of earthfill and earth & rockfill dams need protection from erosion by wave action on the reservoir.

Methods of protecting upstream slopes include:

- dumped riprap
- precast and cast-in-place concrete pavements
- soil cement and shotcrete, etc.

Modern dams are generally protected by dumped rock, known as dumped rip-rap or simply rip-rap.



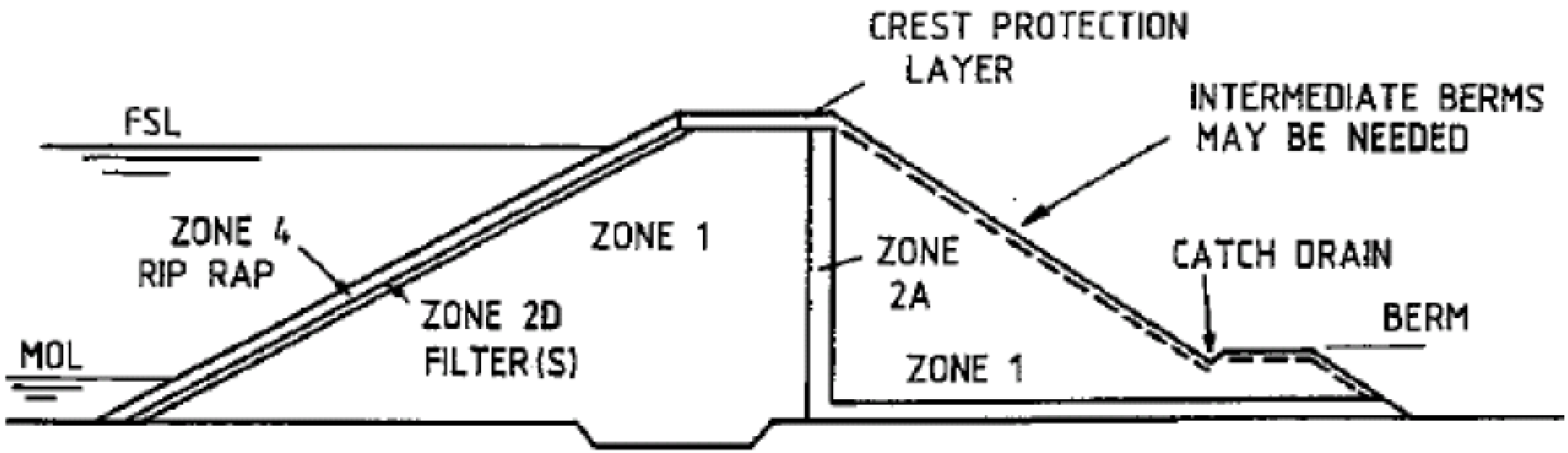
Typical rip-rap layer for the protection of the upstream slope (Lewis, 2002).

Upstream slope protection (ctd.)

General requirements:

Rip-rap comprises quarried blocks of rock which have to be:

- Large enough to dissipate the energy of the waves without being displaced;
- Strong enough to do this without abrading or without breaking down to smaller sizes;
- Durable enough to withstand the effects of long term exposure to the weather and varying periods of inundation without becoming weaker and, hence, wearing or breaking down to smaller sizes.



For earthfill dams, the rip-rap is constructed as a separate layer and should be underlain by a filter to prevent erosion of the earthfill through the rip-rap as shown in Figure

Design of riprap

Placement

Dumping

- The riprap is dumped from hauling trucks onto the prepared surface

Hand Packed rock Riprap

- Laid by hand in a more or less definite pattern with minimum amount of voids and with top surface relatively smooth.

Sizing and layer thickness

- The sizing of rock needed for rip-rap and the layer thickness required are determined from the size of the waves expected.

I. Rip-Rap layer recommended by U.S. Corps of Engineers

Wave height (m)	Average rockfill diameter – D_{50} (m)	Layer thickness (m)
0 – 0.60	0.25	0.30
0.60 – 1.20	0.30	0.46
1.20 – 1.80	0.38	0.61
1.80 – 2.40	0.46	0.76
2.40 – 3.00	0.53	0.91

ii. Using equations (Rip-Rap layer) by Davis

$$W_{50} = \frac{\gamma H^a}{K (G-1)^3 \cdots (Cot\ \theta)^b}$$

$$W_{max} = 4 W_{50} \quad \text{and} \quad W_{min} = W_{50}/8$$

W_{50}	=	Average stone weight (lbs)
H	=	Wave height (ft)
γ	=	Stone unit weight (lbs/cft) (bulk unit weight after placement) ~ 156 lb/cft
G	=	Sp. Gravity of stones material (2.3 – 2.7)
θ	=	angle (degrees) of slope surface with horizontal
K	=	stability coefficient (K ~ 4.37)
a, b	=	empirical coefficient (In general coefficient are as: a = 3, b ~1)

iii. By Novak

$M = 103 \times Hs^3$ where M=mass of stone required (kg), and Hs =significant wave height (m).

The size of riprap is estimated as: $D = [7 W / 5 \gamma]^{1/3}$ where D = stone size (ft), W = stone weight (lbs), γ = bulk unit weight (lbs/cft).

iv) Dimensions of upstream rip-rap (French Guidelines on Small Dams)

Wave height (m)	Thickness - e (m)	Block diameter – D_{50} (m)
0.30	0.30	0.20
0.55	0.40	0.25
0.80	0.50	0.30
1.05	0.60	0.40
1.30	0.70	0.45
1.55	0.80	0.50

Design of filters under rip-rap

There are two requirements for filters under rip-rap:

- That they are coarse enough not to wash out of the rip-rap
- That they are fine enough to prevent erosion of the soil beneath the filter.

Upstream slope protection (ctd.)

-Generally, design of filter under rip-rap is not as critical as for ordinary Zone 2A filters. Thus, it is fairly common to use more relaxed filter criteria.

-If there is a reasonably well-graded sandy gravel /gravely sand, from 0.075 mm to 50 mm or 75 mm available, this should be satisfactory in most cases

- Sherard et.al. (1963) recommended the following minimum thickness:

Table 13.5. Minimum thicknesses of filters under rip-rap (from Sherard et al., 1963).

Wave height (m)	Minimum filter thickness (mm)
0–1.2	150
1.2–2.4	225
2.4–3.0	300

Downstream slope protection

General requirements

For earthfill dams, the downstream face is potentially erodible and considerable care needs to be taken to prevent erosion. This is done by:

- Covering the surface with a layer of rockfill or by establishing grass cover;
- Providing berms to limit the vertical distance over which runoff can concentrate;
- Providing lined drains on the berms to catch the runoff

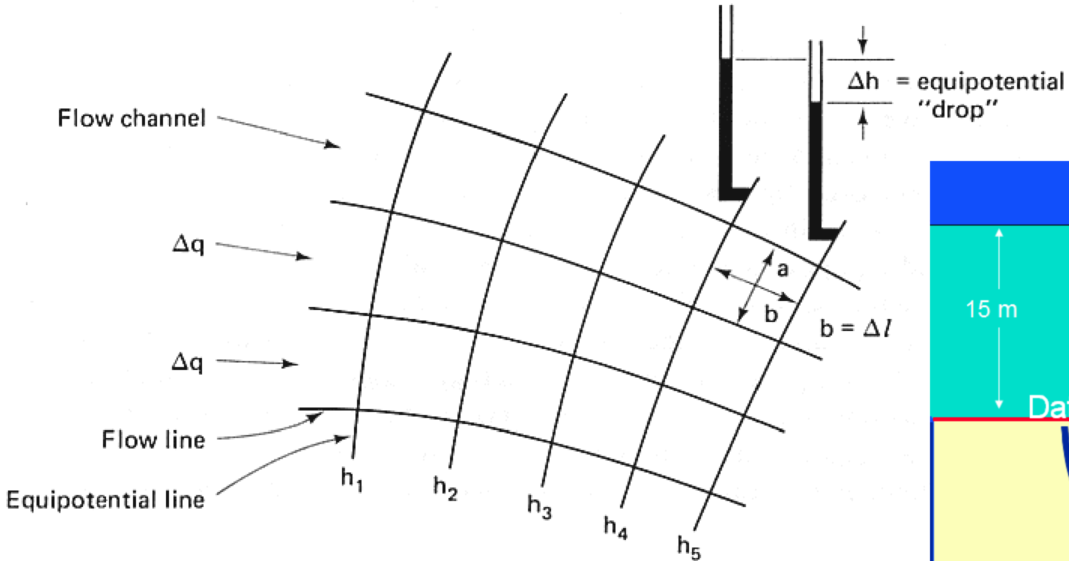
- Berms should be provided at no greater than 10 m vertical intervals

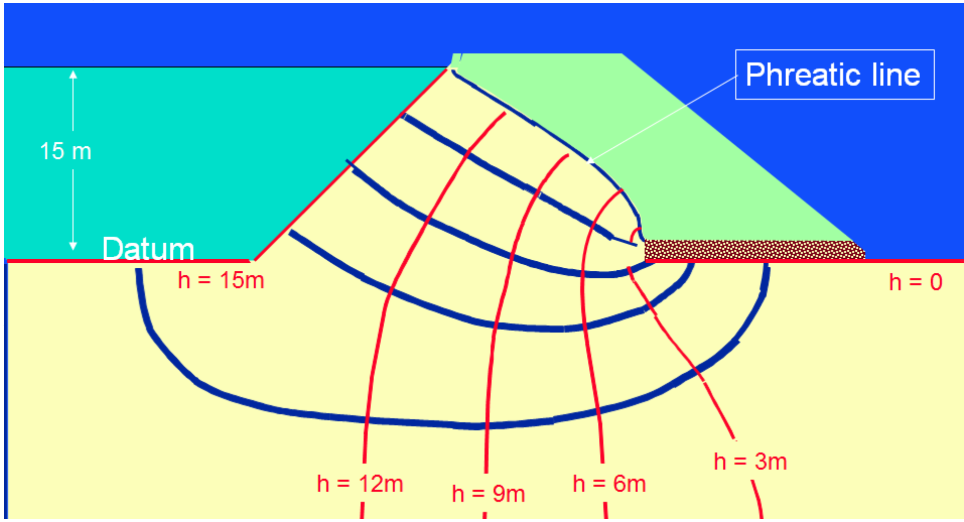
Seepage Through Dams

- The two dimensional flow of fluid through porous soil can be expressed by Laplace's equation

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

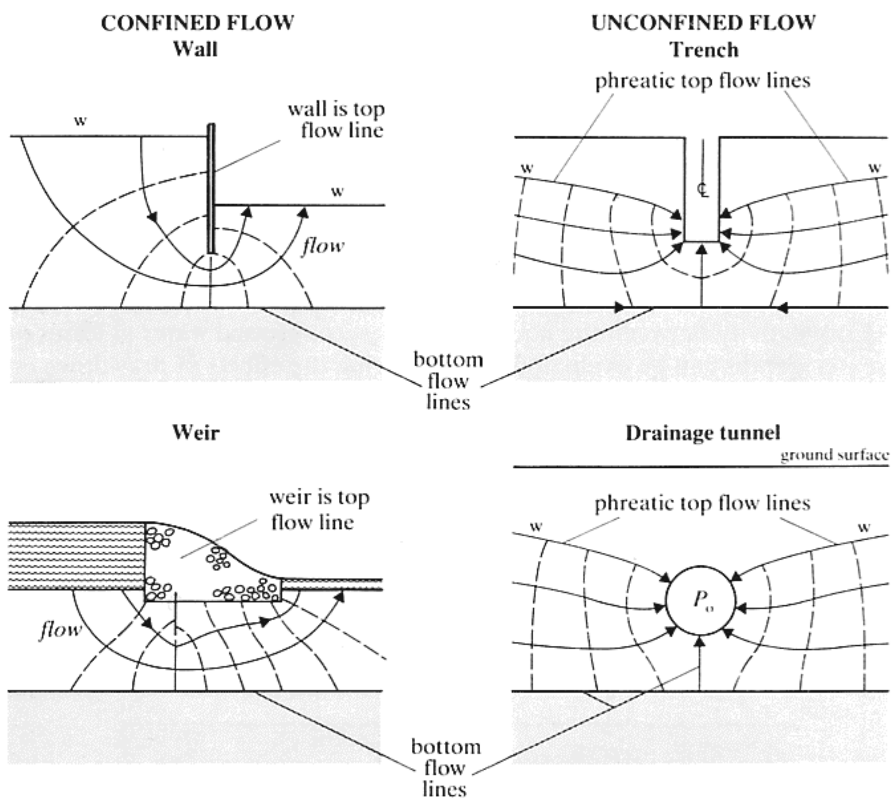
For most practical geotechnical problems, it is simpler to solve this equation graphically by drawing **flow nets**.





Flow Nets

- A flow net consists of two sets of curves – **equipotentials** and **flow lines** – that intersect each other at 90°.
- Along an equipotential, the total head is **constant**.
- A pair of adjacent flow lines define a **flow channel** through which the **rate of flow** of pore fluid is **constant**.
- The loss of head between two successive equipotentials is called the **equipotential drop**.



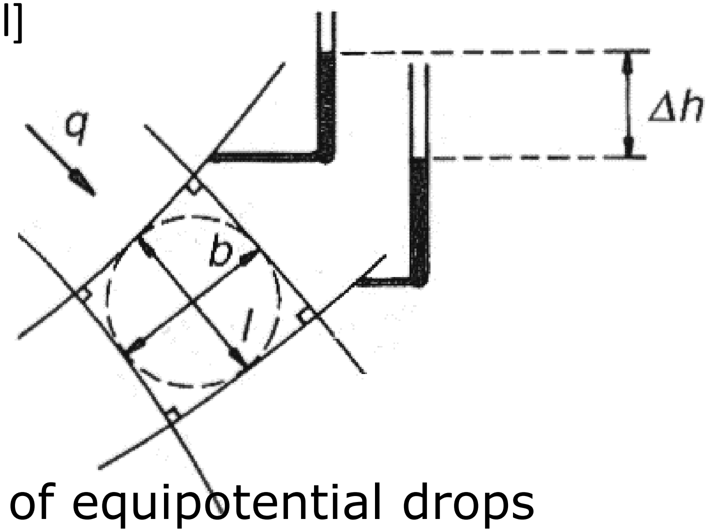
Typical Flow Nets

Flow Rate Calculation using a Flow Net

- Consider groundwater flow through a single flow element shown in the figure
- The flow rate through this element, **q**, is given by:

$$\boxed{q = k \cdot i \cdot A = k \cdot \frac{\Delta h}{l} \cdot b}$$

[k – permeability of soil]



- If the element is a curvilinear square, i.e. **b = l**, the above equation reduces to:

$$\boxed{q = k \cdot i \cdot A = k \cdot \Delta h}$$

For $\mathbf{N_F}$ number of flow channels, $\mathbf{N_H}$ number of equipotential drops and an overall head drop of **H**:

$$\boxed{q_T = q \cdot N_F} \quad \text{and} \quad \boxed{\Delta h = \frac{H}{N_H}}$$

Therefore, the expression for **flow rate per unit length**, **qT**, can be obtained as:

$$\boxed{q_T = k \cdot H \cdot \left(\frac{N_F}{N_H} \right)}$$

- The **total flow rate**, $\mathbf{Q_T}$, is given by:

$$\boxed{Q_T = q_T \cdot L}$$

L is the length perpendicular to the 2-D seepage plane

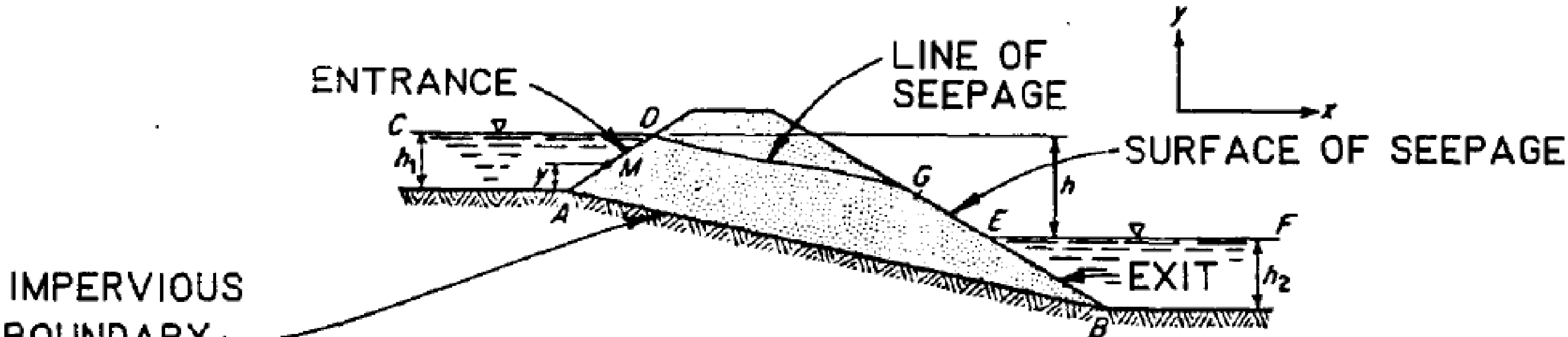
Entrances and Exits

The lines defining the area where water enters or leaves the pervious soil mass are known as **entrances or exits**, respectively

Entrances and exits are also called reservoir boundaries

Surface of Seepage

The saturated pervious soil mass may have a boundary exposed to the atmosphere and allow water to escape along this boundary, line GE, figure below. Pressure along this surface is atmospheric. The surface of seepage may also be called a seepage face

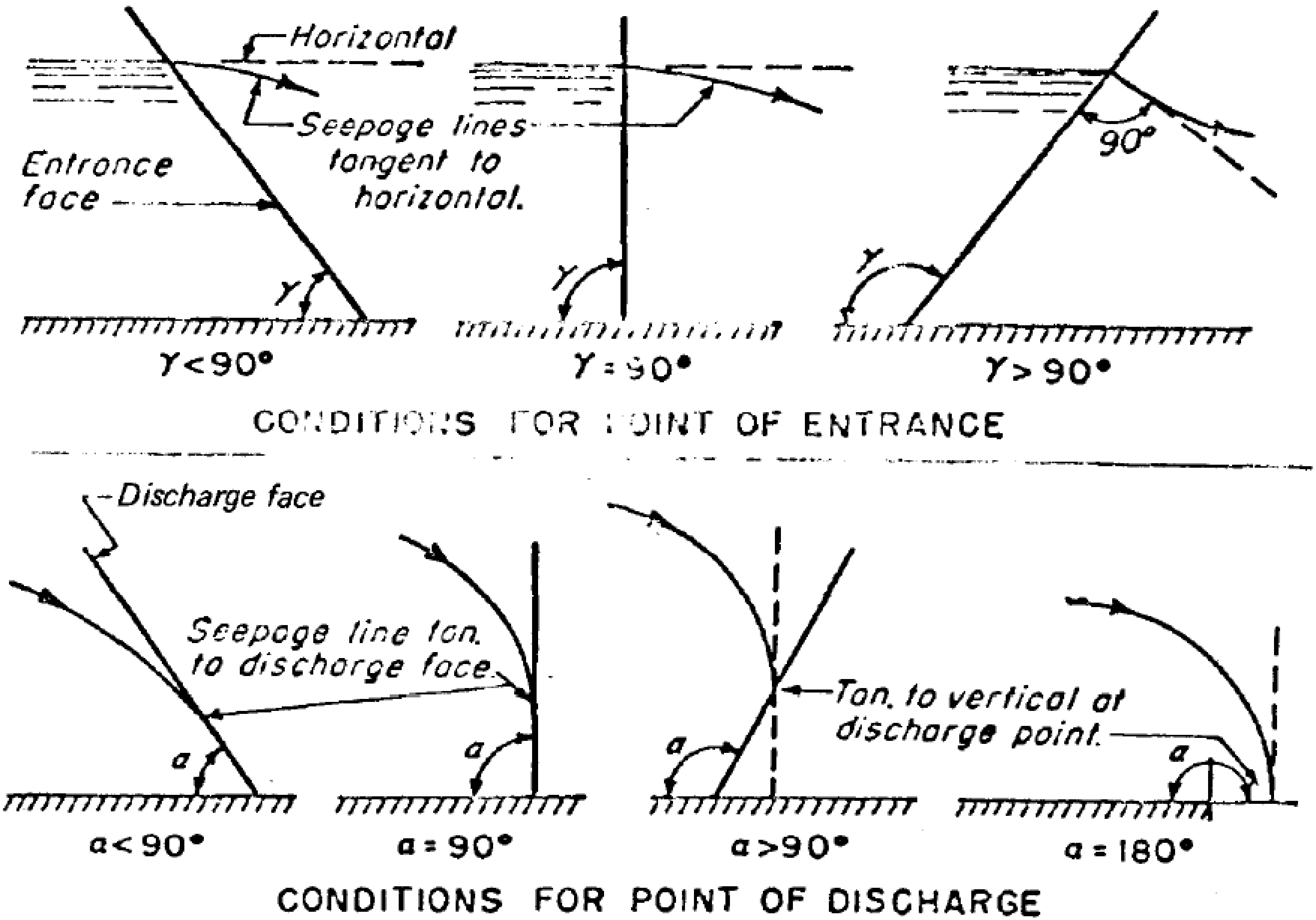


Examples of boundary conditions

Line of Seepage

- Known also as the free surface, this boundary is located **within the pervious soil** where water is at atmospheric pressure, line DG, figure above

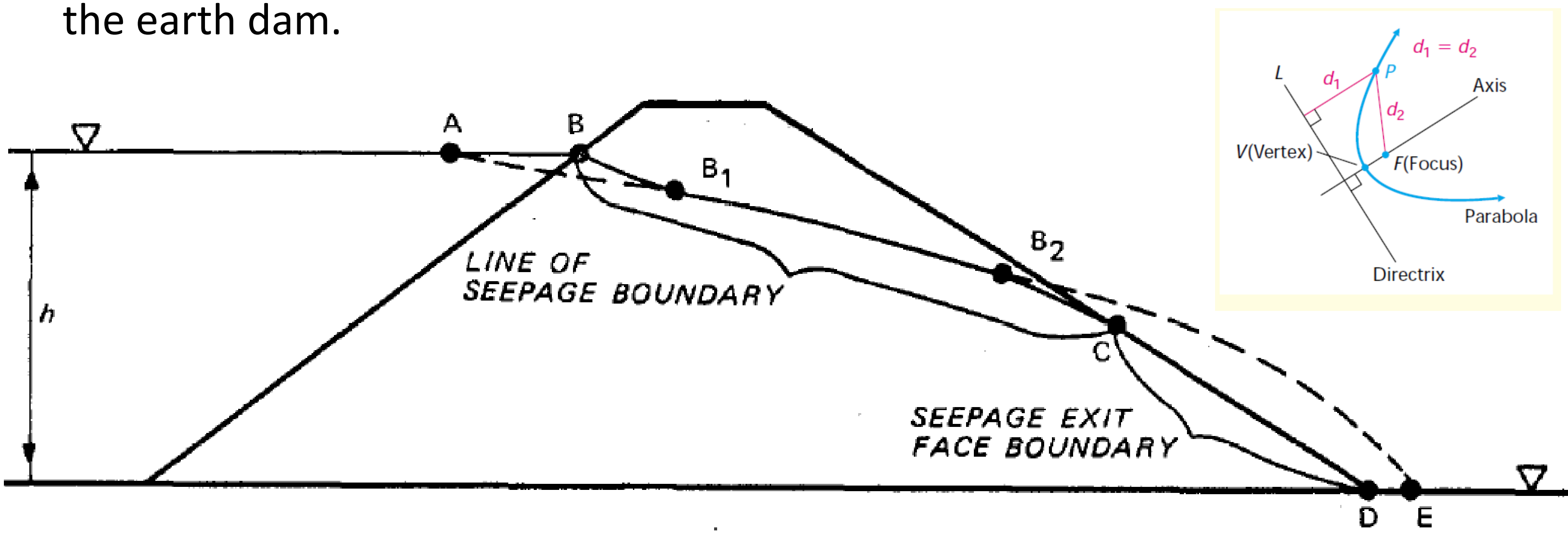
Entrance and exit conditions for a line of seepage (phreatic line)



Definition of Unknown Seepage Boundaries and Calculation of Flow per Unit Length of Embankment, q

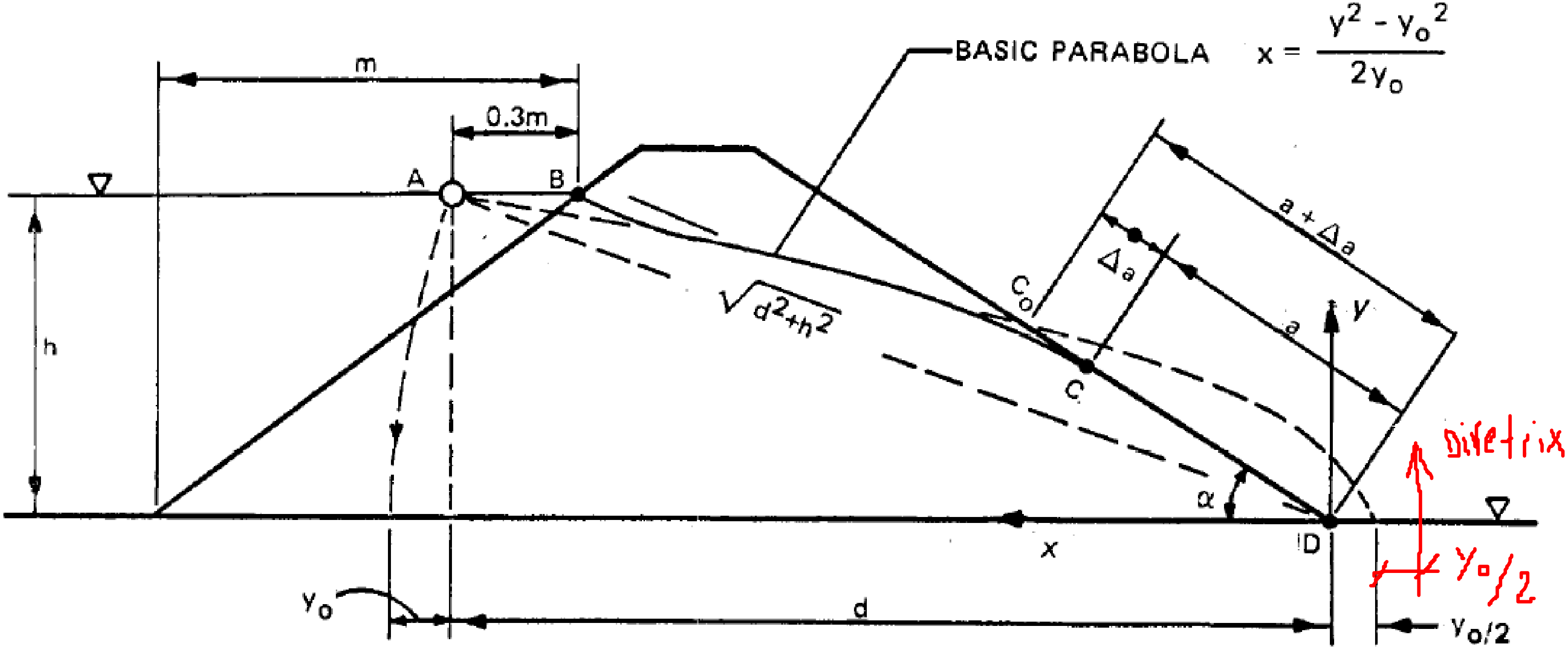
a. Homogeneous Earth Dam on Impervious Foundation

- It is desired to define the flow and pressure distribution within the embankment and total flow through the embankment
- first step is determination of the upper flow line (which is the line of seepage boundary) and the length of the seepage exit face on the downstream slope of the earth dam.



- A parabola, shown by the dashed line, is the basic geometric member used to define the location and extent of the two boundaries.

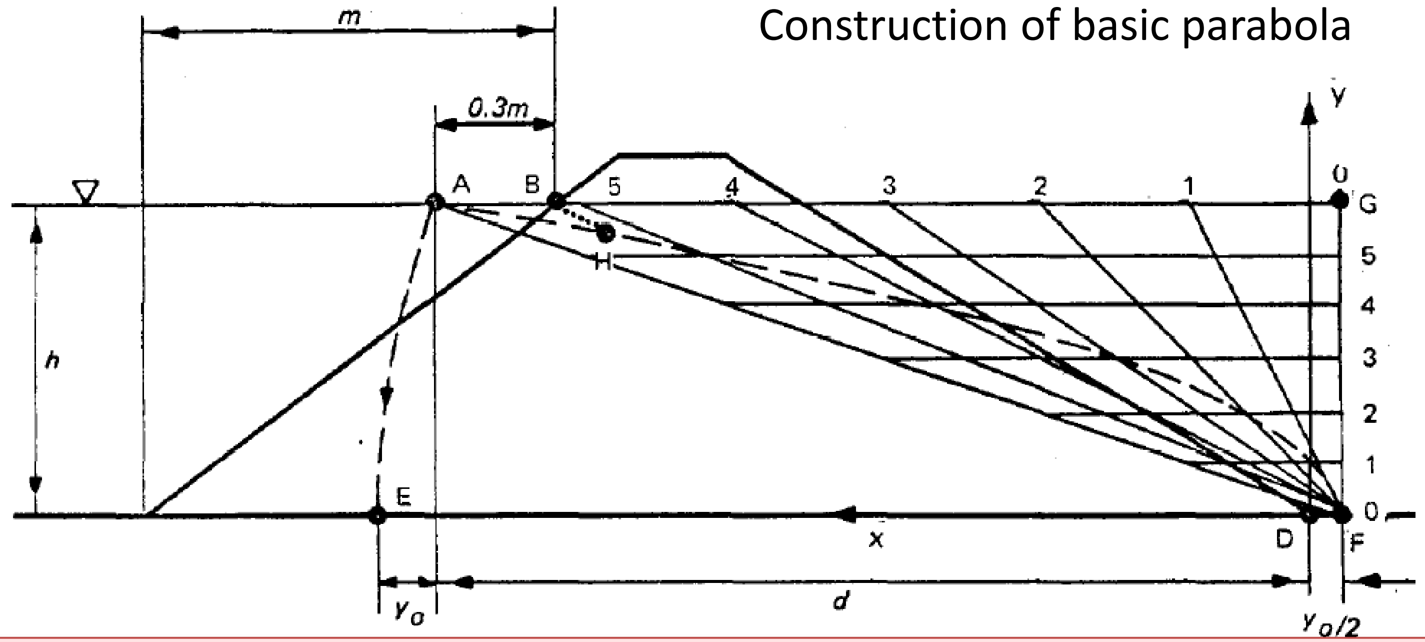
Determination of line of seepage and seepage exit face for embankments on impervious foundations



- Embankment geometry and head water elevation provide values for h , m and α which allow location of points A and B and determination of distance, d.

α	METHOD	EQUATIONS
$<30^\circ$	SCHAFFERNAK - VAN ITERSON	$a = \frac{d}{\cos\alpha} - \sqrt{\frac{d^2}{\cos^2\alpha} - \frac{h^2}{\sin^2\alpha}}$ $q = k\, a \sin\alpha \tan\alpha$
$\leqq 90^\circ$	L. CASAGRANDE	$a = S_o - \sqrt{S_o^2 - \frac{h^2}{\sin^2\alpha}}$ FOR $\alpha \leqq 60^\circ$, USE $S_o = \sqrt{d^2 + h^2}$. FOR $60^\circ < \alpha < 90^\circ$, USE MEASURED $S_o = \widehat{AC} + \overline{CD}$ $q = k\, a \sin^2\alpha$
180°	KOZENY	$a_o = \frac{y_o}{2} = \frac{1}{2}[\sqrt{d^2 + h^2} - d]$ $q = 2ka_o = ky_o$
30° TO 180°-	A. CASAGRANDE	DETERMINE $(a + \Delta a)$ AS THE INTERSECTION OF THE BASIC PARABOLA AND DAM SLOPE. THEN DETERMINE Δa FROM C VALUE ON FIG. 6-5. $q = k\, a \sin^2\alpha$ OR $q = ky_o = k(\sqrt{d^2 + h^2} - d)$

i. α< 30º Schaffernak-Van Iterson



Construction of basic parabola

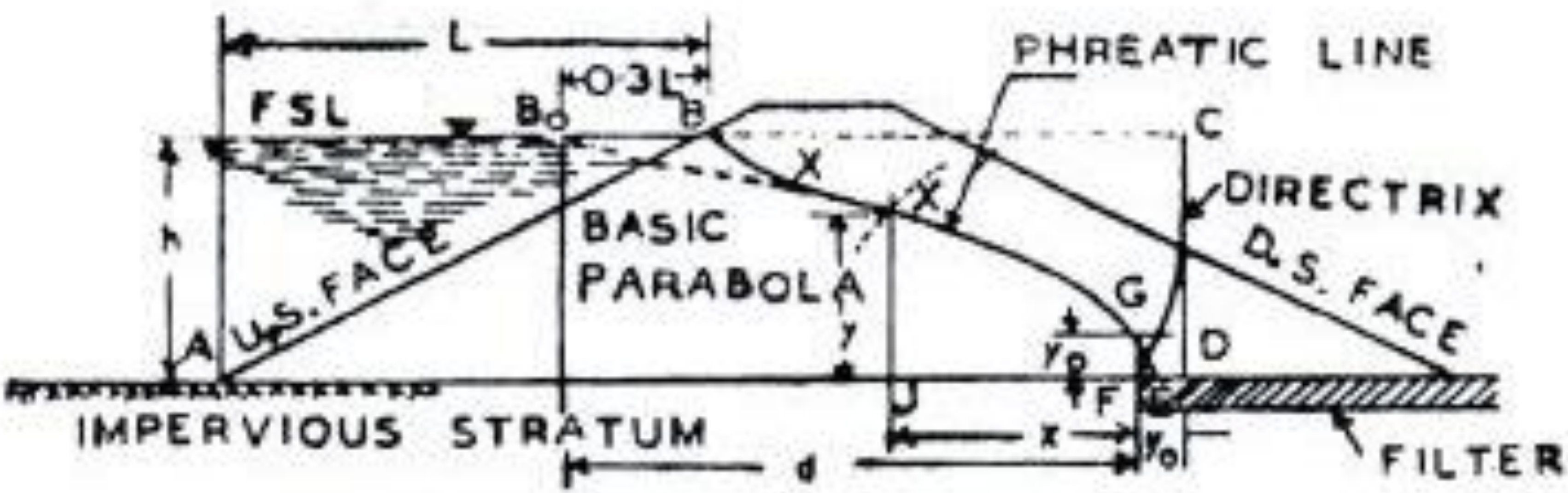
D is d/s end of dam and focus of parabola
F is vertex of parabola

- Direct determination of a and q
- Locate point A
- Locate d and y_o by scribing an arc, with radius DA through point E. Also locate F

- Line AG, parallel to the embankment base and horizontal axis of the parabola, is drawn and divided into an equal number of segments (6 in this case)

- Line GF, the vertical tangent to the parabola, located at yo/2 from the downstream toe of the embankment is divided into the same number of equal segments as line AG. The points dividing line AG into segments are connected with point F. The intersection of these lines with their counterpart lines drawn from the points on line GF define the parabola. Thus the basic parabola, dashed line A-F, is defined

ii. α = 180° Kozeny (***With horizontal drainage filter)***



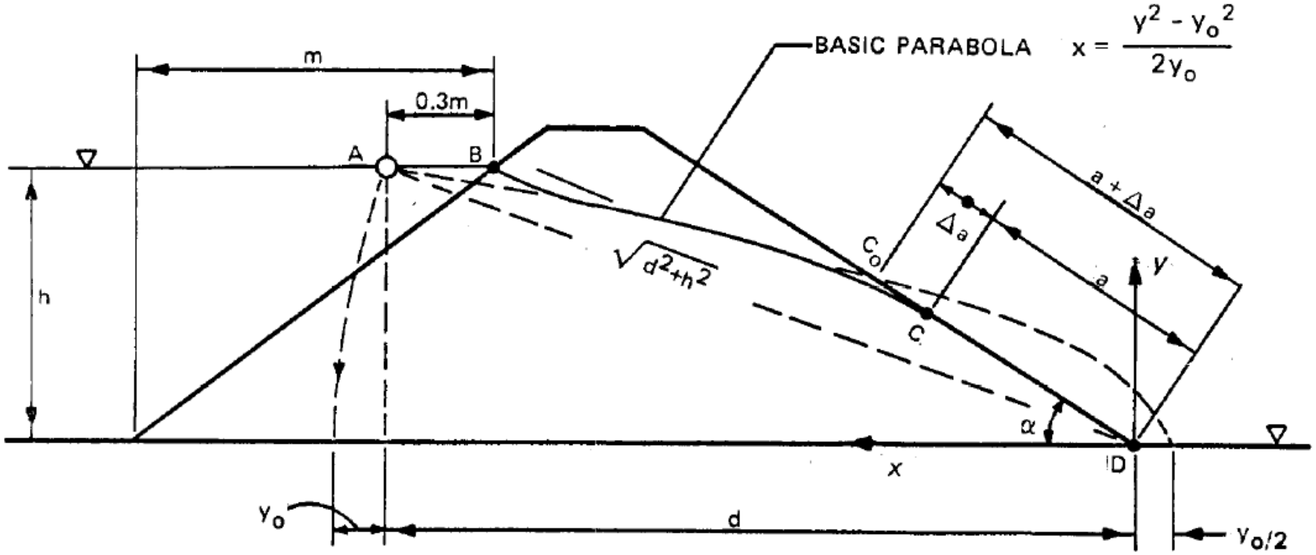
- Locate point B_0 on the water surface at a distance 0.3 L from B.
- The basic parabola has to pass through B_0 and have **its focus at F** which is the starting point of the horizontal drainage.
- With centre B_0 and radius $\mathbf{B_0F}$, draw an arc to meet the water line at **C**. Draw the vertical line **CD** which is the directrix. Let **FD**, the focal distance = yo. Bisect the distance **FD** to get the point **E**, the vertex of the parabola.
- if (x y) is one point on the parabola, $\sqrt{x^2 + y^2} = x + y_o$

$$y_o = \sqrt{d^2 + h^2} - d$$

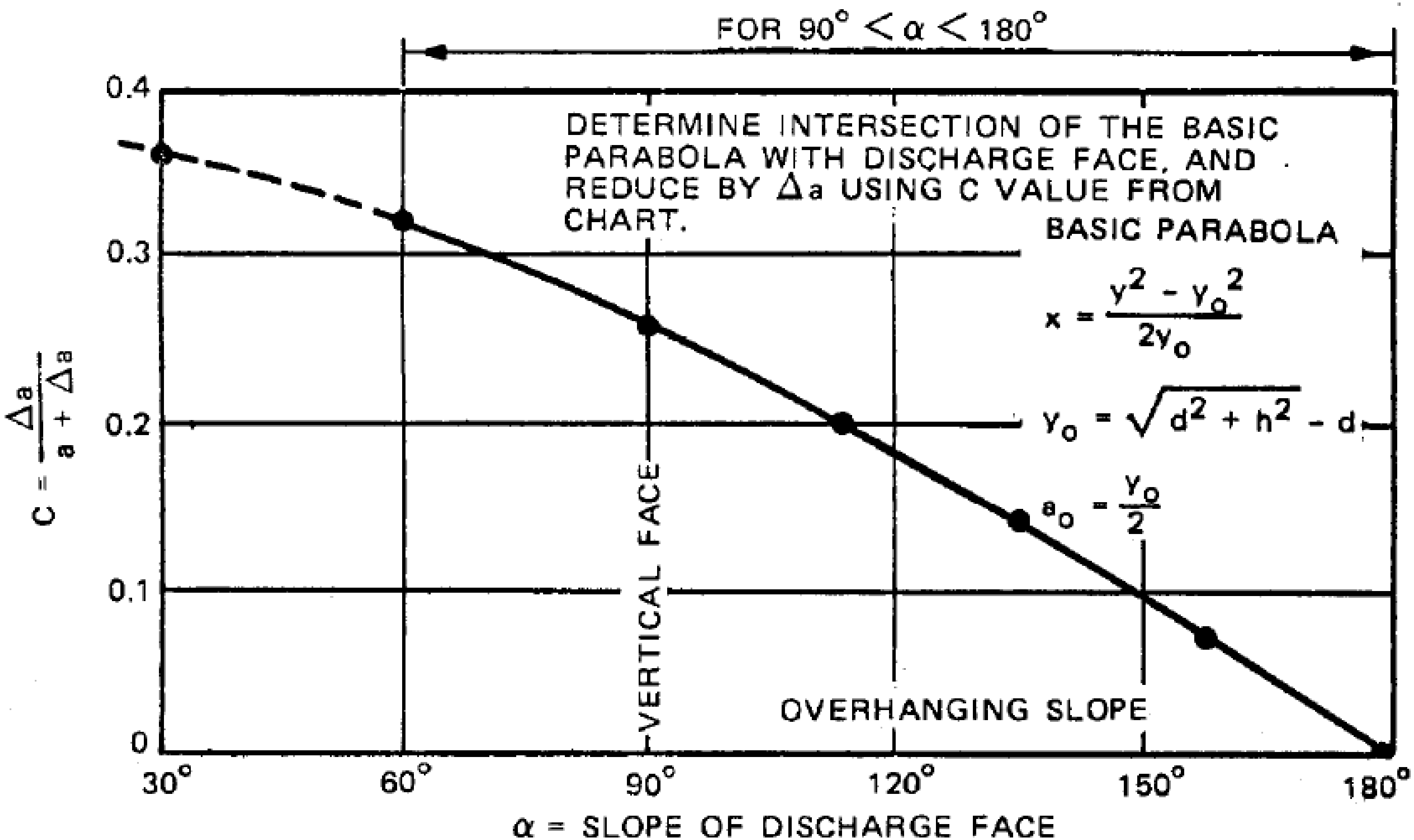
Horizontal filter (Blanket) length

- The horizontal filter extends from the toe (d/s end) of the dam, inward, up to a distance varying from 25% to 100% of the distance of the toe from the center line of the dam.
- Generally a length equal to three times the height of the dam is sufficient.

iii. With inclined discharge face (α between 30° to 180°)

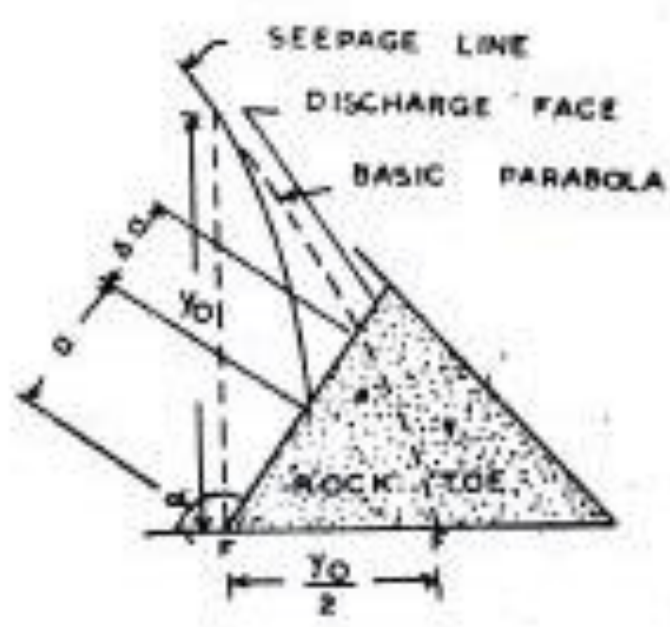


- Construct the basic parabola
- Determine the point, C_o , as shown in figure, where the basic parabola intercepts the downstream slope
- Measure the distance a +Δa or calculate from ⟶ $a + \Delta_a = \frac{y_o}{1 - \cos\alpha}$
- Knowing α , c can be found in next figure and Δa calculated



FOR 90° < α < 180°
DETERMINE INTERSECTION OF THE BASIC PARABOLA WITH DISCHARGE FACE, AND REDUCE BY Δa USING C VALUE FROM CHART.
BASIC PARABOLA
$x = \frac{y^2 - y_o^2}{2y_o}$
$y_o = \sqrt{d^2 + h^2} - d$
$a_o = \frac{y_o}{2}$
VERTICAL FACE
OVERHANGING SLOPE
$C = \frac{\Delta a}{a + \Delta a}$
0.4
0.3
0.2
0.1
0
30°
60°
90°
120°
150°
180°
α = SLOPE OF DISCHARGE FACE

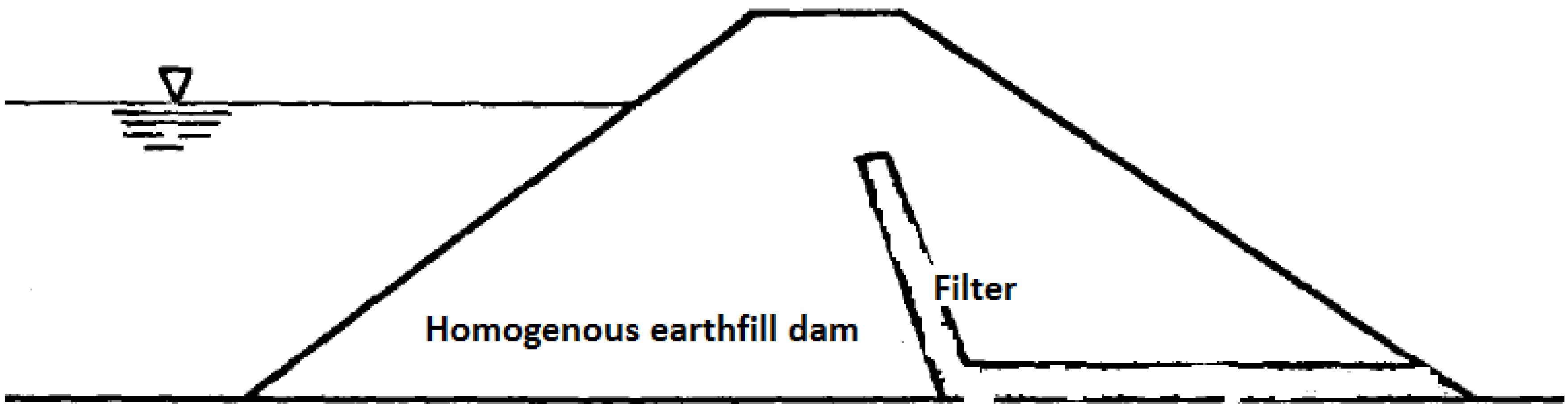
iv. With rock toe (Rock toe drain)



- For a rock toe, an appropriate value of α, measured clockwise from the horizontal base should be taken and the value of $\left(\frac{\Delta_a}{a+\Delta_a}\right)$ read from the curve given

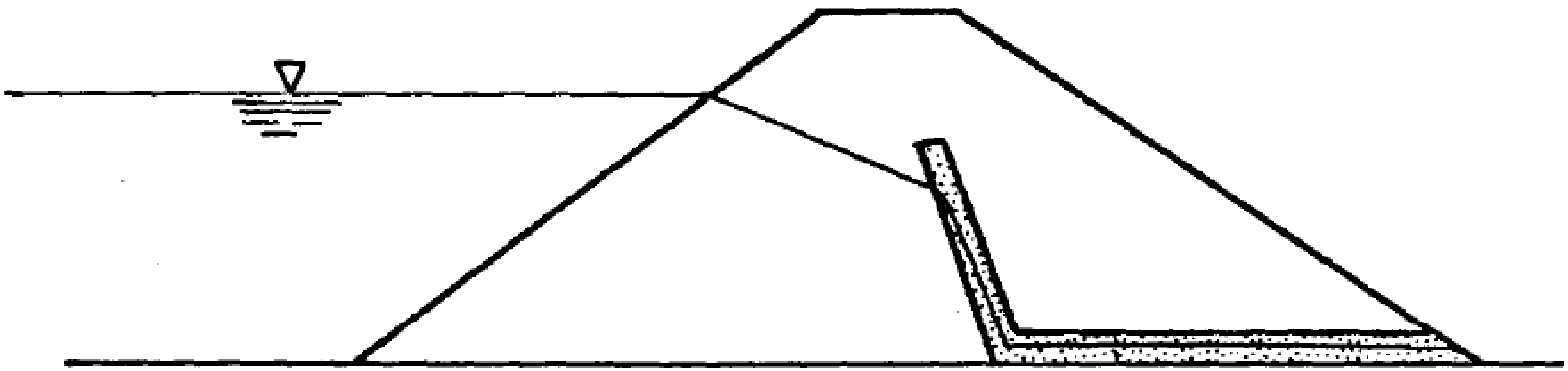
- The parabola is corrected at the egress point.

- Discuss how to draw the parabola to determine the upper line of seepage for an earth Dam with Vertical or near Vertical and Horizontal Drains on Impervious Foundation (shown below)
- Show an indicative position of the parabola

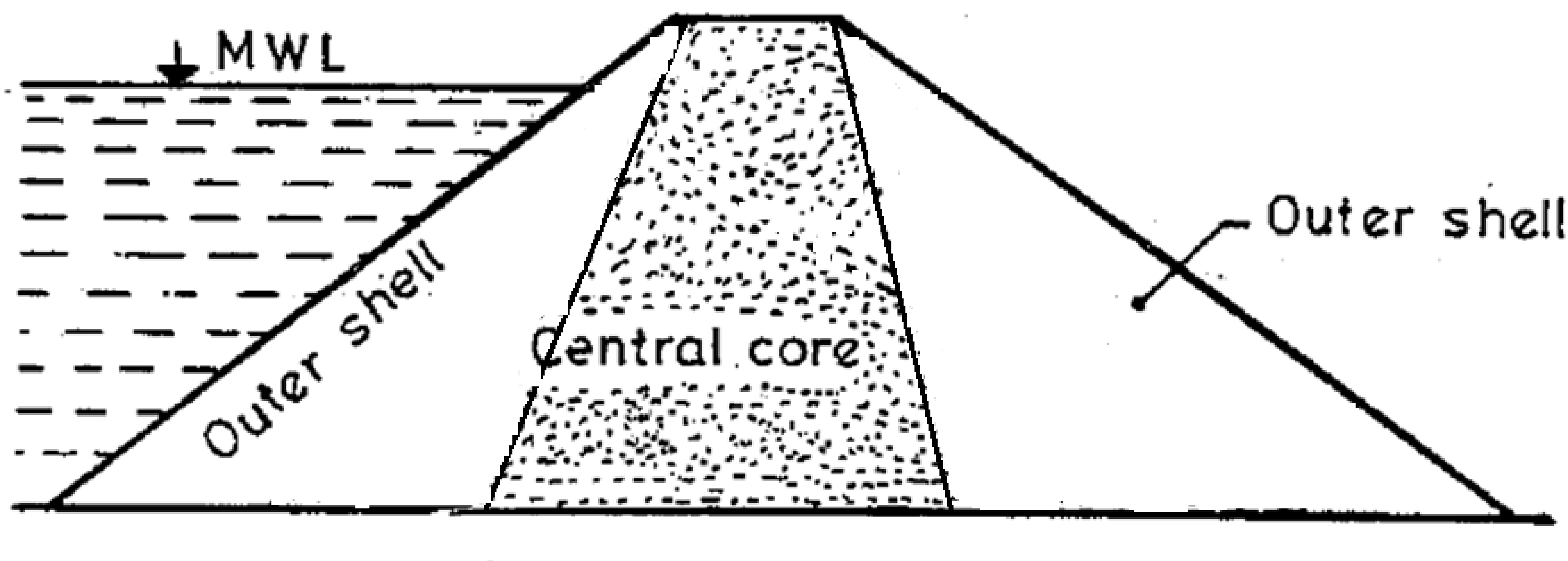


v. Earth Dam with Vertical or near Vertical Horizontal Drains on Impervious Foundation.

- The method recommended by A. Casagrande for drawing the parabola to determine the upper line of seepage can be used.
- The interface of the core and inclined drain is used as the downstream slope for the seepage face since the drain has a much higher permeability than the core material.
- Provision must be made in sizing the drain to pass all the water coming out of the core without building up a tailwater on the downstream slope of the core.

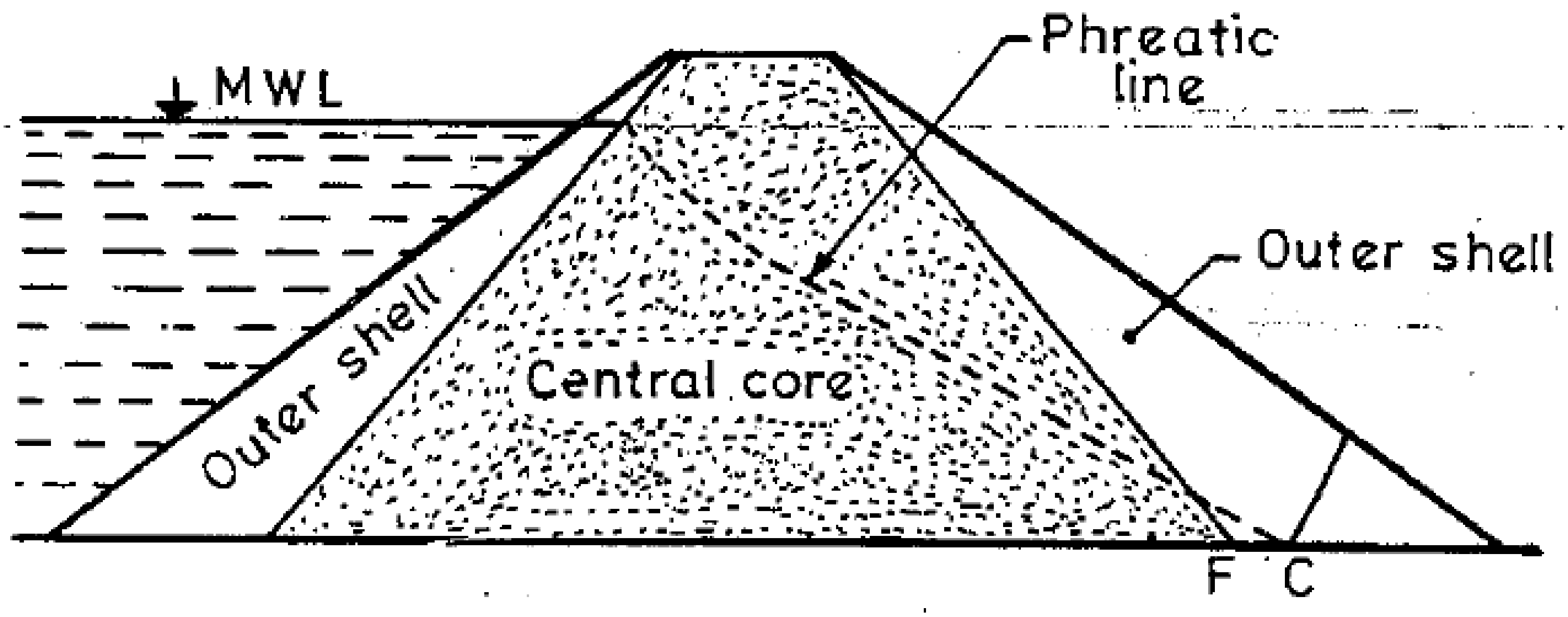


- Discuss how to draw the parabola to determine the upper line of seepage for an earth dam with a zoned section shown.
- Show an indicative position of the parabola



vi. Phreatic line for a zoned section

- The effect of the outer zone can be neglected altogether
- Focus of base parabola located on d/s toe of core
- The phreatic line can then be drawn as usual (previous discussions)



Slope Stability

General Considerations

- Stability analyses are conducted with the following aims:

I. To determine the factor of safety for various slip surfaces of upstream and downstream slopes **under steady state seepage condition** with or without earthquake.

-Usually done for the long-term stability assessment.

-Based on the reservoir at Full Supply Level.

-In reality it may take many years to reach steady state condition in the dam core.

ii. To determine the factor of safety for various slip surfaces of upstream slope **under sudden drawdown condition.**

-performed for conditions occurring when the water level adjacent to the slope is lowered rapidly.

-Stabilizing influence of the water pressure on upstream slope is lost.

-Water level dropped rapidly that the pore pressure in the slope do not have time to reach equilibrium. For analysis purpose, no drainage occurs in materials with low permeability.

- Rapid drawdown: Drawdown rates of 6 inches or more per day after prolonged storage at high reservoir levels (USBR, Design of Small Dams).

iii. To determine the factor of safety for various slip surfaces of upstream and downstream slopes **during and end of construction condition**

-Performed using drained strengths in free-draining materials and undrained strengths in materials that drain slowly.

Stability Analysis and Design Procedure

Slope stability involves the following chain of events

- Explore and sample foundation and borrow sources
- Characterize the **soil** strength
- Establish the 2-D idealization of the **cross section**, including the surface geometry and the subsurface boundaries between the various materials of the dam
- Establish the **seepage (pore water pressures)** and groundwater conditions in the cross section as measured or as predicted for the design load conditions
- Select **loading conditions** for analysis
- *Select **trial slip surfaces** and compute factors of safety **using appropriate method**
- Repeat step * above until the "critical" slip surface has been located
- Compare the computed factor of safety with experienced-based criteria

Loading Conditions for Analysis

Loading conditions vary from the commencement of construction of the embankment until the time when the embankment has been completed and has a full reservoir pool behind it.

Prominent loading conditions are

- End of Construction
- Sudden drawdown
- Steady seepage, normal pool
- Earthquake

Loading Conditions & FoS_{min} (USACE)

Case	Loading Condition	Critical Slope	FOSmin
I	End of construction	Upstream	1.3
		Downstream	1.3
II	Sudden drawdown	Upstream	1.3
III	Steady state seepage	Upstream	1.5
		Downstream	1.5
IV	Steady state seepage with earthquake	Upstream	1.1
		Downstream	1.1

Seismic Design

Pseudostatic Analysis

- Use of the pseudostatic method of stability analysis is recommended for reasonably well-built dams on stable soil or rock foundations, if estimated peak ground accelerations are less than 0.2g.
- A pseudostatic analysis represents the effects of earthquake shaking by accelerations that create inertial forces.

The forces are defined as:

$$F_h = \frac{a_h W}{g} = k_h W$$

$$F_v = \frac{a_v W}{g} = k_v W$$

- The horizontal & vertical coefficient of acceleration (α_h & α_v) can be determined from the acceleration contours based on a design base earthquake (DBE).

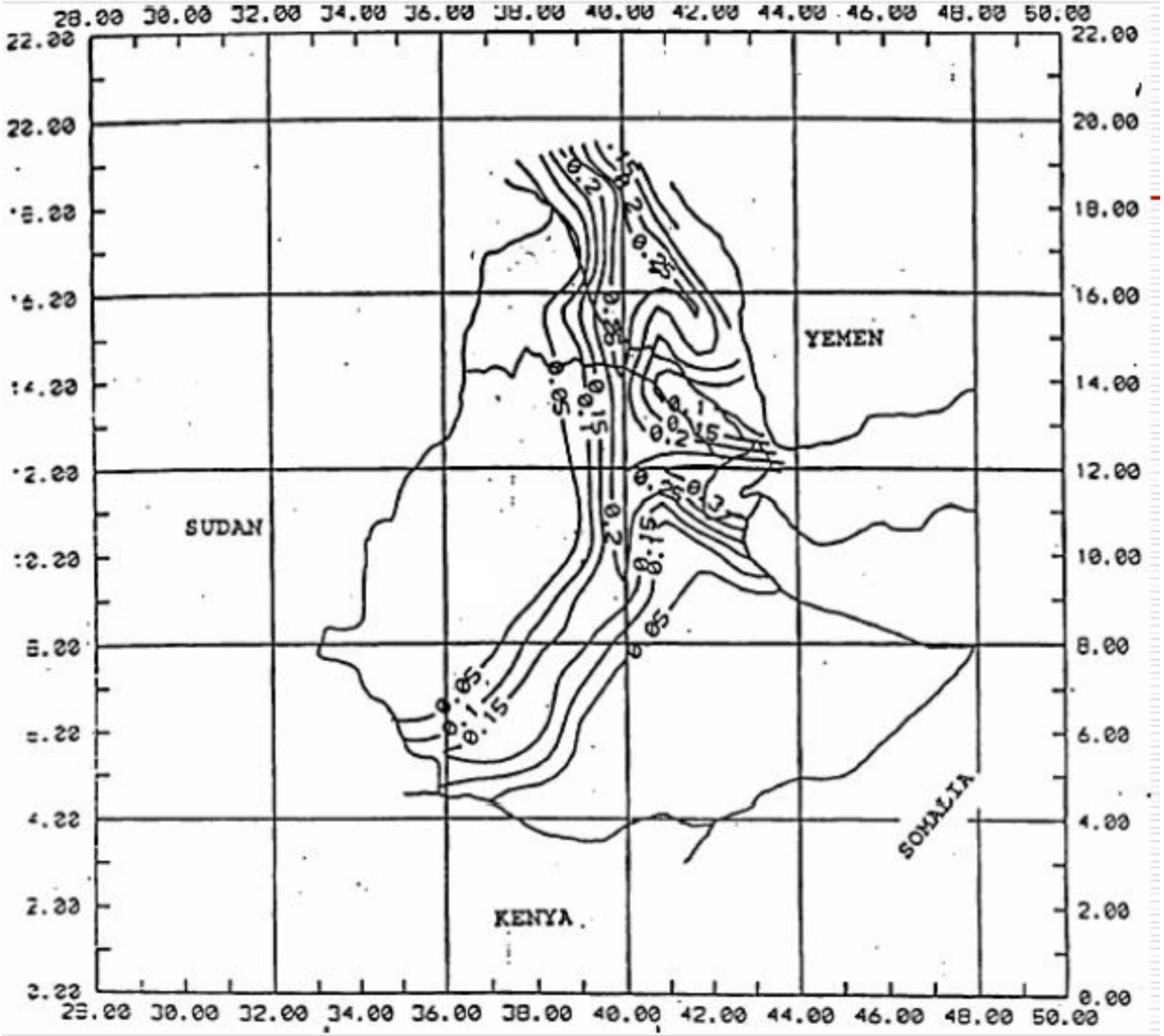


Figure: Seismic hazard map of Ethiopia and its Northern & Eastern neighboring countries (for a probability of exceedance of 0.0033 or return period of 300 years)

Method of Slope Stability

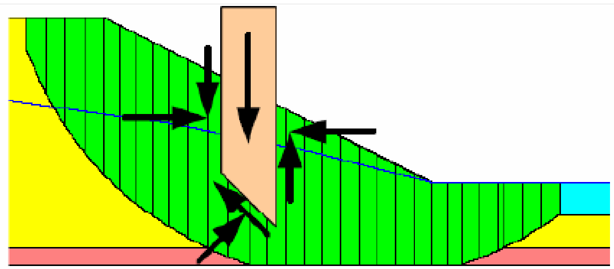
- Stability Analysis of dams is usually carried out using Limit Equilibrium Methods.
- All limit equilibrium methods employ the same definition of the factor of safety

$$F = \frac{\text{Shear strength of the soil}}{\text{Shear strength required for equilibrium}}$$

Method of slices

$$\boxed{S = \frac{c'\Delta l}{F} + \frac{(N - u\Delta l)\tan\phi'}{F}}$$

Method of slices



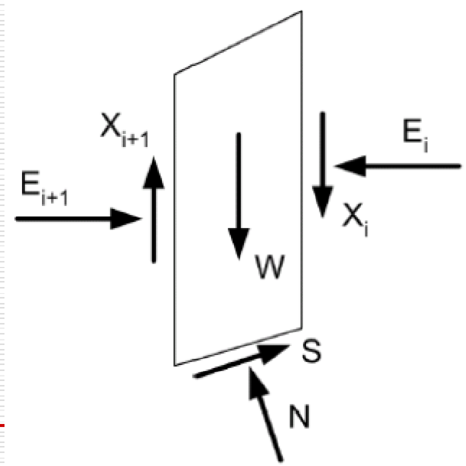
W - slice weight

E - horizontal (normal) forces on the sides of the slice

X - vertical (shear) forces between slices

N - normal force on the bottom of the slice

S - shear force on the bottom of the slice



Forces & other Unknowns

- Forces and other unknowns that must be calculated from the equilibrium equations are summarized below (USACE).

Table C-1
Unknowns and Equations for Limit Equilibrium Methods

Unknowns	Number of Unknowns for n Slices
Factor of safety (F)	1
Normal forces on bottom of slices (N)	**n**
Interslice normal forces, E	n – 1
Interslice shear forces, X	n – 1
Location of normal forces on base of slice	**n**
Location of interslice normal forces	n – 1
TOTAL NUMBER OF UNKNOWNS	5n – 2
Equations	**Number of Equations for n Slices**
Equilibrium of forces in the horizontal direction, $\Sigma F_x = 0$	n
Equilibrium of forces in the vertical direction, $\Sigma F_y = 0$	n
Equilibrium of moments	n
TOTAL NUMBER OF EQUILIBRIUM EQUATIONS	3n

$$\boxed{\text{for } n > 1,\ 5n - 2 > 3n}$$

-Therefore, assumptions are made to make the problem statically determinate.

-The various limit equilibrium methods use different assumptions to make the number of equations equal to the number of unknowns. They also differ with regard to which equilibrium equations are satisfied.

Various methods of stability analysis

- The Ordinary, or Fellenius method
- Simplified Bishop (1955)
- Janbu (1957)
- Spencer (1967)
- Morgenstern and Price (1965)
- Corps of Engineers

The differences between the methods are depending on: what equations of statics are included and satisfied and which interslice forces are included and what is the assumed relationship between the interslice shear and normal forces?

Methods of slope stability analyses

Method	Factor of Safety (FS)		Interslice Force Assumption (H=horizontal, V=vertical)
	Force Equilibrium	Moment Equilibrium	
(1) Ordinary (Swedish or USBR)	-	Yes	Ignore both H, V
(2) Bishop's Simplified	**-**	**Yes**	**V ignored, H considered**
(3) Janbu's Simplified	Yes	-	V ignored, H considered
(4) Janbu's 'Generalised'	Yes	-	Both H, V considered
(5) Spencer	**Yes**	**Yes**	Both H, V considered
(6) Morgestern-Price	**Yes**	**Yes**	Both H, V considered
(7) Lowe-Karafiath	Yes	-	Both H, V considered
(8) Corps of Engineers	Yes	-	Both H, V considered

Ordinary or Fellenius Method (also Swedish method of slices ***)***

- First method of slices developed
- All interslice forces are ignored
- The normal force on the base of the slice is calculated by summing forces in a **direction perpendicular to the bottom of the slice**.
- Once the normal force is calculated, moments are summed about the center of the circle to compute the factor of safety
- The factor of safety is computed from the equation

$$F = \frac{\sum \left[c'\Delta\ell + \left(W\cos\alpha - u\Delta\ell \right) \tan\phi' \right]}{\sum W \sin\alpha}$$

where

c' and ϕ' = shear strength parameters for the center of the base of the slice

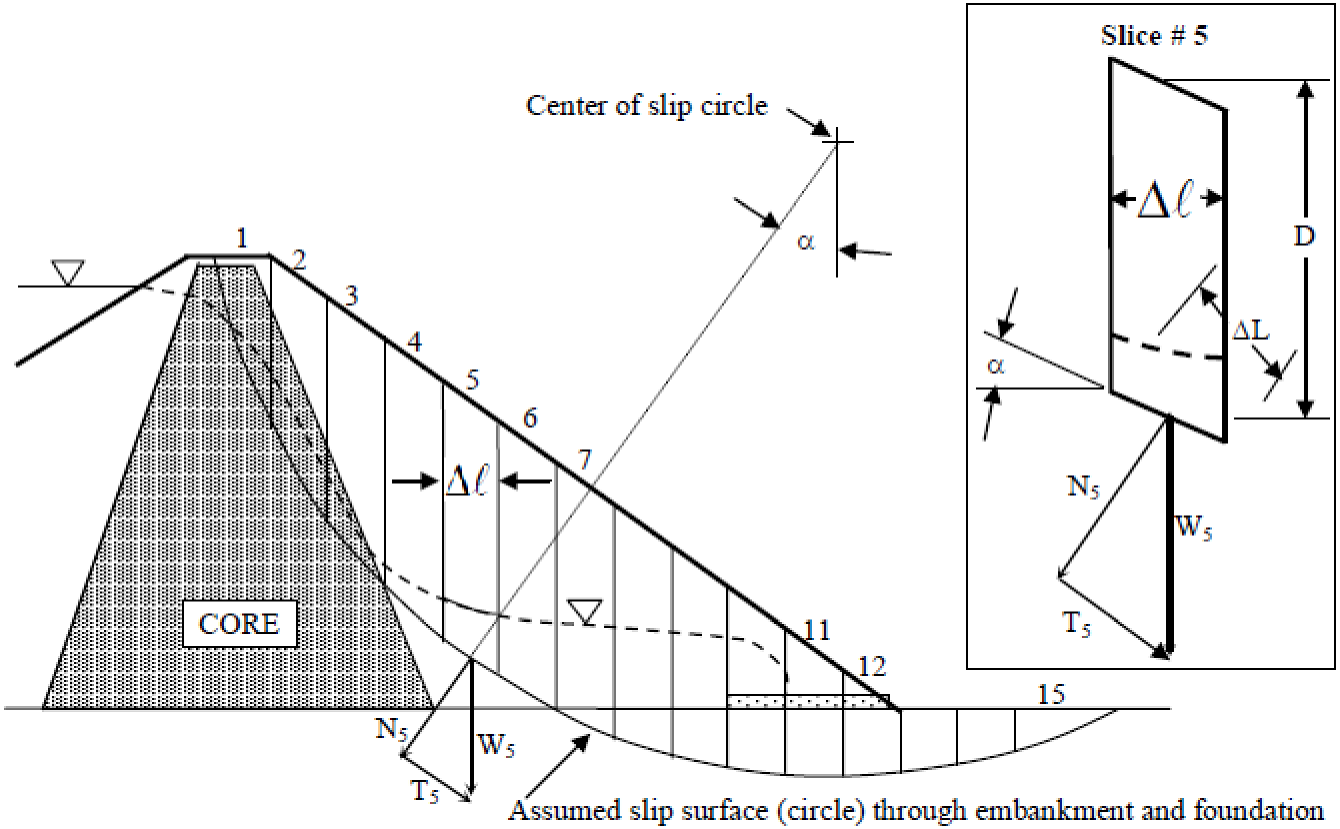
W = weight of the slice, α = inclination of the bottom of the slice

u = pore water pressure at the center of the base of the slice

Δl = length of the bottom of the slice

Limitations

- Neglecting the forces on the sides of the slice
- Does not satisfy equilibrium of forces in either the vertical or horizontal directions
- Moment equilibrium is satisfied for the entire soil mass above the slip surface, but not for individual slices



Dam stability analysis by method of slices

Normal component of W force acting on bottom of slice: $N = W \cos \alpha$

Tangential component of weight: $T = W \sin \alpha$

The Simplified Bishop Method

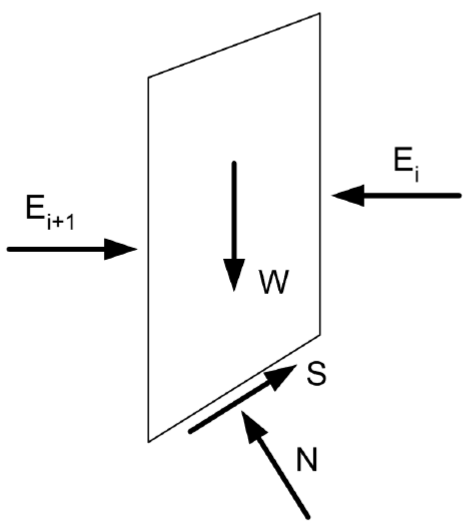
- is based on the assumption that the interslice forces are horizontal, as shown in Figure below (i.e also ignores the interslice shear forces).
- The normal force on the base of the slice is perpendicular to the bottom of the slice (similar to the ordinary method)
- A circular slip surface is also assumed in the Simplified Bishop Method.
- A simple form of the Bishop's Simplified factor of safety equation in the absence of any pore-water pressure is

$$FS = \frac{1}{\sum W \sin\alpha} \sum \left[\frac{c\beta + W \tan\phi - \frac{c\beta}{FS} \sin\alpha \tan\phi}{m_\alpha} \right]$$

where β is slice base length, and mα is defined by the following equation

α = inclination of the bottom of the slice

$$m_\alpha = \cos\alpha + \frac{\sin\alpha \tan\phi}{FS}$$



In summary, the Bishop's Simplified method,
- considers horizontal interslice forces, but ignores interslice shear forces, and
- satisfies over all moment equilibrium, but not overall horizontal force equilibrium

Morgenstern-Price method

The Morgenstern-Price method:

- Considers both shear and normal interslice forces,
- Satisfies both moment and force equilibrium, and
- Allows for a variety of user-selected interslice force function

Assumptions: Recall that

The Ordinary, or Fellenius method: Interslice forces completely ignored (both magnitude and direction)
The Simplified Bishop Method: Considered interslice normal forces and assumed to act horizontally

The method put out by Morgenstern and Price is considered to be the most general limiting equilibrium method to date. What they proposed is as follows,

1. Consider a different angle for each resultant force on each interface: Let x be a horizontal axis starts from the toe toward the slope, this assumption can be written as,

$$\frac{X_i}{E_i} = f(x_i) = \tan \theta_i$$

where, Xi, Ei are the vertical and horizontal components of the side force acting on interface i that has a coordinate of xi. f(x) is an arbitrary function which should be specified in advance. f(x) can take any shape.

2. Assume all these angles are related through an unknown scale factor, λ– one additional unknown. In equation form, this becomes,

$$\frac{X_i}{E_i} = \lambda f(x_i)$$

Spencer has simplified the Morgenstern and Price's method by assuming that f(x) is a constant. In other words, Spencer poses the equation as,

$$\frac{X_i}{E_i} = \lambda = \tan\theta$$

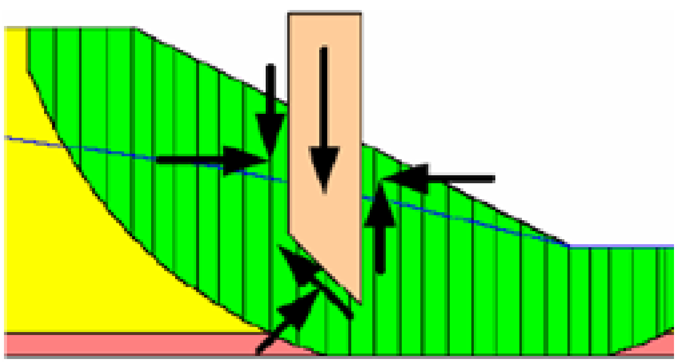
That is, the angles of all resultant interslice forces are the same that have to be solved.

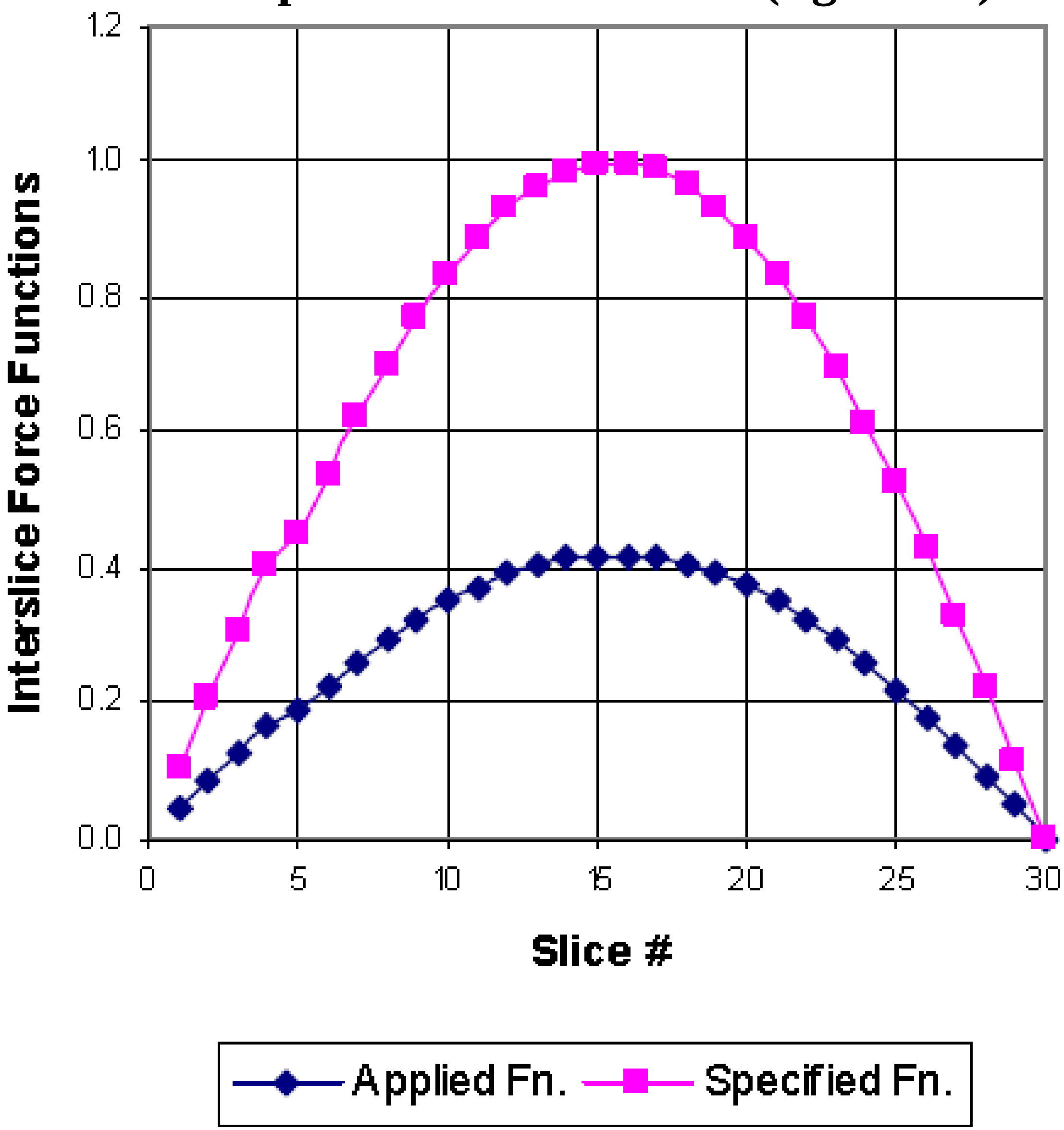
$$\frac{X_i}{E_i} = f(x_i) = \tan\theta_i$$ =interslice force function

Example: half sine function (fig below)

- f(x) can take any shape
- Assume all these angles are related through an unknown scale factor, λ

$$\frac{X_i}{E_i} = \lambda f(x_i)$$



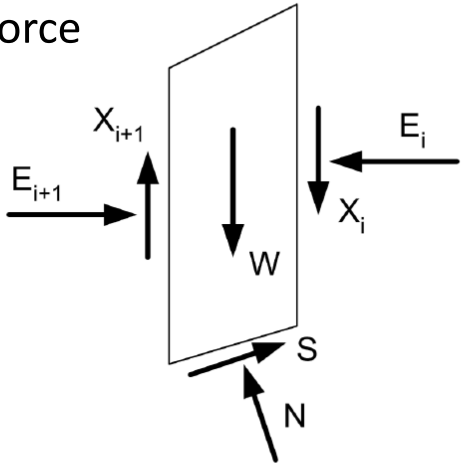


Interslice force function : How the interslice shear forces are handled and computed

General interslice shear forces are handled with an equation proposed by Morgenstern and Price (1965). The equation is:
$X = E\lambda f(x)$

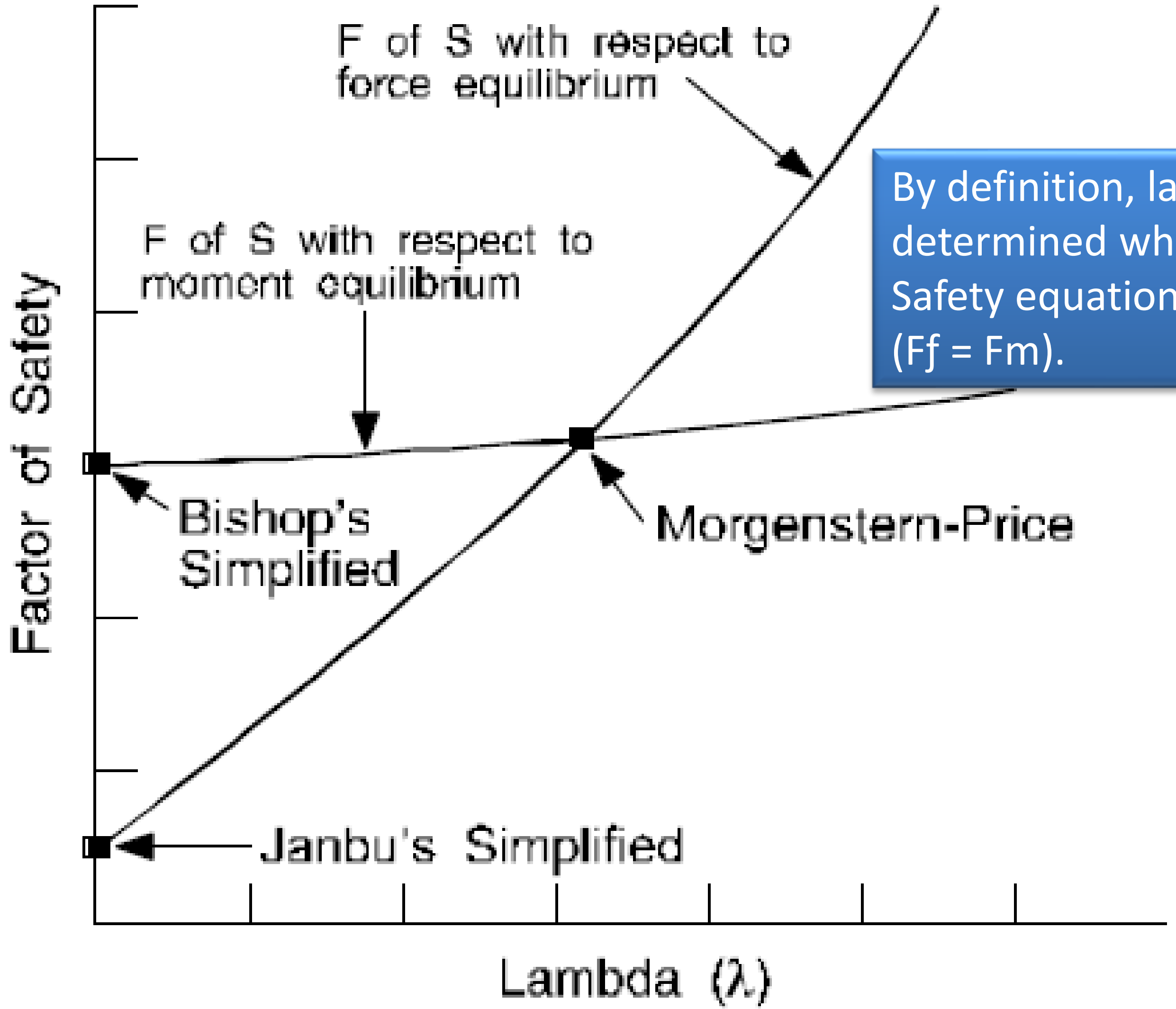
where, $f(x)$ is a function, λ is the percentage (in decimal form) of the function used, E is the interslice normal force and X is the interslice shear force

W - slice weight
E - horizontal (normal) forces on the sides of the slice
X - vertical (shear) forces between slices
N - normal force on the bottom of the slice
S - shear force on the bottom of the slice



Within the framework of Spencer's solution, the simplified Bishop method can be viewed as a partial solution in that (1) the angle of the inter-slice force is assumed beforehand a fixed number, tanΘ=0 that is; and (2) only moment equilibrium is satisfied. Spencer has shown that the factor of safety from moment equilibrium is not very sensitive to the angle of the inter-slice force when a constant value is used. Thus Bishop was able to obtain a reasonable solution.

Moment and Force Factors of Safety as a Function of the Interslice Shear Force



By definition, lambda value is determined where the two Factor of Safety equations become equal ($Ff = Fm$).

Case histories of typical dam failures

i. Failure due to Overtopping

The South Fork Dam

The dam was located in western Pennsylvania, about 70 miles east of Pittsburgh(USA)

- The 72-foot high dam was an earthfill embankment, with the original construction completed in 1852
- The reconstructed dam failed on May 31, 1889, due to overtopping failure during a large flood.
- Over 2200 people were killed.

Several factors contributed to the dam failure, including:

- settlement of the dam resulted in lowering the dam crest at the maximum section by about 6 inches (Frank, 1988)
- the lowering of the dam crest reduced surcharge capacity in the reservoir and correspondingly reduced the spillway capacity;
- the bridge piers and the screens across the piers, in combination with debris that was caught on the screens reduced the spillway capacity; and

ii. Failure due to inadequate investigation and design on the dam site and reservoir area

Baldwin Hills Dam in Los Angeles, California

- Designed as a homogeneous earth fill, the dam was 71 meters high and 198 meters long

On December 14, 1963, at about 11: 15 A.M., an unprecedented flow of water was heard in the spillway pipe. The water came from drains under the reservoir lining

Geologic Setting

- Composed of sedimentary formations, principally of marine origin, overlying crystalline schist at depths of 3050 to 3660 meters
- Several minor, steeply dipping faults were mapped in the Baldwin Hills during construction
- The reservoir foundation consisted of sediments that were susceptible to densification and erosion. During construction the formations were seen to be intensely jointed. Most of the joints were tight, but a few had gaps of as much as 6 mm.

Design and Construction

- Construction began on January 13, 1947, and was completed on April 18, 1951
- The embankment was constructed of materials excavated from the reservoir bowl
- The design incorporated under drain systems and a reservoir lining

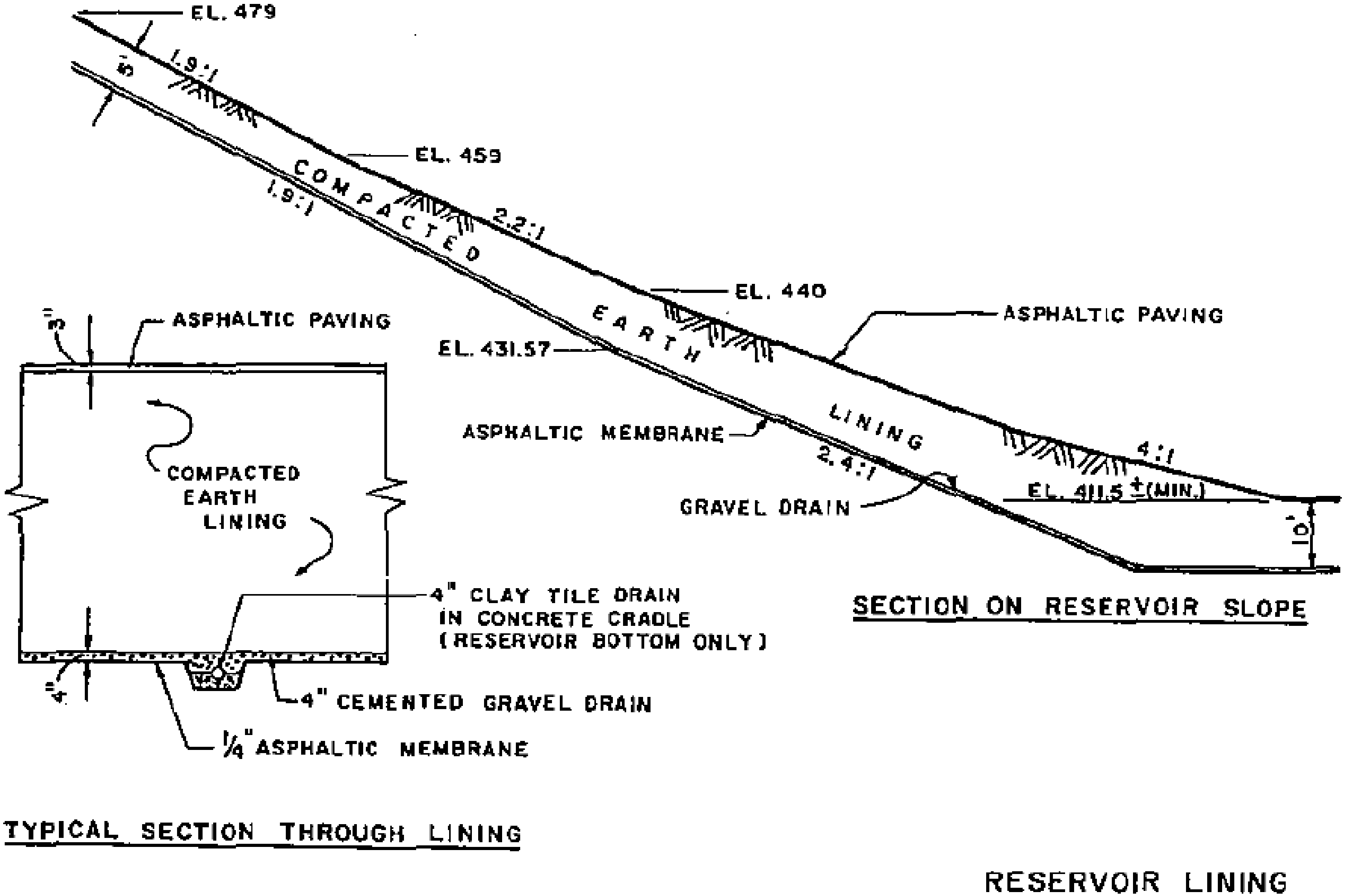


Fig. Baldwin Hills Reservoir lining

Surveillance

- Flows of the reservoir underdrains were measured at weirs in the inspection chamber before discharge into a 24- in. (61-cm) outfall line, which connected to the spillway pipe.
- Flows in the embankment foundation drains were measured monthly. Throughout the project operation no flows were observed in the drains for the embankments on the east, west, and south sides.
- Periodic inspections were made of observation wells at the reservoir perimeter. Reportedly there was never any water in these wells

History of Operation

In the early years of operation following the initial remedial work in 1951, the underdrains required much maintenance

- Calcium carbonate deposits developed in the drains, requiring frequent cleaning. Clogging, and possibly displacement of the drain tile, caused a reduction in the total seepage entering the inspection chamber
- A crack was discovered in the drainage inspection chamber near Fault I
- the concrete encasement (for drains entering the inspection chamber) was found to be cracked approximately 3.7 meters from the discharge end of the west toe drain
- Inspectors found some cracking in the thin cement coating on the asphaltic pavement when the reservoir was emptied, and the lining cleaned and checked
- In the weeks immediately preceding failure an apparent uplift developed in the inlet tunnel, the gate tower, and the part of the inspection chamber east of Fault I.
- In the final year of operation, the flows from discharging horizontal drains under the main dam varied rapidly, followed by continued fluctuations

Post failure Conditions

After the failure,

- ❖ about 2 in. (5 cm) of fine silt and clay covered the reservoir floor.
- ❖ There was a continuous crack
- ❖ Vertical displacement averaged about 2 in. (5 cm), but was as much as 7 in. (18 cm)
- ❖ cavities were discovered beneath the gravel drain in some of the excavations

Analysis of Failure

Several investigators have offered premises about the cause and mechanism of the reservoir failure.

- ❖ Hamilton and Meehan concluded in 1971 that fluid injection caused shear displacements along Fault I, and that rupture propagating to the surface sheared the earth lining
- ❖ Casagrande et al believed differential settlement occurred and could be explained by the greater compressibility of fractured and loosened material

In summary,

- ❖ the reservoir and its immediate environs were subjected to many adverse forces, including horizontal and vertical displacement due to subsidence;
- ❖ Local breaking of the weak foundation; some erosion at the faults;
- ❖ rebound effects due to oil field repressurization,
- ❖ reservoir loading and unloading in 1951 and 1957, and the
- ❖ final inrush of water into the Fault zones at time of failure

Lessons Learned

i. Foundations in erodible rocks must be thoroughly explored to disclose any preexisting cavities/defects.
ii. The total prevention of leakage into a reservoir foundation over the lifetime of the facility may be unattainable under usual circumstances.
iii. Associated faults that lie in close proximity and subparallel to an active fault should be regarded as susceptible to movement in a seismic event.
iv. The possibility of differential fault movement unrelated to tectonic activity must be considered.
v. Potential effects of ground subsidence must be recognized in designing dams and reservoirs.
vi. External causes and effects of subsidence must be closely monitored.
vii. Foundation discontinuities should be given special treatment in construction.
viii. Rigid buried elements such as cemented drains should not be incorporated into designs where differential settlement is a possibility.
ix. Drains should be amply sized and provided with access, where possible, to facilitate maintenance.
x. Application of sprayed asphalt as a reservoir seal must be questioned as to its long-term effectiveness.
xi. Earth linings preferably should have appreciable plasticity.
xii. Erodible embankment and foundation elements must have adequate filter protection.
xiii. Structures placed across faults should be conservatively designed to accommodate predictable movements.
xiv. The use of heavy construction equipment must be carefully controlled to avoid damage of critical reservoir features on soft foundations.
xv. Surveillance of a reservoir must be extended to its environs and to the consequences of adjacent developments and physical changes.