Chapter - 1

Introduction:

Hydraulic structures:

Defn:

. Any structures directly in contact with water(structures constructed on water bodies).

. Example: river training works, weir, dam, diversion tunnels and appurtenant structures e.t.c

Dam structures and reservoir

Dam: - barriers built across the river/stream.

Ex. Tekeze dam, Hoover dam

Purpose:

- to create enough head

- to impound excess water u/s of the dam for water supply, irrigation & power generation.

- flood control

Continued….

Classification of dam:

a) Based on purpose

* single & multipurpose

b) Based on hydraulic design

* overflow/over topped & non over topped dam

c) Based on construction materials

* concrete & embankment dam.

Continued….

Basic elements/components of dam:

. crest
. heel
. Toe
. Npl
. Dead storage zone
. Surcharge storage
. Mpl(minimum pool level) e.t.c.

. u/s face
. D/s face
. Spillway
. Bottom outlet

Continued….

Reservoir; -

- An artificial lake formed/created by the barrier constructed across the stream.

-the impounded water behind the dam.

Classification:

* According to the purpose, it can be:-

Continued….

1) storage/conservation reservoir:

   - attains excess amount of water during peak flow and releases during low flow when demand arises.

2) Flood control reservoir:

   - formed so as to prevent the d/s area from flood inundation/flooding risk.

Continued….

3) Multipurpose reservoir:

- planned and constructed to serve dual purpose.

- Ex. Beles project.

Concrete dam and its characteristics

Concrete dam;

-built of cementing materials (cement, sand and aggregate, reinforcement bars)

-Ex. Gravity dam, Arch dam and Buttress dam.

Continued……

Gravity dam:

- a concrete dam with vertical or inclined face having triangular c/section.

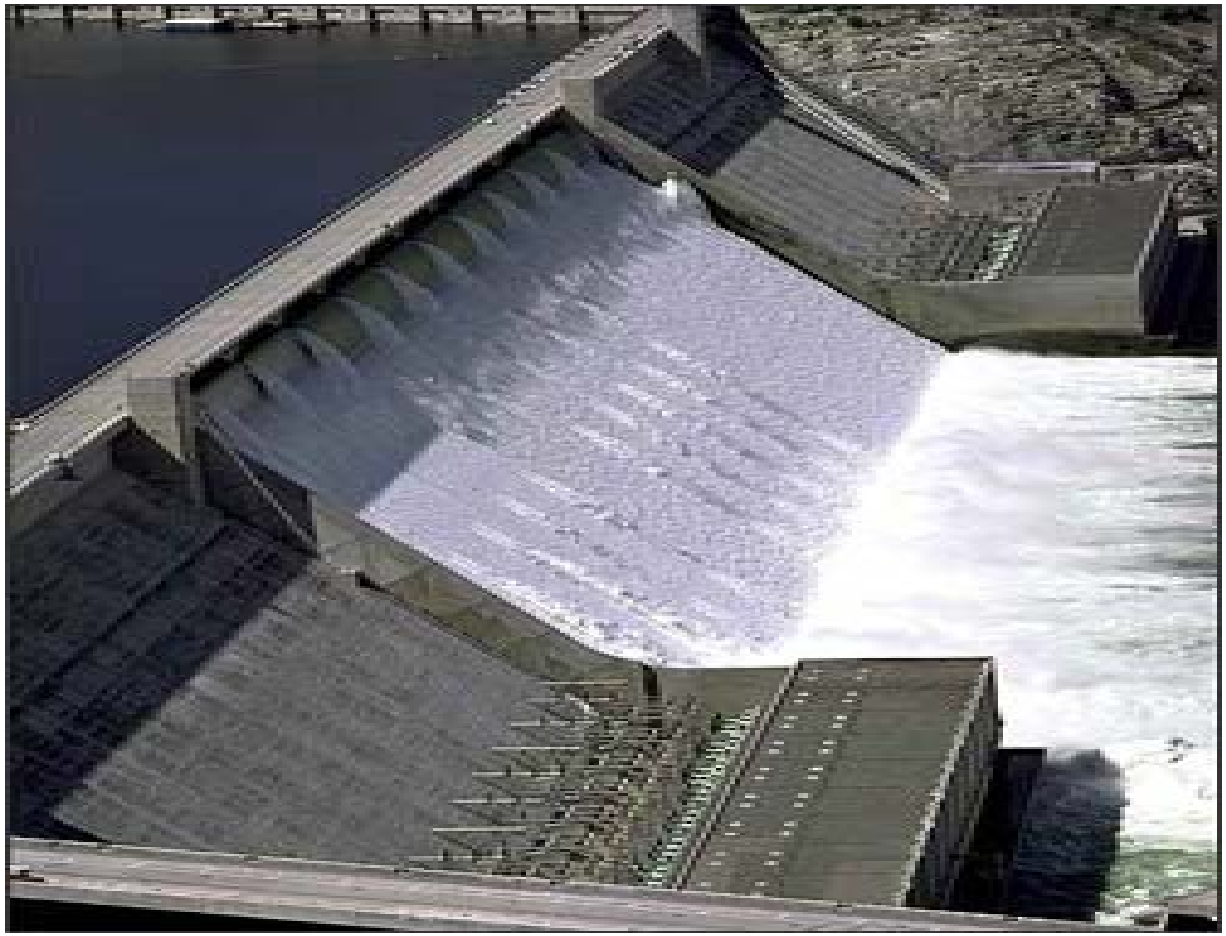
-ensures its stability by its own weight(resist earthquake)

- requires maximum amount of concrete as compared with others.

-requires strong and sound rock beds.

-best suitable for wide valleys(U-shape valley)

Continued……



Continued……

Arch dam:

- A thin concrete dam with u/s curvature.(transmits load to abutment).
- require much less concrete than Gravity dam.(economical).
- suitable for narrow gorges with sound bed and abutment.(V-shaped valley).

Arch dam



Continued….

Buttress dam:

- a dam with watertight u/s face supported by buttresses.
- requires 60% less concrete than Gravity dam
- requires skilled manpower/ workers.

Buttress Dams



Embankment dam and its x-sts

Defn:

*Any dam built of naturally excavated materials like soil, sand, clay e.t.c.

- suited for wide valleys with pervious foundation/week.

- resists earthquake.

- requires extra site for appurtenance structures.

Embankment dam



Site investigation and selection of dam type

*factors affecting selection of dam type:

1- geology (foundation, seismicity and topography)

2- availability of skilled workers

3- cost effectiveness

4- availability of materials

5- hydrology e.t.c

Continued….

1) geology:

- studies about rock and earth
- it comprises of,
    a) foundation requirement
    b) seismicity
    c) topography

Continued……..

a) Foundation requirement:

- it should be impervious and free from fault, cracks, fractures and joints to minimize loss due to seepage and cost of foundation treatment.

b) Seismicity (earthquake risk):

- gravity and embankment dams are best suited in earthquake regions.

Continued……..

c) topography:

-Geometry of river bed and abutment affects the selection of dam.

- site should accommodate the main dam & associated structures to minimize spillway & diversion cost.

- for wide valley (u-shape)---gravity, buttress & embankment dam

- for narrow valley (V-shape)----Arch dam.

Continued……..

2) Availability of skilled manpower:

* At many sites neither skilled contractor nor artesian are available.

--------in such a case Embankment/ masonry gravity dam is best suite.

Continued……..

3) Cost effectiveness:

* standard prices of materials (cement) are varying time to time.

* site condition/topography can influence the cost of dams.

Ex.

- Sites can’t accommodate both the spillway and the main dam requires extra cost for spillway.

Continued…..

4) Availability of materials:

- materials should be available locally so as to minimize cost of transportation.

- Areas in rich of naturally excavated materials are suitable for embankment dam.

Dam assessment and investigation

Parties directly involved in dam eng'g activities.

1) owner/client:

-can be private/governmental organ

purpose:

- provision of financial resource.

2) consulting engineers:

-are a group of engineers like

hydraulic

hydrologist

geologist

civil e.t.c

Continued…..

Purpose:

- provide construction supervision and quality control.
- establishing design and plan.e.tc

3) contractors:

- construction of required infrastructures.
- construction of civil works.
- assembly of equipments.

Phases of Engineering Activities

*Dam Eng'g Activities are carried out in 3 phases:

A) before construction:

- preliminary study
- feasibility study
- basic design

-preliminary study:

* collection and evaluation of data

* field trips / reconnaissance of dam site, reservoir area and D/s area

*preliminary dam design including selection of dam type, main dimension

Continued……

Feasibility study:

* detailed investigation of site condition
* final selection of site
* final selection of dam type, layout, dimension

Basic design:

* completion of detailed site investigation
* detailed design of dam, cost estimation and construction schedules
* development of basic document, tender documents.

Continued……

B) During construction:

* preliminary installation of contractor at site.

* construction of required infrastructures by contractor(road, camp water energy supply e.t.c)

* river diversion works.

* construction of main dam and associated structures.

Continued……

C) After construction:

* monitoring and surveillance

Collection and evaluation of basic data

-basic input data:

*topographical and geomorphology

* geology and hydrogeology

* metrology and climate

* Hydraulics and hydrology

Continued……

1) topography and geomorphology

- are obtained from topographic survey

Aim:

-determine the configuration of dam site

-the configuration of reservoir area

-accessibility to the site

-accessibility to construction material sources and deposits.

Continued……

Reason:

- important for selection of dam type
- important for selection of appurtenant structures

Method:

- Aerial surveying for preparation of topographic maps.
- Ground surveying
  - to prepare scale maps(ground profile section)

Continued…..

Geology and geotechnical investigation

Aim ----- to determine

- eng’g parameters used to evaluate stability of foundation
- seepage pattern and parameters(K)
- used to asses the availability (sources & classn), quality(type & gradation) and quantity of construction materials.
- foundation competence of dam site.

Continued….

Method:

- surface exploration on flood plian deposits(residual soils)

- geophysical exploration (seismic refraction)

- subsurface exploration by using CPT, Auger drilling.

Continued…….

Hydrogeology

- deals with the interaction of water and rock/soil.

Aim

- ground water condition (path & connection)
- mechanical & chemical action of water on geological formation.

Continued…..

Method:

- by water permeability test
- peizometric measurement

meteorology and climatology

- Obtained from meteorological stations.

Aim: - to back up hydrological data (ppt, temperature distribution, humidity of air e.t.c)

Method: - from metrological gauging stations

Continued…..

Hydrology & hydraulics

- obtained from;

*river discharge series

* variation of discharge with time at a given section on a river.

Aim: - determine availability of water

- determine production capacity of the plant for Hp, irrigation & water supply.

- determine design flood.

Method: using - envelop curve, statstical distribn

- Empirical formula and unit hydrograph

Chapter-2

Design & construction of concrete dam

Concrete dam:

- built of concrete (cementing materials)
- ex. Gravity dam, arch dam, buttress dam e.t.c

Forces acting on concrete dam:

* as per degree of relative importance loads can be

1) primary loads:

-major loads acting on the dam continuously.

- ex. Water load,
  seepage/uplift,
  self weight of the dam

2) Secondary loads:

- minor loads w/h are smaller in magnitude.

-ex. Wind load , silt load , wave & ice load.

Continued…..

3) Exceptional load:

- minor loads w/h has limited applicability.

- the probability of occurrence of the load with in the life of the structure can’t exceed 1 or 2 times.

- ex. seismic load/earthquake load.

Continued…..

Magnitude of loads acting on the dam & their centroidal location

A) Primary loads:

1) water pressure:

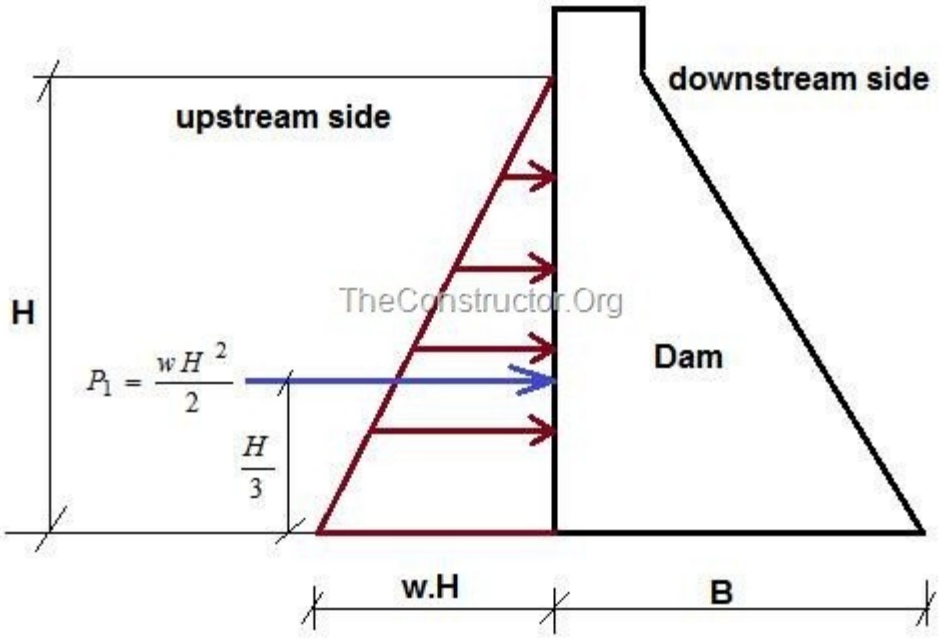
- major load

- the horizontal component is estimated by rule of hydrostatic pressure distribution(HPD).

- HPD is triangular in shape

- the resultant force(pwh1) = area of triangle($\frac{1}{2}*\gamma_w*H^2$) & acts at H/3 from base.

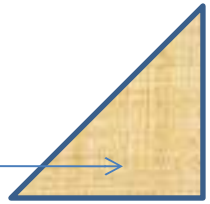
Continued……


upstream side
downstream side
H
TheConstructor.Org
$P_1 = \frac{wH^2}{2}$
Dam
$\frac{H}{3}$
w.H
B

Continued…..

$Pwh1 = \frac{1}{2} * \gamma_w * H^2$

PWh1

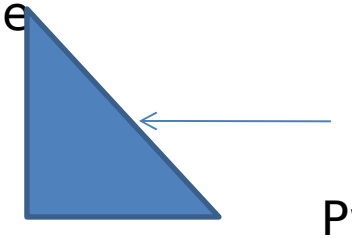


$\gamma_w * H$

Similarly if there is tail water

$Pwh2 = \frac{1}{2} * \gamma_w * h^2$ acts at h/3 from base

h – depth of tail water



h Pwh2

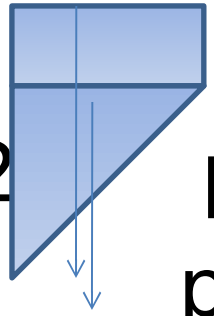
$\gamma_w * h$

- If the u/s and d/s face of the dam is inclined, the water placed on the inclined face of the dam may exert a vertical load

Continued……

For u/s (pwvu) = pvu1 + pvu2

b



pvu2

pvu1 @B-b/2 from toe

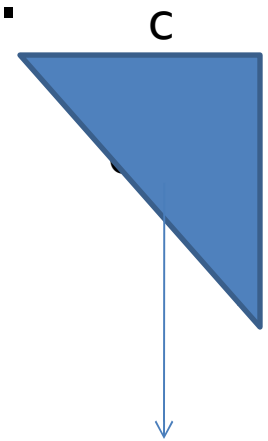
pvu2 @ B-b/3 from toe

B – total bottom width of dam

The magnitude of the resultant vertical water force = area * unit weight of water

Continued….

For d/s (pwvd) = area * γ_w acts at c/3 from toe.



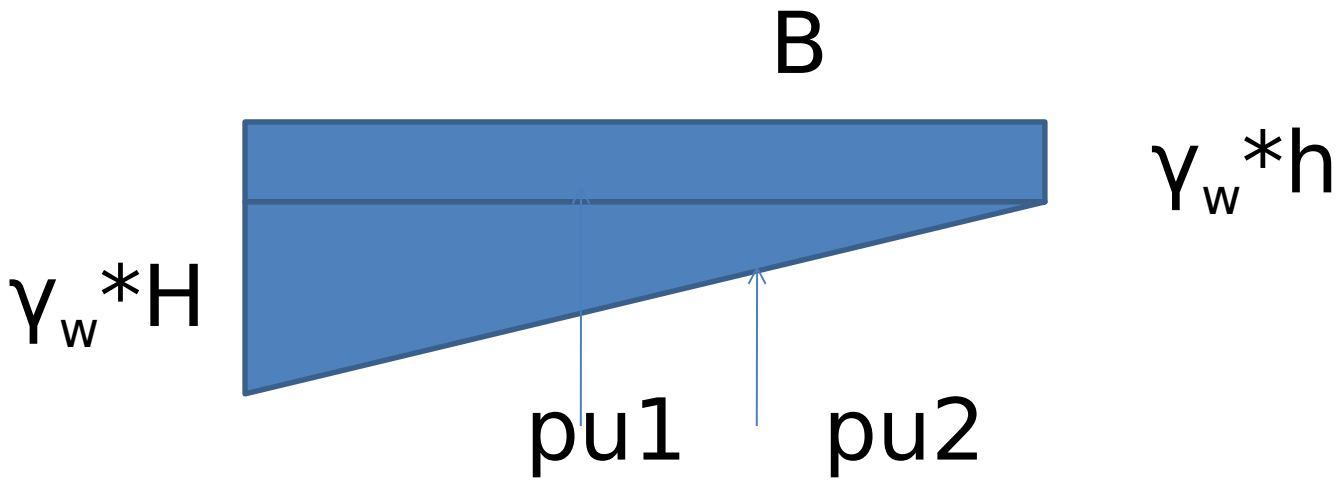
pwvd

Continued……

2) Uplift pressure:

- percolating water below the foundation exerts upward pressure
- major load acts against the dam stability.
- this force reduces the weight of the dam
- according to U.S.B.R recommendation the uplift pressure intensity @ heel & toe = their respective hydrostatic pressure.

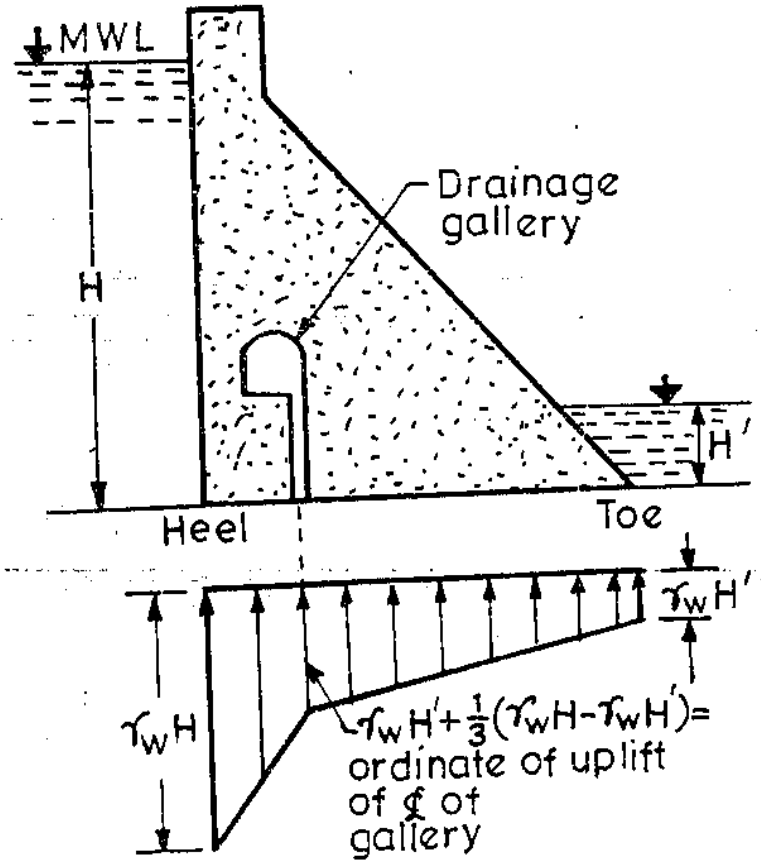
Continued……..



* pu1 = area of rectangle = B*γ_w*h@ B/2
- pu2 = area of triangle = ½ *(γ_w*H-γ_w*h)B

@2/3 B from toe.

if drainage gallery is available inside the body of the dam



Continued……..

- Uplift pressure is controlled by:
  - cutoff wall below foundation of the dam
  - pressure grouting on the foundation
  - providing relief drain/ drainage gallery in side dam

Continued……..

3) Self weight of dam:

- exerted by the structure on the foundation

- it is stabilizing force

- the magnitude of this force

$W = W_C * V$

Where W_C is unit weight of concrete

V – volume of concrete section

- acts @ centroid of dam

Continued……

b) Secondary loads:

1) silt load:

- generated by the accumulated sediment.

- a function of (h_s, γ_{sub}, k_a & Ø)

- $P_{silt} = \frac{1}{2} * K_a * \gamma_{sub} * h_s^2$
- acts @ h/3 from base.

Continued……..

If the u/s face is inclined there is a load acted vertically on the sloping face of dam.

- Magnitude of this load

= vol. of silt * unit weigth

Continued……..

2) Wave load:

- generated on the surface of reservoir by the blowing wind

- is a function of (hw , f & U)

where

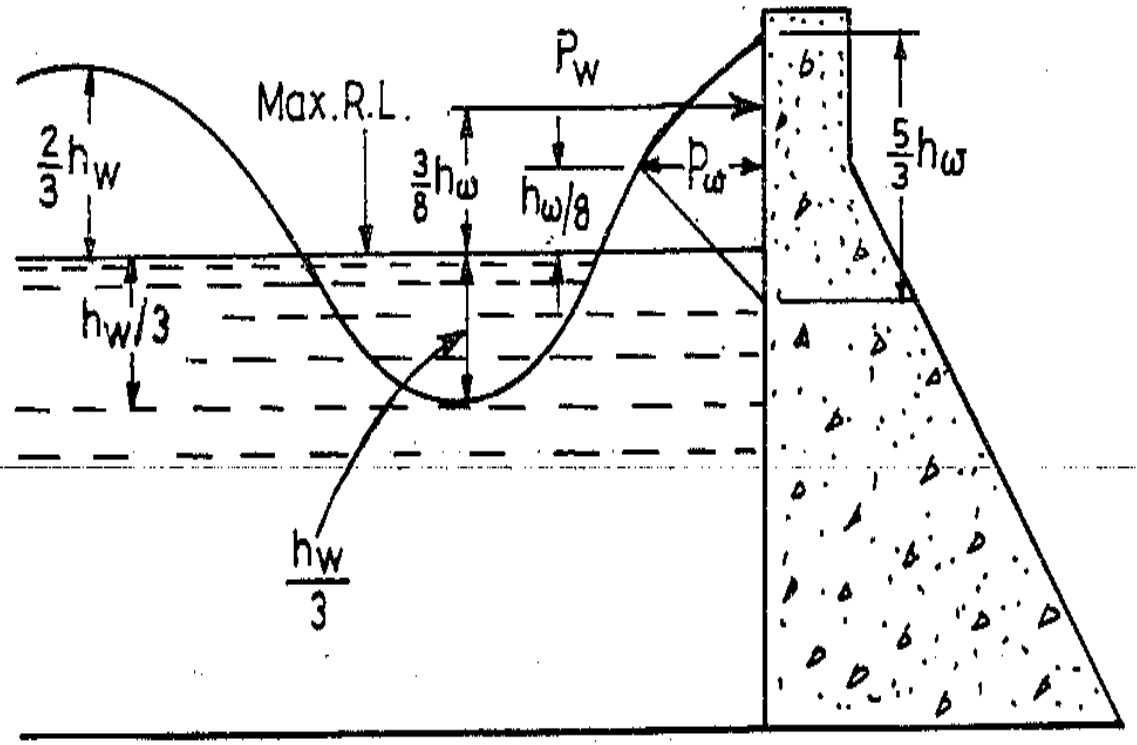
hw – height of wave(m)

f – fetch length of the reservoir(km)

U - velocity of the blowing wind (km/hr)

Continued....

The pressure distribution may be assumed to be triangular, of height $\frac{5h_w}{3}$, as shown in Fig. 19.6.



Continued…..

- Magnitude of load by wave:

$$Pw = ½ * 2.4 * \gamma w * hw * 5/3hw$$

$$= 2 * \gamma w * hw^2$$

- Acts @ 3/8hw from max. reservoir level

where hw is height of wave expressed as:

Continued.....

$$h_w = 0.032\sqrt{V \cdot F} + 0.763 - 0.271\,(F)^{3/4} \textbf{ for F < 32 km, and}$$

$$h_w = 0.032\sqrt{V \cdot F} \textbf{ for F > 32 km}$$

Continued…..

3) Ice pressure:

- a horizontal thrust acted on dam @ the level of reservoir.
- insignificant when depth of ice < 0.6m
- not a problem for hot areas like Ethiopia.
- the magnitude varies (250-1500kn/m2)
- take average 500kn/m2 under ordinary condition

Continued…..

c) Exceptional load (seismic load):

- a stress generated on the dam by earthquake.
- produces a wave w/h shakes the earth in all direction
- two accelerations are induced by earthquake.

Continued…..

i) Vertical acceleration(ah):

ii) Horizontal acceleration (av):

Where ah & av are expressed as %age of acceleration due to gravity.

ah = (0.1-0.2)g & av = (0.5ah)

-- for extreme earthquake region

ah = (0.15-0.3) g may be adopted.

Continued…..

ah = kh*g & av = kv*g

Where kh & kv are constants / fraction of gravity.

Continued……

i) Effect of vertical acceleration(av):

- may acting upward or down

- when it acts upward,

*the foundation will be lifted up ward & becomes closer to the body of the dam.

* Increases the effective weight of the dam.

Continued……

- The inertia force exerted by av:

  = av * m

  = av * W/g, W = weight of dam

- When av acts downward:

  - reduces effective weight of dam

  - worst condition (the foundation moves away from the body of the dam)

- So , net effective weight of dam

  = W- av * W/g

Continued…..

ii) Effect of horizontal acceleration(ah):

- may act in u/s or d/s direction
- may causes the following two forces:

1) hydrodynamic pressure
2) horizontal inertia force

Continued…..

1) Hydrodynamic pressure:

- when ah acts towards the reservoir an extra water pressure called hydrodynamic pressure may develop.

- estimated by von- Karman formula:

$Pe = 0.555Kh*\gamma w*H^2$ @ $4H/3\prod$ from the base.

Continued…..

Zanger's formula:

$$Pe = 0.726 * Cm * Kh * \gamma w * H^2$$

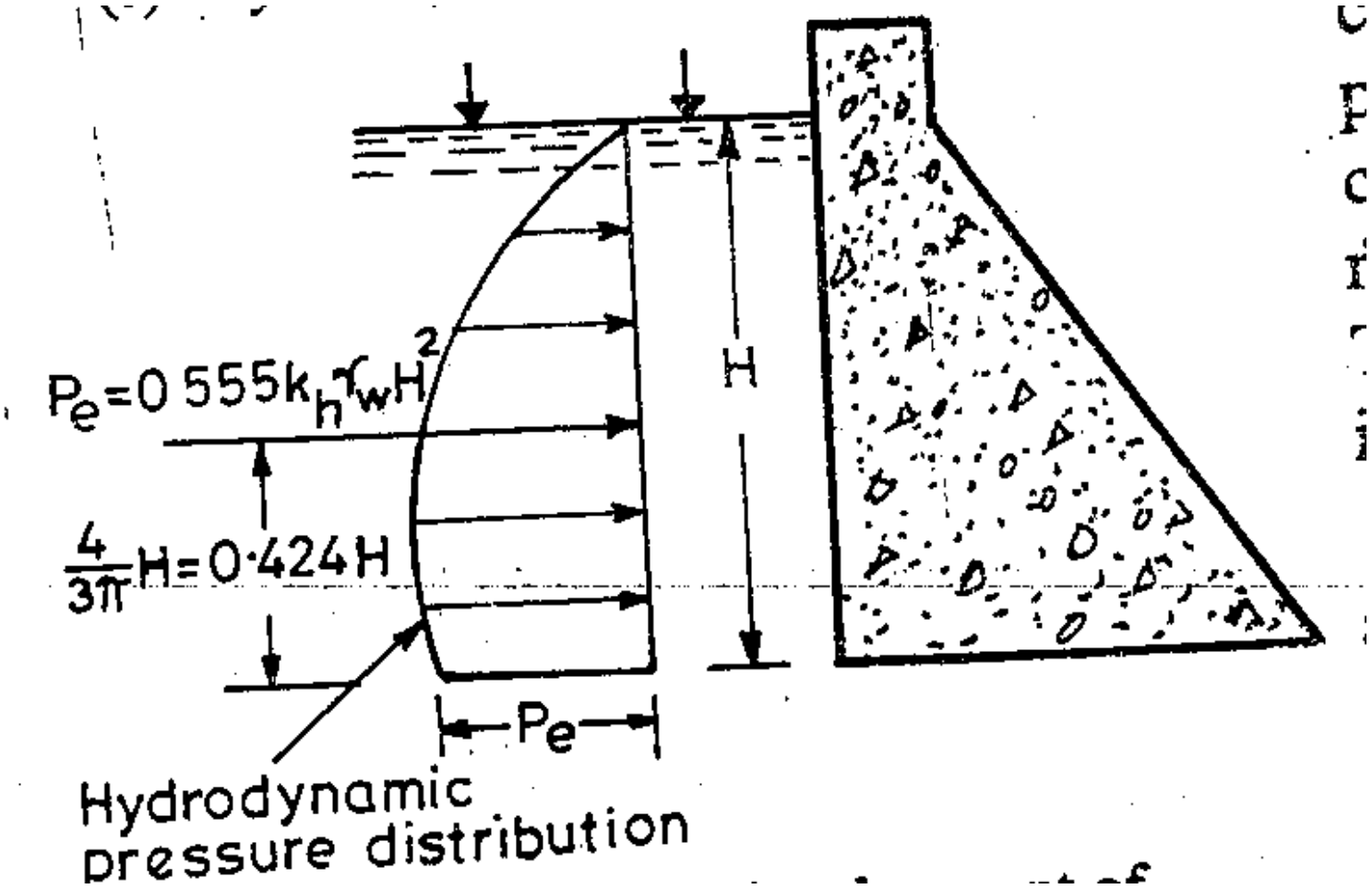
$$Pe = Cm * Kh * \gamma w * H$$

Where Cm = max. value of pressure coefficient for a given slope.

$$Cm = 0.735 * (\theta / 90^0)$$

θ - angle b/n u/s face with horizontal.

Continued.....


$P_e = 0.555 k_h \gamma_w H^2$
$\frac{4}{3\pi} H = 0.424 H$
H
P_e
Hydrodynamic
pressure distribution

Continued…..

2) Horizontal inertia force: -

The inertia force exerted by ah:

= ah * m = Kh*W

= ah * W/g, W = weight of dam

* during Reservoir full condition:

- worst result exists when ah acts towards the reservoir.

* during reservoir empty condition:

- worst , when ah acts towards the d/s direction.

Gravity dam design and analysis

Mode of failure of gravity dam:

i- overturning / rotation

ii- translation / sliding

iii- crushing / compression

iv- development of tension

Continued…..

i) Overturning / rotation:

- F.s.o = $\sum M+ / \sum M-$

Where M+ moment created by resisting forces

M- moment created by disturbing forces

- F.s.o varies b/n (2-3)

*under NLC F.s.o > 1.5 is satisfactory

Continued…..

ii) Sliding stability:

- sliding appears when net horizontal force @base of dam exceeds frictional resistance.

- $\sum H >$ frictional resistance($u\sum V$)

Where u – coefficient of friction b/n two surfaces.(0.65-0.75) = tanØ

- $F_{ss} = u\sum V / \sum H > 1$--- safe!

Continued……

- If the foundation is inclined @ small angle (a).
- F.s. = (∑H/V-tan a)/((1+(∑H/∑V)tan a)u
- To be safe F.s.s < 0.75

Continued…..

- Shear friction factor (Fsf):

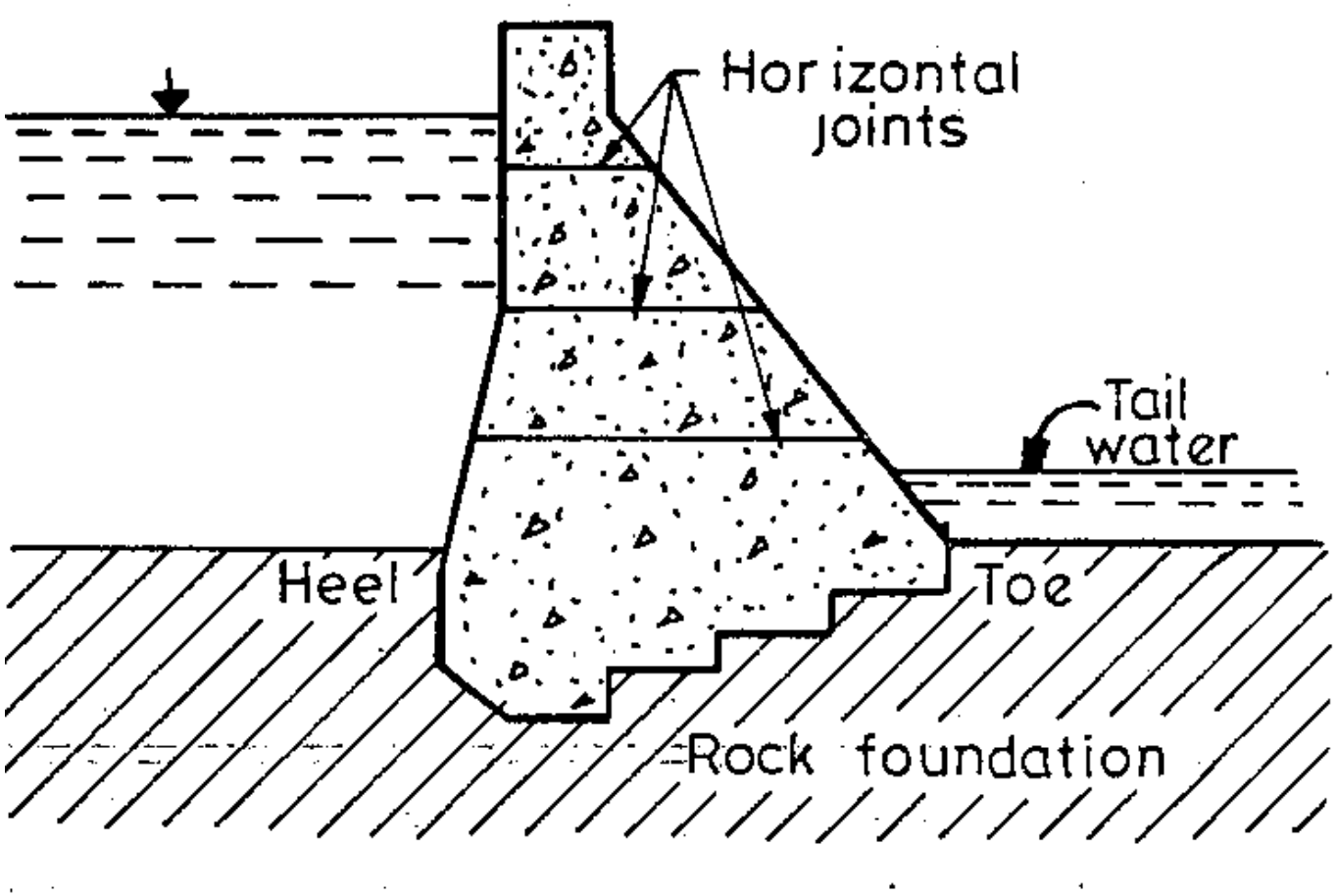
  $= (u\sum V + qB)/\sum H$ lies b/n (3-5)

-where q –shear strength of the joint

q (1400-4000Kn/m2)depends upon strength of rock.

- To increase q the foundation should be stepped at the base.

Continued.....



Continued…..

iii) Compression / crushing:

- dam may fail by failure of its material.
- when ∂ > ∂all –crushing may develop.
- ∂ compressive stress produced.
- ∂all – allowable compressive stress of am material (concrete).

Continued…..

- Vertical direct stress distribution at the base:
  - P = direct stress + bending stress
- Direct stress = ∑V/B*1
- Bending stress = M*Y/I
- pmin=∑V/B(1-6e/B) &
- pmax= ∑V/B(1+6e/B)
- e – eccentricity of the resultant force from the center of the base.(= B/2-X)
- X = ∑M/ ∑V

Continued……..

-When the reservoir is full:

- the resultant is nearer to toe.
- max. stress occurs @ toe.
- min. stress occurs @ heel.

- When the reservoir is empty:

- the resultant is nearer to heel.
- max. stress occurs @ heel.
- min. stress occurs @ toe.

Continued…….

- If Pmin < 0, tension is developed @ either ends.
- If Pmax. > ∂all (2000-4000KN/m2)dam can be failed by crushing.

Continued……

iv) Development of tension:

- concrete is low in tension so the max. tensile stress produced should not exceed allowable (500KN/m2).

- to ensure no tension is developed anywhere pmin > 0. or the resultant should pass the middle third of the base.

Stress analysis

- Generally d/t types of stress are produced on the plane and in the body of the dam.

  a) vertical normal stress

  b) horizontal shear stress

  c) principal stress & intensity of shear stress on principal plane

a) Vertical normal stress:

- normal to the plane
- produced on the horizontal plane
- = direct stress + bending stress

Pvu = $\sum V/B(1-6e/B)$ &

Pvd = $\sum V/B(1+6e/B)$ during the reservoir is full.

Where ∂u & ∂d are vertical stresses @ heel & toe of the dam respectively.

Continued……

b) Horizontal shear stress:

- generated by the variation of vertical normal stress over a horizontal plane.

- designated as ζu & ζd for u/s and d/s horizontal shear

- ζu = (pw - Pvu) tan(Øu)

- ζd = (Pvd - pw) tan(Ød)

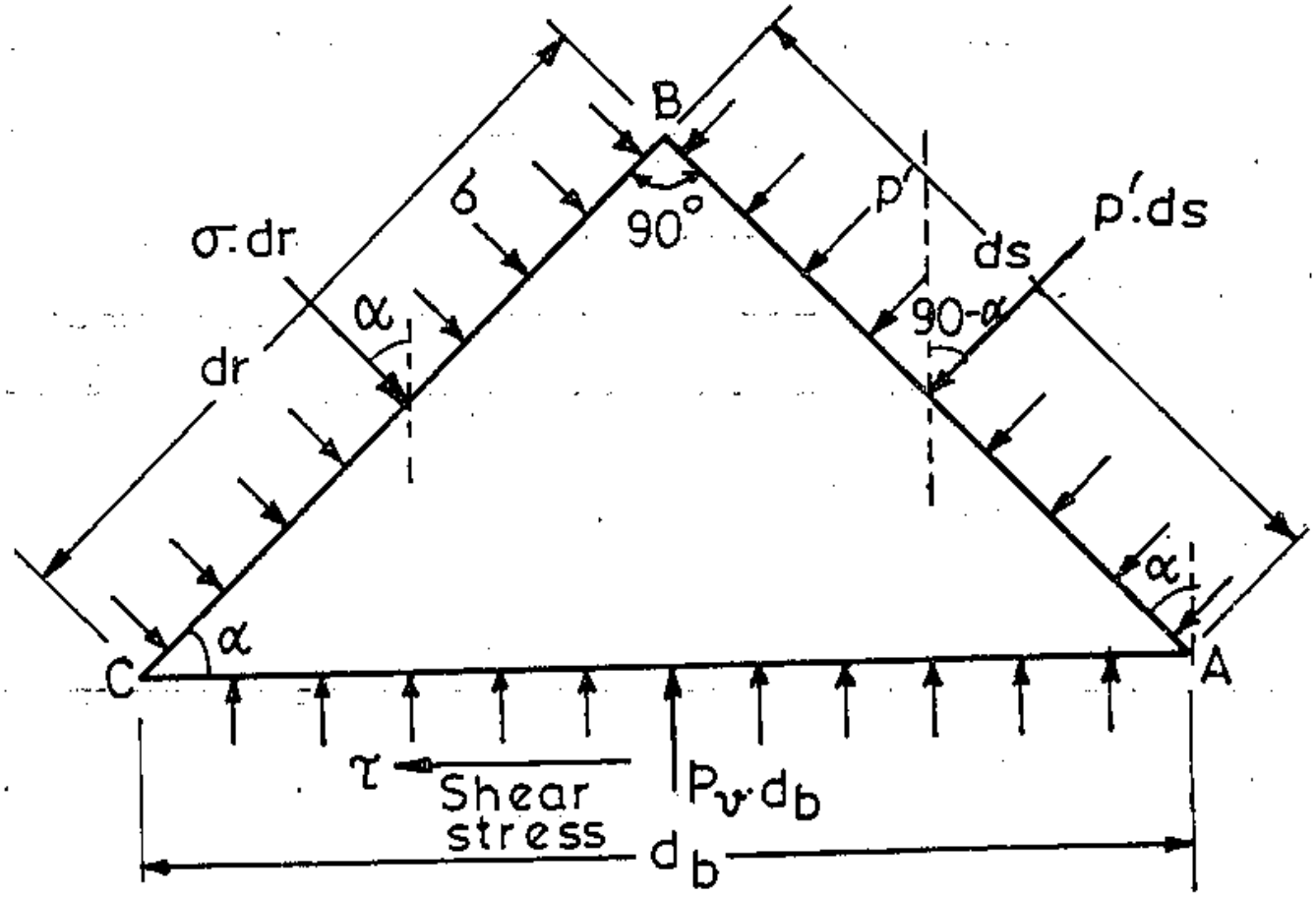
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Principal stress:

- designated as ∂
- max. stress produced with in the body of the dam
- consider a small triangular element

Having 3 principal plane of ds , dr & db dimension each.

Continued.....



Continued…..

- A principal plane AB is subjected to water pressure exerted by tail water.
- BC is subjected to principal stress.
- AC ---- vertical normal stress.
- ds , db & dr are lengths.
- Resolving all forces in vertical and solve for ∂:

$$\partial = Pv \sec^2 a - P' \tan^2 a, \text{ max. if } P' = 0$$

Continued…

-Resolving all the forces in horizontal and solve for ζ.

- ζ = cos(a)sin(a)[∂-P’]
- ζ is max. when P’ = 0

- When the foundation accelerates by effect of earthquake
- ∂toe = Pv sec^2a – (P’-P’e) tan^2a
- ∂heel = Pv sec^2a – (P+Pe) tan^2a

Similarly:

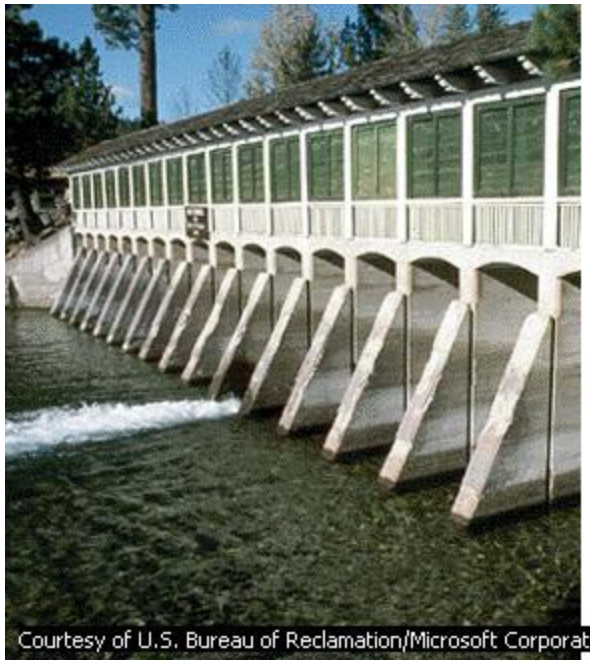
ζtoe = cosa*sina*[∂-(P’-P’e)]

ζheel = cosa*sina*[∂-(P+Pe)]

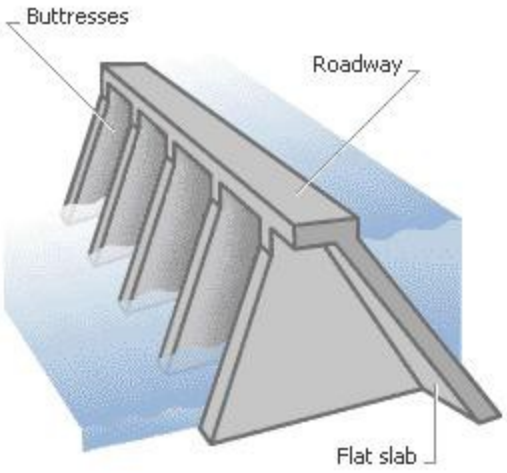
- Example:

Buttress dam

- It is a hollow gravity dam with watertight u/s face supported by a series of buttresses.
- Suitable in areas having wide river valleys
- It requires skilled manpower & supervision .



Courtesy of U.S. Bureau of Reclamation/Microsoft Corporation Illustration



Flat slab buttress dam

- Advantages of buttress dam over gravity dam:

  - reduction of uplift

  - requires smaller concrete

  - less massive than gravity dam so it requires weak foundation.

Disadvantages:

1) needs more reinforcement bars.

2) needs more skilled labor

3) slabs and columns are highly stressed.

4) more susceptible to damage by sabotage

- Classification of buttress dam:

According to the water supporting membrane utilized in the body, they can be:

- flat/simple slab deck
- multiple arch type
- other types

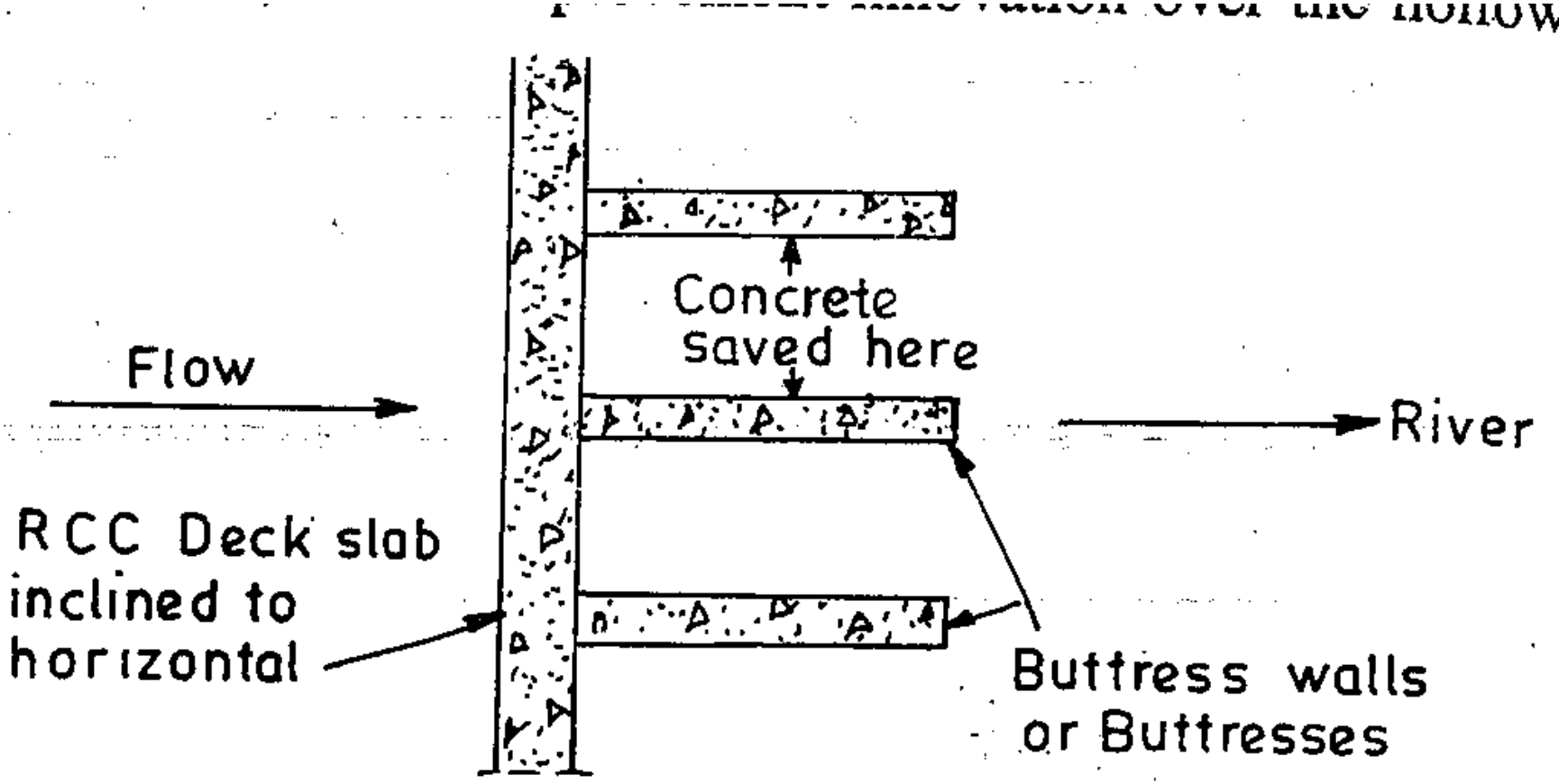
- 1) simple slab deck:

  - also named as Ambersen dam.

  - they are preferred for smaller heights

  - inclined flat slab supported by buttresses.

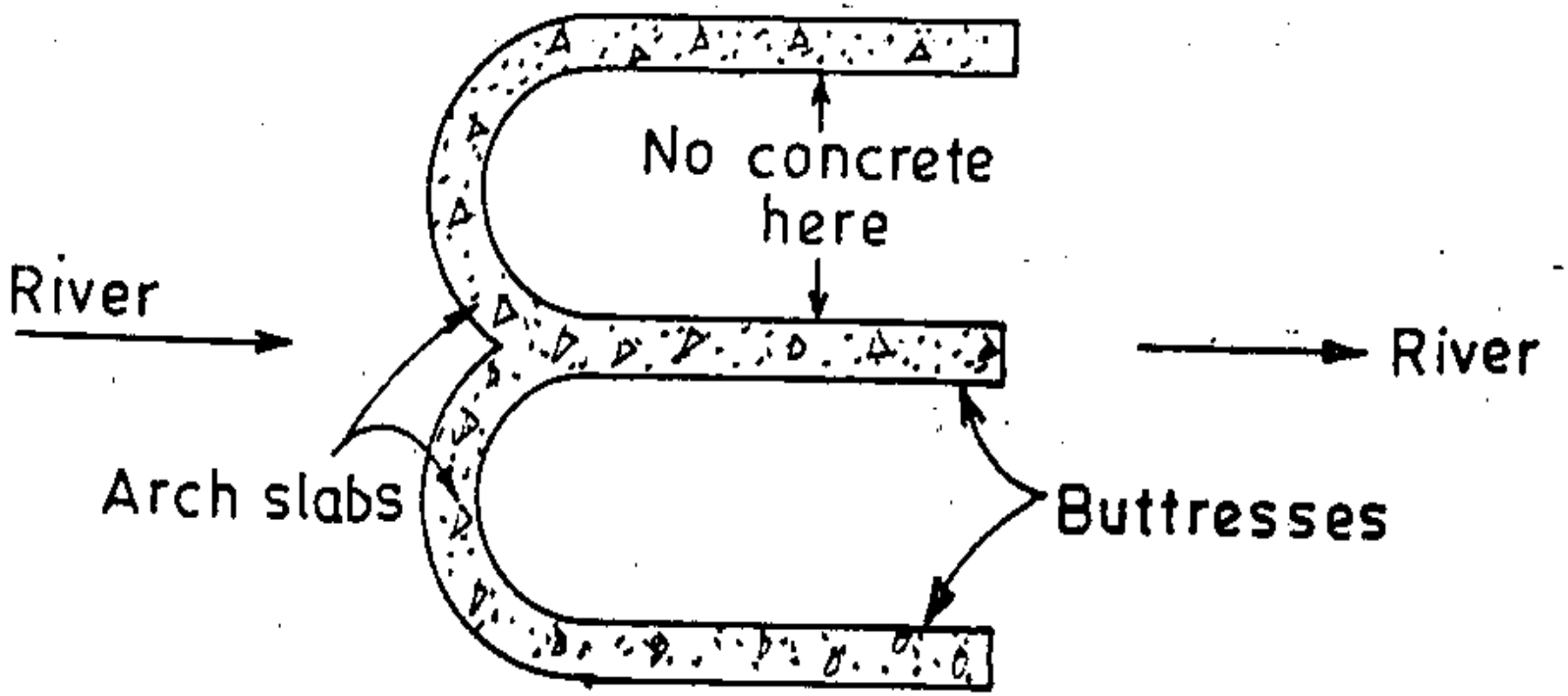
  - the deck slab is inclined with horizontal by about 40^0-55^0


Concrete
saved here
Flow
River
RCC Deck slab
inclined to
horizontal
Buttress walls
or Buttresses

Spacing b/n buttresses

- Height of dam
- Length of dam
- Presence of spillway over the dam
- Slope of u/s deck slab

- 2) multiple arch type:
  - has several arch water supporting membrane
  - preferred for higher heights


No concrete here
River
River
Arch slabs
Buttresses



Design principle of buttress dam

- The stability analysis of buttress dam is done in a similar way as for a gravity dam.

- the design is carried out by considering the full panel unlike gravity dam.

- buttress dam is subjected to the same force as gravity dam.

- as buttress dam reduces uplift pressure, uplift is acted only on buttress head.

Safety factors:

- f.s.o = $\sum M+/\sum M- > 1.5$ (NLC)
- f.s.s = $(C*A + u*\sum V)/\sum H > 2.4$ (NLC)

Example:

Arch concrete dam

- A concrete dam with u/s curvature resisting the imposed load by arch action.

- Transmits the major portion of water load to abutments, so large horizontal reaction is required by the abutment.

- Abutment should be sound rock and stable to resist the horizontal thrust developed by arch action.

Topography:

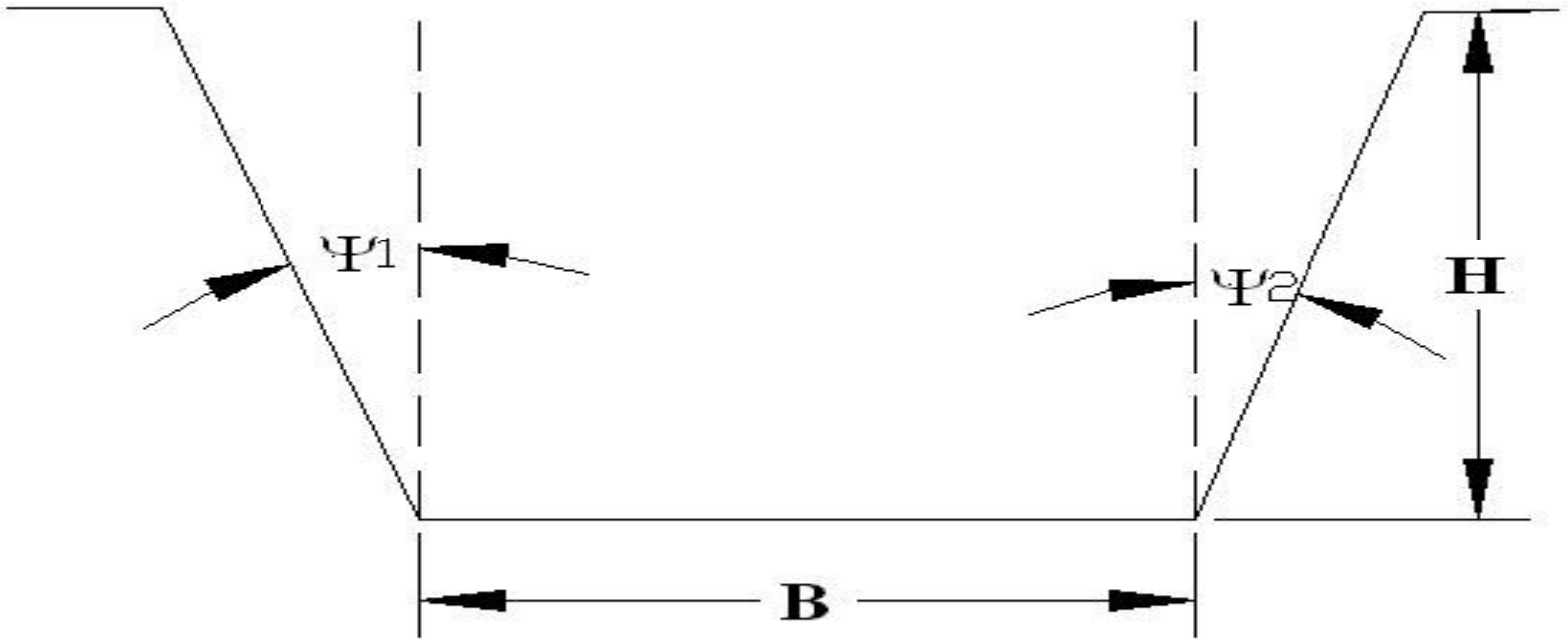
- narrow gorges/ v- shaped valley

According to sarkaria,

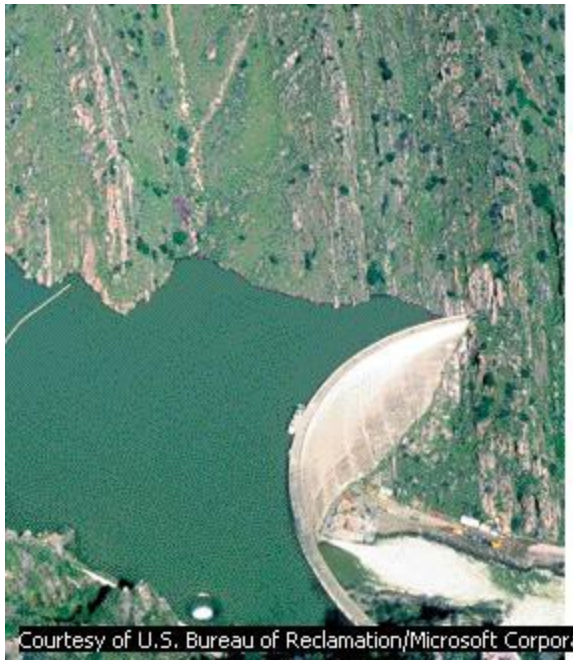
$$Sr = [B + H(\tan\emptyset_1 + \tan\emptyset_2)]/H$$

< 5

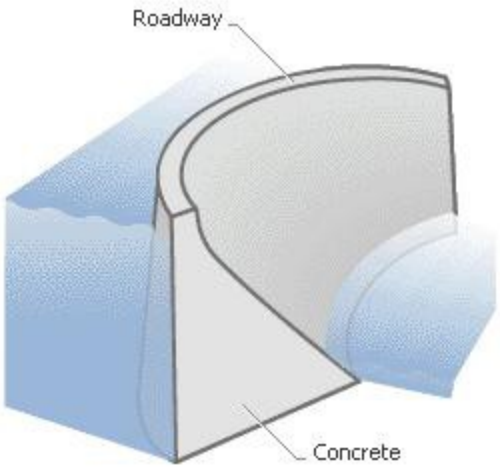
Where, Sr ratio of crest length to dam height. Usual value of sr (2-5)



Valley Shape



Courtesy of U.S. Bureau of Reclamation/Microsoft Corporation Illustration



Arch dam

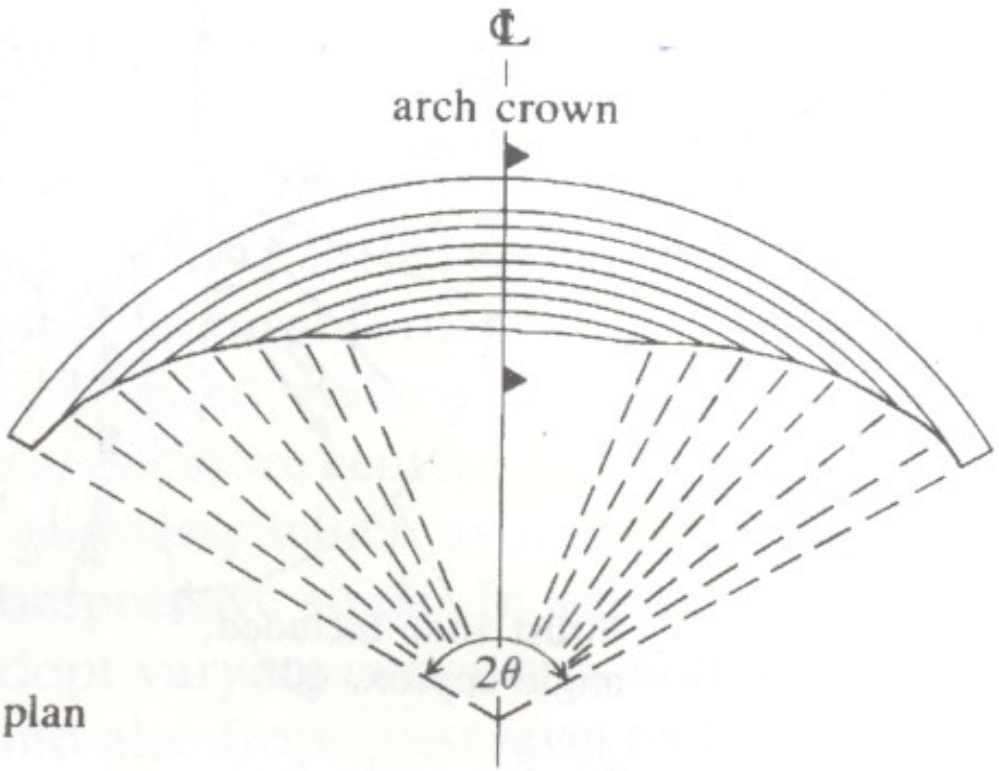
Types of Arch dam

- Divided in to the following types:
    1) constant radius arch dam
    2) constant angle
    3) variable radius and angle

1) Constant radius Arch dam

- Simplest type of dam can be analyzed and constructed easily.
- Re is constant @ all elevation & it has vertical u/s face with uniform radial d/s slope.
- 2a (central angle) is variable @ all elevation & max. @ top
- Suitable to relatively wide(U – shaped) valley.
- requires max. concrete as compared with others.
- 2θ at the top Arch ranges 70^0-110^0

(a) Constant radius arch dam



plan

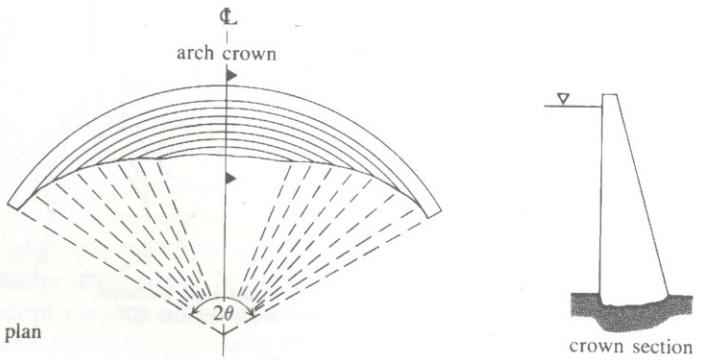


crown section

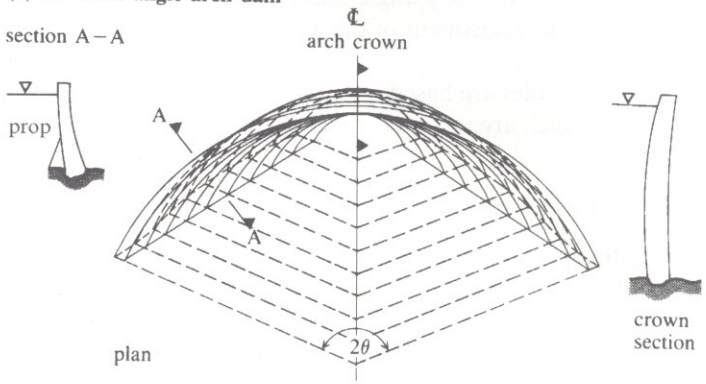
2) Constant angle Arch dam

- requires 70% of concrete utilized by constant radius dam.
- 2a is constant @ all elevation but the radiuses (Re & Ri) are variable.
- Complex than the previous type to analyze and construct.
- requires narrow & steep sided v-shaped valley.
- Re is max. @ center / crown.

(a) Constant radius arch dam



(b) Constant angle arch dam



3) Variable radius & angle Arch dam

- It is a compromise b/n constant angle & constant radius arch dam.
- neither the radius nor the central angle is constant.
- radius (Re & Ri) is max. @ top and min. @ the base.
- requires minimum amount of concrete as compared with others so best economical type.
- Adaptable to v-shaped/steep-sided valley.
- The design and construction of dam is so complex and beyond the scope of the course.

Design analysis of arch dam

- Loads acting on arch dam:

  - Loads on arch dams are essentially the same as loads on gravity dams.

  - Uplift forces are less important,

  - Internal stresses caused by temperature change , ice pressure, and yielding of abutment are very important

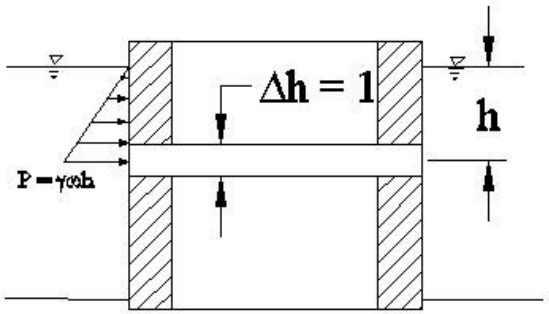
Design of arch dam

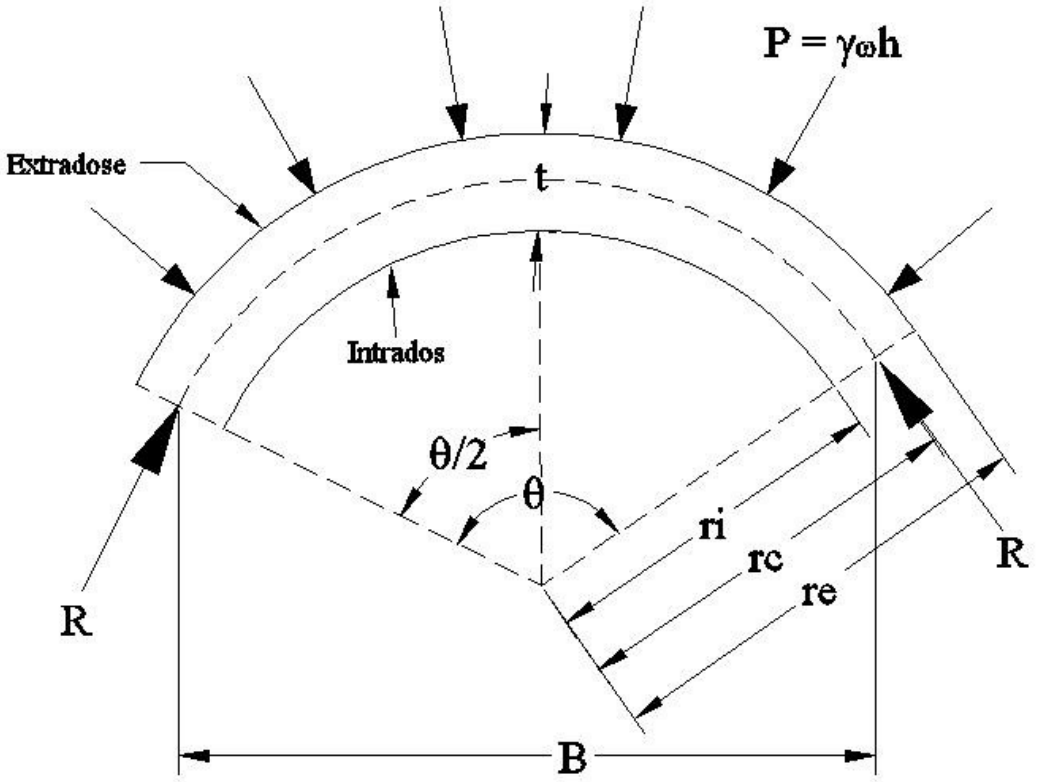
Methods of design of arch dams.

- thin cylinder theory
- Thick Cylinder Theory
- elastic theory
- trial load method

Thin cylinder theory:

- The theory assumes that the arch is simply supported at the abutments and that the stresses are approximately the same as in thin cylinder of equal outside radius re.


Δh = 1
h
P = γωh


P = γωh
Extradose
t
Intrados
θ/2
θ
ri
rc
re
R
R
B

Summing forces parallel to the stream axis:

$2R\sin\theta/2 = 2\gamma h\, re \sin\theta/2$

$R = \gamma h\, re$, where R is reaction by abutment

- the compressive stress developed in the arch ring at the abutments can be:

$$\partial = R/(t*1) = \gamma h\, re/t,$$

If $\partial < \partial all$ safety against compression is ensured. So the minimum thickness is

$$t > \gamma h\, re/\partial all \text{ for constant radius arch}$$

re = ri + t, substitute in the previous equation

t > γh ri/ (∂all- γh) for constant angle arch

Condition for least volume of arch(economical central angle):

V = t*re θ*1

- t = γh re/ ∂all = K*re
- Re = B/2 sinθ/2
- Differentiating *V with respect to* θ *and setting to zero,* θ = *133.5^o which is the most economical* angle for arch with minimum volume.
- For θ = 133.5^0 re/ri = 0.544B

Limitation of thin cylinder theory:

- assumes the arch is simply supported at abutment.

- the arch is considered as a thin cylinder of equal re

- doesn't consider the bending moment developed at abutment

- internal stress by temperature change and yield of abutment is not accounted for.

Thick Cylinder theory

- According to this theory the compressive horizontal thick ring arch stress can be computed as follow,
- ∂hu = γh(re2+ (re2 * ri2 / r2))/(re2- ri2)
- This stress is max. at d/s face when r=ri
- ∂hd = 2γh re2/(t*(re+ ri)

similarly the maximum compressive stress at the abutment is equals to Kr* ∂hd

Where Kr is a correction factor depends up on 2a and re/t…….look the curve

- **Design example 1**
- Design a 100m height constant radius arch dam, by the thin cylinder theory for a valley 100m wide at the base and 150m wide at a height of 100m ∂all = 4MPa? Take θ = 140^0

- Solution

  Re = B/2 sinθ/2 = 79.8=80m

  t > γh re/ ∂all

- **Design example 2**

  Design a 100m high constant angle arch dam by thin cylinder theory for a valley 40m wide at the base and 240m wide at a height of 100m. Take ∂all = 5MPa.

- Solution

Taking $\theta = 133.440$

$ri = B/2 \sin \theta /2 = ri = 0.544B$

$t > \gamma h\ ri/ (\partial all- \gamma h)$

$re = ri + t$

CHAPTER-3
Embankment dam

Defn.: - a type of dam built of naturally excavated materials (clay sand rock stones etc).

Topography:

- ED is best suited to wide/u-shape valley

- adaptable to any foundation

- preferred to areas in rich of natural materials.

Types and classification

Based on method of construction:

1) rolled fill dam
2) hydraulic fill dam

1) Rolled fill dam:

- the materials are excavated, transported from borrow pits by machineries and placed on the site with applying compaction to form limited layer of proper water content.

2) Hydraulic fill dam:

- the materials are excavated transported and placed by hydraulic method.

- the flumes are pumped suitable materials mixed with water at borrow pits and discharge the slush through the outlet at a suitable interval.

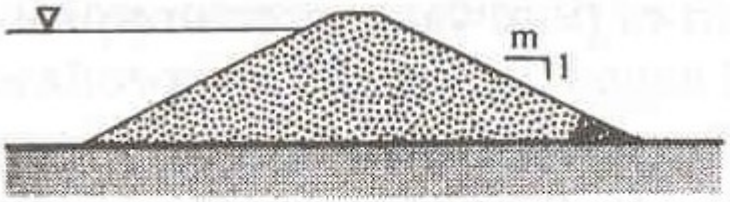
- finally the materials get deposited over the site, the coarser materials of the slush settle at outer edge & the finer settles at the center with no compaction.

Rolled fill dam:

- homogenous
- Zoned
- Diaphragm

homogenous:

- built of single or one type of material exclusive of slope protections
- usually constructed at smaller height.



(a) Homogenous with toedrain: small secondary dams
$m = 2.0–2.5$

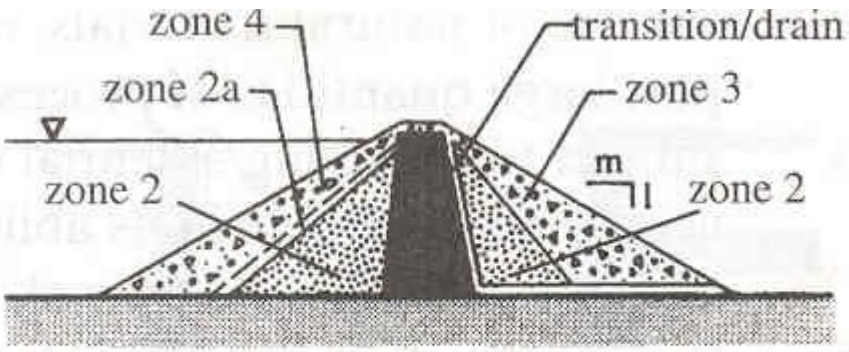
Zoned:

- contains different types of materials in d/t parts

- there is central impervious core(clay, cement) supported by outer shell.

- the outer pervious zone is outer shell provides support to core.

- u/s shell is responsible in providing of stability against sudden draw down of reservoir.

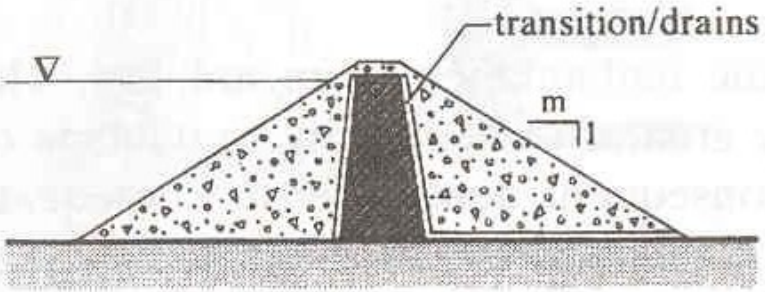


(f) Earthfill/rockfill with central rolled clay core: zoned with transitions and

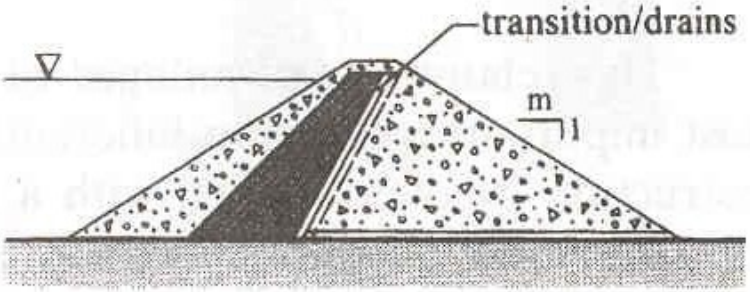
Diaphragm:

-the embankment is constructed of pervious materials with thin diaphragm of impervious materials.

- the diaphragm can be placed @ the center as a central core vertically or inclined as u/s blanket.



(a) Central rolled clay core
$m = 1.6-2.0$



(b) Inclined rolled clay core
$m = 1.6-2.0$

Causes of Failure of Earth Dams:

Mode of failure of ED

- Hydraulic failures : 40%
- Seepage failures : 30%
- Structural failures: 30%

1) <u>Hydraulic Failures</u>: Hydraulic failures include the following:

- Overtopping
- Erosion of U/S face
- Erosion of D/S face
- Erosion of D/S toe

2) Seepage failures: Seepage failures may be due to:

a) Piping through the body of the dam

b) Piping through the foundation of the dam

d) Sloughing of downstream toe/face.

3) Structural Failures: Structural failures may be due to the following reasons:

- Upstream and Downstream slope failures due to pore pressures
- Upstream slope failure due to sudden draw down
- Down stream slope failure during full reservoir condition.
- Foundation slide
- Failure due to Earth quake
- Slope protection failures
- Failure by burrowing animals

Criteria for Safe Design of Earth Dam

1) No overtopping:

- Accurate estimation of design flood
- Adequate spillway and Sufficient outlet works
- Sufficient free board

2) No seepage failure

- Phreatic line should not touch d/s face
- Seepage through the body of the dam, foundation and abutments should be controlled by adapting suitable measures.

- There should be no opportunity for free passage of water from U/S to D/S both through the dam and foundation.

3) **No structural failure**:

- Safe U/S & D/S slope during construction
- Safe U/S slope during sudden draw down condition.
- Safe D/S slope during steady seepage condition
- Earth quake resistant dam

4) Proper slope protection against wind & rain drop erosion.

5) Proper drainage

6) Economic section

Preliminary design of ED

1) Top width
2) Free board
3) Casing or outer shells
4) Central impervious core
5) Cut-off trench
6) Downstream drainage system

1) Top width:

- The crest width of an earth dam depends on the following considerations:
  - minimum percolation distance.
  - Height of the structure
  - Importance of the structure
  - Width of highway on the top of the dam
  - Protection against earthquake forces.

Empirical formula:

b = H/5 + 3 , For very low dam (H<10m)

$b=0.55H^{1/2}+0.2H$, For medium dam (10m<H<60m)

$b=1.65(H+1.5)^{1/3}$, For large dam (H>60m)

2) Free board.

- Free board is the vertical distance between the horizontal crest of the embankment and the reservoir level.

- According to U.S.B.R free board is depending upon nature of spillway and height of dam.

- controlled spillway(< 60m)=2.5m above top of gate,

-controlled sp(>60m)=3m above top of gate

- free sp (any h) = (2-3m)over max flood level

3) casing/ shells:

The design slopes of the u/s and d/s embankments may vary depending upon:

- the character of the materials available,
- foundation conditions
- the height of the dam.
- The slopes also depend up on the type of the dam (i.e. homogeneous, zoned or diaphragm).

- u/s slope varies (2:1-4:1) or stability (H:V)
- Usual D/s slope varies(2:1-3:1)

Seepage Analysis

Seepage analysis: is used

•To determine the quantity of water passing through the body of the dam and foundation.

•To obtain the distribution of pore water pressure.

Seepage Analysis

Seepage analysis: is used

- To determine the quantity of water passing through the body of the dam and foundation.
- To obtain the distribution of pore water pressure.

Assumptions to be made in seepage analysis:

1) materials in embankment and foundation are incompressible and isotropic

2) the seepage water flows under a hydraulic gradient due to gravity (Darcy law is valid)

3) Quantity of water entering in to and leaving out from a small element of soil is equal.

Laplace equation:

$$\partial^2 Ø / \partial X^2 + \partial^2 Ø / \partial Y^2 = 0$$, this eqn. can be solved by graphically analytically or experimental method.

- The solution gives two sets of curves known as flow net

Computation of rate of seepage from flow net

- A portion between any two successive flow lines is known as flow channel.
- A portion enclosed b/n two equipotential lines and successive flow lines is known as field.

Then, from Darcy's law of flow through soils:

$$q = b*k\Delta h/l,\ b=l$$

- If N_d = total number of potential drops in the complete flow net,

$$\Delta h = h/Nd$$

Total discharge $q = b*Nfkh/Ndl = k*h*Nf/Nd$ for isotropic soil.

- For anisotropic soil

$$q = \sqrt{(Kx*Ky)}h*Nf/Nd$$

Where Nf no of flw channel

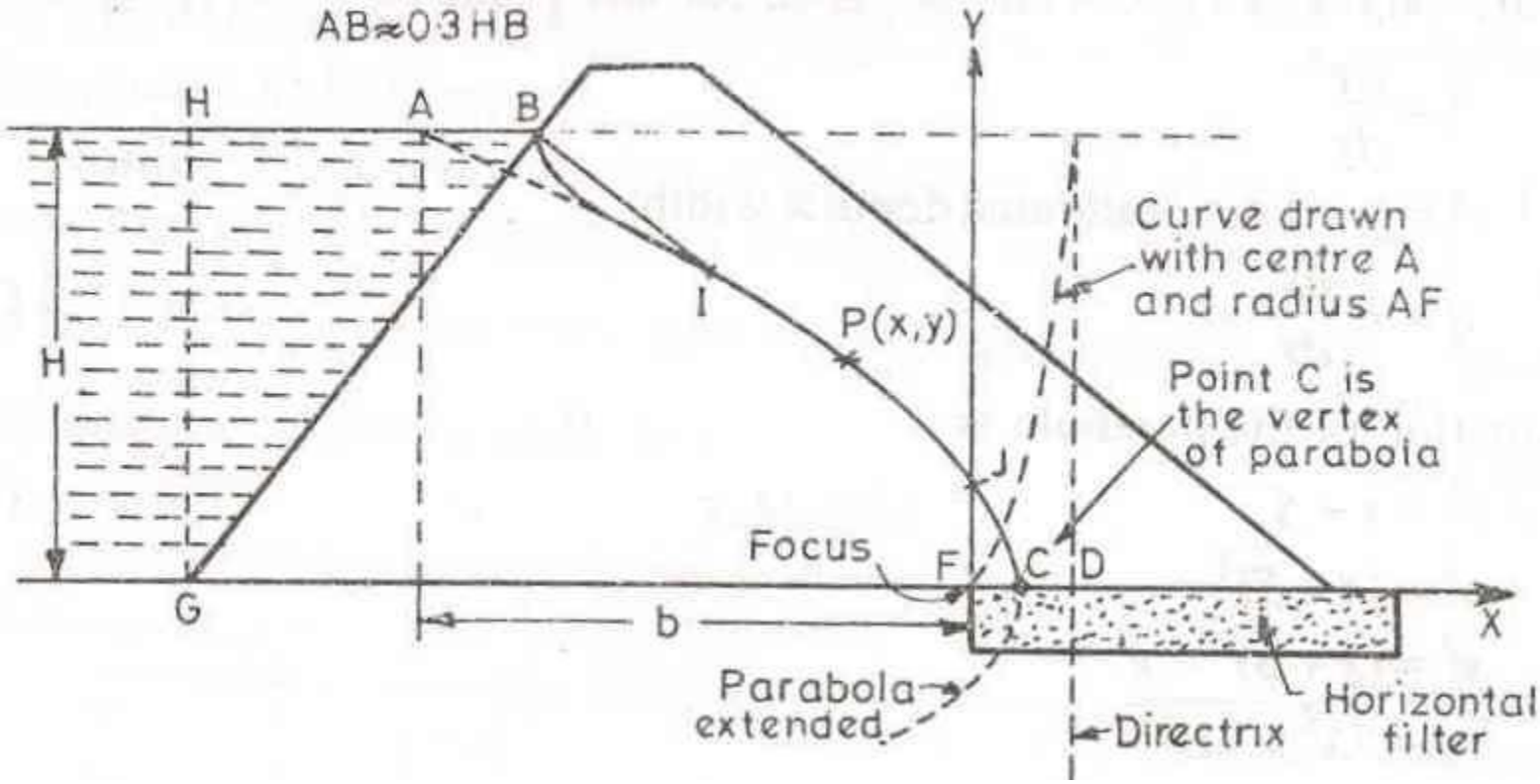
Nd no of equi. Line

h head causing flow

Phreatic Line in Earth Dam

Phreatic line / seepage line / Saturation line

- is the line at the upper surface of the seepage flow at which the pressure is atmospheric.


AB≈0·3HB
H
A
B
Y
Curve drawn
with centre A
and radius AF
I
P(x,y)
H
Point C is
the vertex
of parabola
J
Focus
F
C
D
G
b
X
Parabola
extended
Directrix
Horizontal
filter

Determination of Phreatic Lines

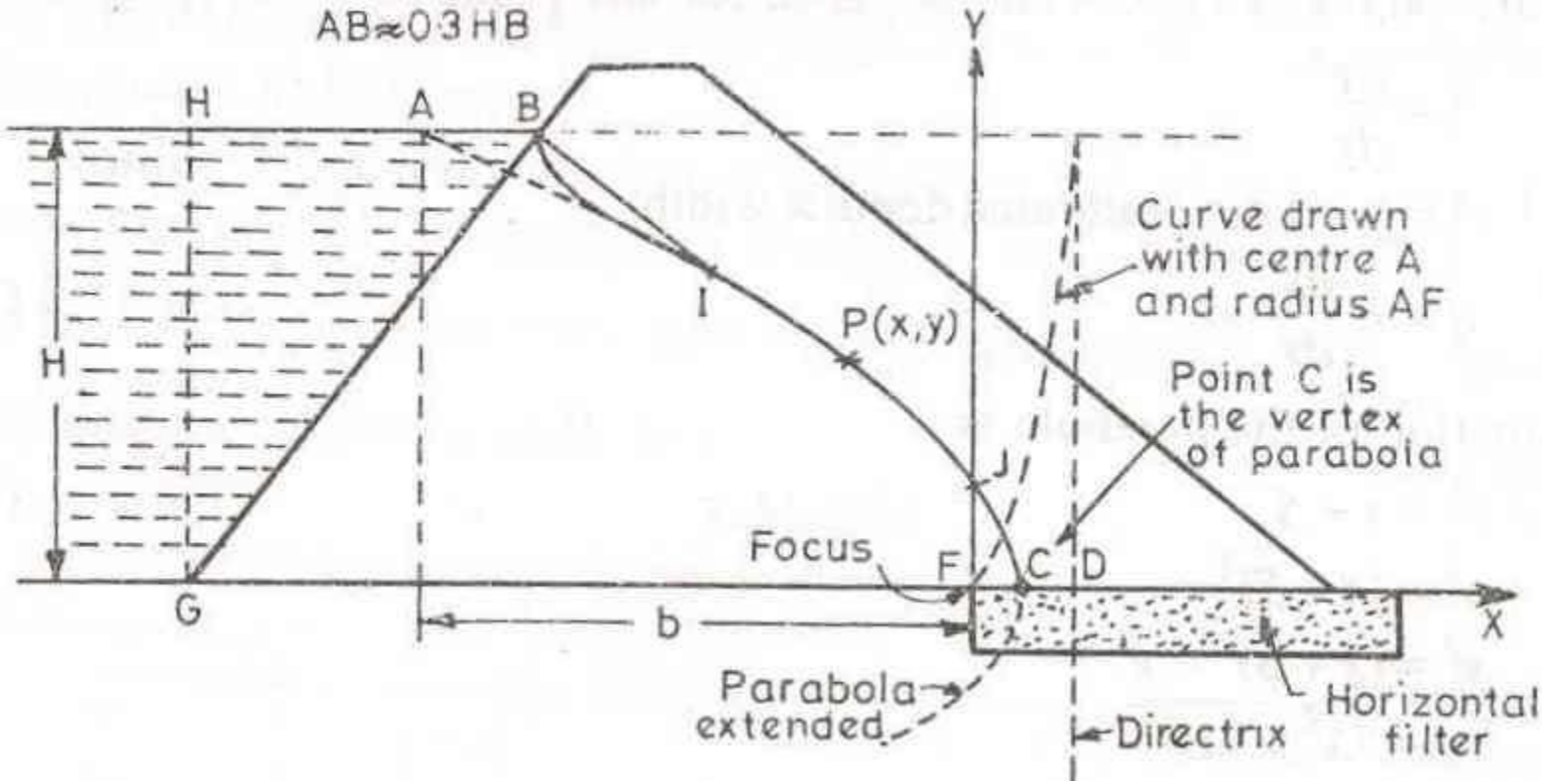
Determining of the position of phreatic line helps to know:

i) The divide line between the dry (or moist) and submerged soil.

Ii) The top stream line helps us in drawing the flow net.

Iii) To ensure that the phreatic line doesn’t cut the downstream face of the dam

1) a homogeneous Earth dam with horizontal Drainage blanket



- **Graphical method**
- **<u>Steps</u>:**
- Starting point of base parabola is @ A,AB = 0.3L
- F is the focal point located @ d/s filter
- Draw a curve passing through F center @ A
- Draw a vertical line EG which is tangent to the curve
- EG is the directrix of the base parabola
- Plot the various points P on the parabola in such a way that PF = PR

- **Analytical method**

$PF = PR,\ (x^2+y^2)^{1/2} = x+s$

From point A (known), x = b and y = h

$s = (b^2+h^2)^{1/2} - b$, s is focal distance

$Y = (2xs + s^2)^{1/2}$ is equ. Of parabola

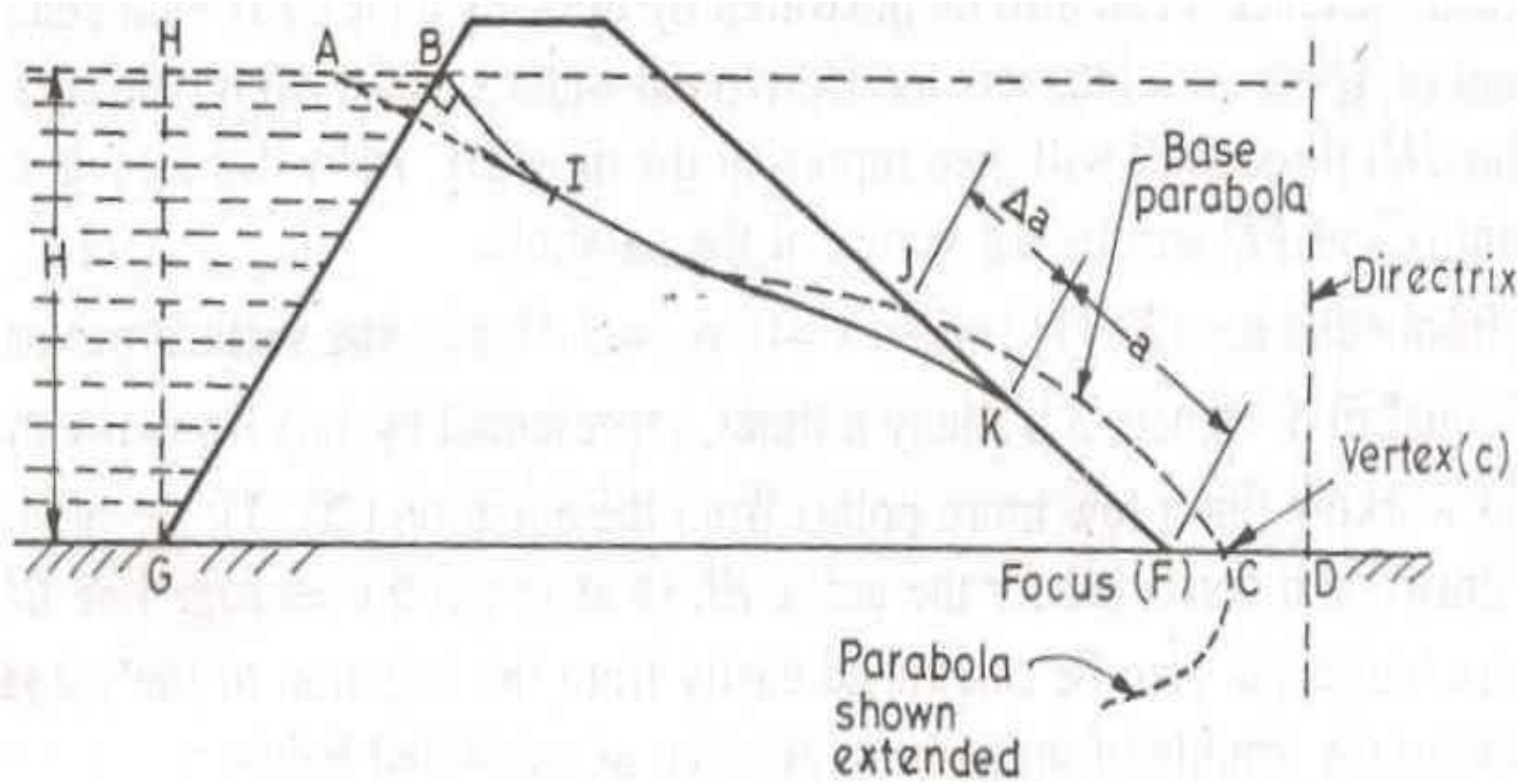
Discharge through the body of Earth dam

from darcy $q = kiA$, $i = dy/dx$, $A = y*1$

$Y = (2xs + s^2)^{1/2}$

$q = K*S$

2) a homogeneous Earth dam with no horizontal Drainage blanket



- **Graphical method**
- **<u>Steps</u>:**
- Starting point of base parabola is @ A,AB = 0.3L
- F is the focal point located @ toe of the dam
- Draw a curve passing through F center @ A
- Draw a vertical line EG which is tangent to the curve
- EG is the directrix of the base parabola
- Plot the various points P on the parabola in such a way that PF = PR

- The phreatic line is corrected at exit as emerged out at k and meets the d/s face tangentially.
- The correction JK (say Δa) by which the parabola is to be shifted downward can be determined as follows:

$\Delta a = (a + \Delta a)[(180^0 - a)/400^0]$

- a (The portion KF) is known as discharge face and always saturated.

- **Analytical method**

$$PF = PR, \quad (x^2+y^2)^{1/2} = x+s$$

From point A (known), $x = b$ and $y = h$

$s = (b^2+h^2)^{1/2} - b$, s is focal distance

$Y = (2xs + s^2)^{1/2}$ is equ. Of parabola

Discharge through the body of Earth dam

Case 1: when $\alpha < 30^{0}$

in such a case the hydraulic gradient of the seepage line is approximately equal to slope of d/s face of dam.

From Darcy's law,

$$q = k*i*y = K* dy/dx * y \text{--------(1)}$$

dy/dx – hydraulic gradient = tan a(slope of d/s face of dam)

$y = a*\sin \alpha$

$$q = k*\tan \alpha * a\sin \alpha \text{.....(2)}$$

- K* dy/dx * y = k*tan α*asin α
- Integrating between the limits:

x= a (Cos α) to x = b’

y= a (sin α) to y = h

, we get

$a = b'/ \cos \alpha - [b'^2/ \cos^2 \alpha - h^2/\sin^2 \alpha]^{1/2}$

$q = k*\tan \alpha * \text{asin}\ \alpha$

Case 2: when α lies b/n (30^0-60^0)

- for steeper slope Casagrande suggested that the use of sin α instead of tanα & (dy/ds) instead of (dy/dx), where s is the distance measured along the phreatic line.

From Darcy's law,

$$q = k*i*y = K* dy/ds * y \text{--------(1)}$$

dy/ds – hydraulic gradient = tan a(slope of d/s face of dam)

$$y = a*\sin \alpha$$

$$q = k*\sin \alpha * a\sin \alpha \text{.....(2)}$$

- $K^* dy/ds * y = k^* a \sin^2 \alpha$
- Integrating between the limits:

$s_1 = a$ to $s_2 = S$ y $= a^* \sin \alpha$ to $y = h$

Where S = d/ce measured along the curve

, we get

$$a = S - [S^2 - h^2/\sin^2 \alpha]^{1/2}$$

approximately $S \approx (h^2 + b^2)^{1/2}$

Finally we get

$$a = (h^2 + b^2)^{1/2} - (b^2 - h^2 * \cot^2 \alpha)^{1/2}$$

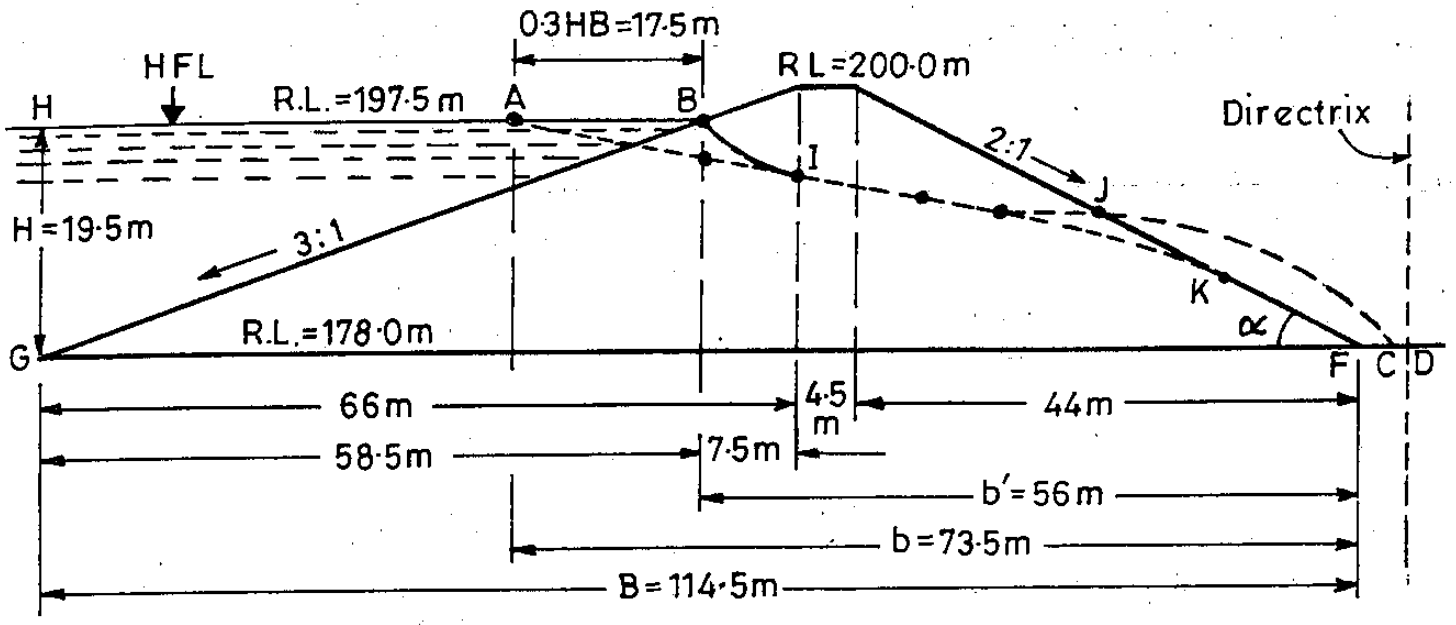
$$q = k^* a \sin^2 \alpha$$

Example 20.1. *An earth dam made of a homogeneous material has the following data :*

Coefficient of permeability of dam material	$= 5 \times 10^{-4}$ *cm/sec.*
Level of top of dam	$= 200.0$ *m.*
Level of deepest river bed	$= 178.0$ *m*
H.F.L. of reservoir	$= 197.5$ *m.*
Width of the top of dam	$= 4.5$ *m.*
Upstream slope	$= 3 : 1.$
Downstream slope	$= 2 : 1.$

Determine the phreatic line for this dam section and the discharge passing through the dam.

Solution. Taking the focus (F) at the d/s toe of the dam as the origin, the equation of the base parabola is given by $\sqrt{x^2+y^2} = x + S$



the point A, in such a way that

$$AB \approx 0.3\, HB$$

or

$$AB \approx 0.3 \times 58.5 = 17.5 \text{ m.}$$

Plot the point A at a distance 17.5 m from B (Fig. 20.17). This is the start-point of the parabola. Now the coordinates of the point A w.r. to F as origin are (73.5 m, 19.5 m). Substituting these ordinates in the equation of the base parabola, we get

$$\sqrt{(73.5)^2 + (19.5)^2} = 73.5 + S.$$

or

$$\mathbf{S = 2.54 \text{ m.}}$$

The vertex (C) of the base parabola shall be situated at a distance equal to $\frac{S}{2}$ or 1.27 m from F, beyond the d/s toe of the dam, as shown in Fig. 20.17.

A few more coordinates of the base parabola at known distances (x) are worked out in table 20.4 using,

$$x^2 + y^2 = x^2 + S^2 + 2xS$$

or

$$y = \sqrt{S^2 + 2xS}$$

x	$y^2 = 2xS + S^2 = 5.08x + 6.45$	$y = \sqrt{S^2 + 2xS}$
0	6.45	2.54
10	57.25	7.68
20	108.05	10.39
30	158.85	12.61
40	209.65	14.48
44	229.97	15.16
48.5	252.83	15.92
56	290.93	17.04
60	311.25	17.64
70	362.05	19.02
73.5	380.00	19.50

Note. The value of 'a' can also be determined from Eq. (20.22), if $(a + \Delta a)$ is not to be measured, as the dam section is not to be plotted to scale.

In that case $$a = \frac{b'}{\cos a} - \sqrt{\left(\frac{b'}{\cos \alpha}\right)^2 - \left(\frac{H}{\sin \alpha}\right)^2}$$

where $b' = 56$ m

$H = 19.5$ m

$\sin 26.56° = 0.447$

$\cos 26.54° = 0.894.$

$$a = \frac{56}{0.894} - \sqrt{\left(\frac{56}{0.894}\right)^2 - \left(\frac{19.5}{0.447}\right)^2}$$

$$= 62.6 - \sqrt{3{,}920 - 1{,}900} = 62.6 - 44.9 = \mathbf{17.7\ m.}$$

Example 20.3. *For the dam section given in example 20.1, draw the seepage line if a horizontal filter of length equal to 25 m is provided inward from the downstream toe of the dam.*

Solution. The dam section is plotted from the dimensions given in example 20.1. The horizontal filter is also provided. Now, the left end of the filter will act as a focus and is designated as F and is taken as origin.

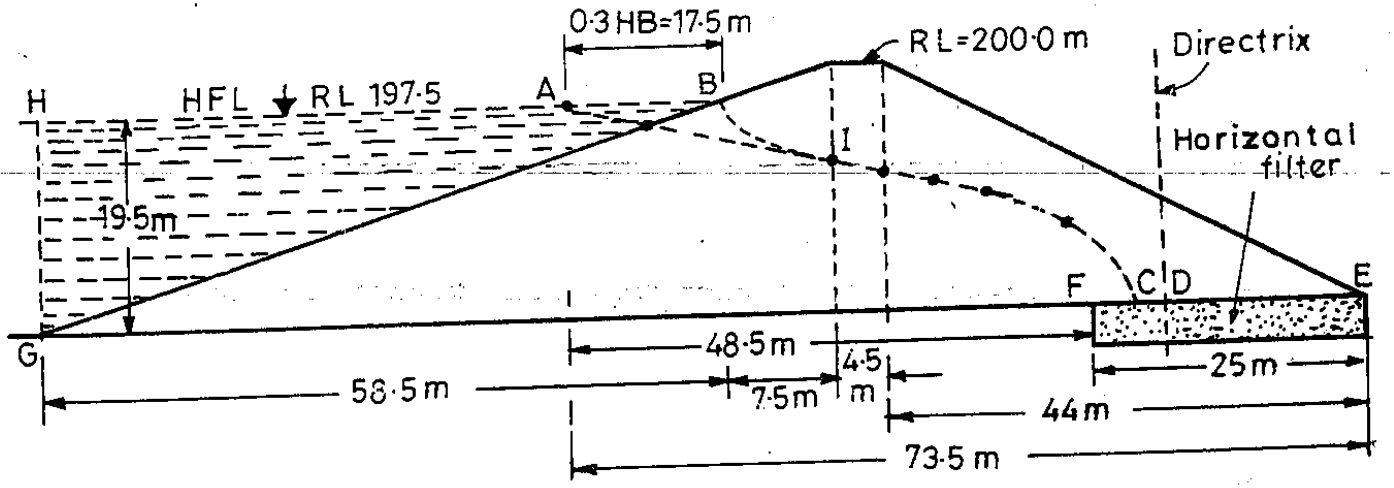


Fig. 20.18

The coordinates of any point (x, y) on a parabola of equation

$$\sqrt{x^2+y^2} = x + S$$

will now be for the point A.

The point A is determined as in example 20.1. Its coordinates w.r. to F as origin are (48.5 m, 19.5 m)

$$\therefore \quad \sqrt{(48.5)^2+(19.5)^2} = 48.5 + S$$

or $$\sqrt{2{,}352+380} = 48.5 + S$$

or $$\mathbf{S = 3.77\ m.}$$

The vertex (C) of the parabola shall be situated at a distance equal to $\frac{S}{2}$, *i.e.* 1.83 m beyond the point F and directrix shall be at a distance 3.77 m from F.

At point F, $x = 0$

$\therefore$ $y = S = 3.77$ m.

A few more coordinates of the base parabola are worked out in table 20.5, using

$$x^2+y^2 = x^2+S^2+2xS$$

or $$y = \sqrt{2xS+S^2}$$

x	$y^2 = (2xS + S^2) = 7.54x + 14.21$	$y = \sqrt{2xS + S^2}$
0	14.21	3.77
10	89.61	9.47
15	127.31	11.27
19	157.47	12.54
23.5	191.40	13.85
31	247.95	15.75
40	315.81	17.78
48.5	380.00	19.5

Earthen Slope stability of Embankment dam

The stability analysis is done under the following four conditions:

1) Stability analysis of down stream slope during steady seepage.

2) Stability of up stream slope during sudden Draw down.

3) Stability of up stream & down stream slope during and immediately after construction.

4) Stability of foundation against shear.

Methods applied:

1) approximate method

2) Swedish circle method

1) Approximate method:

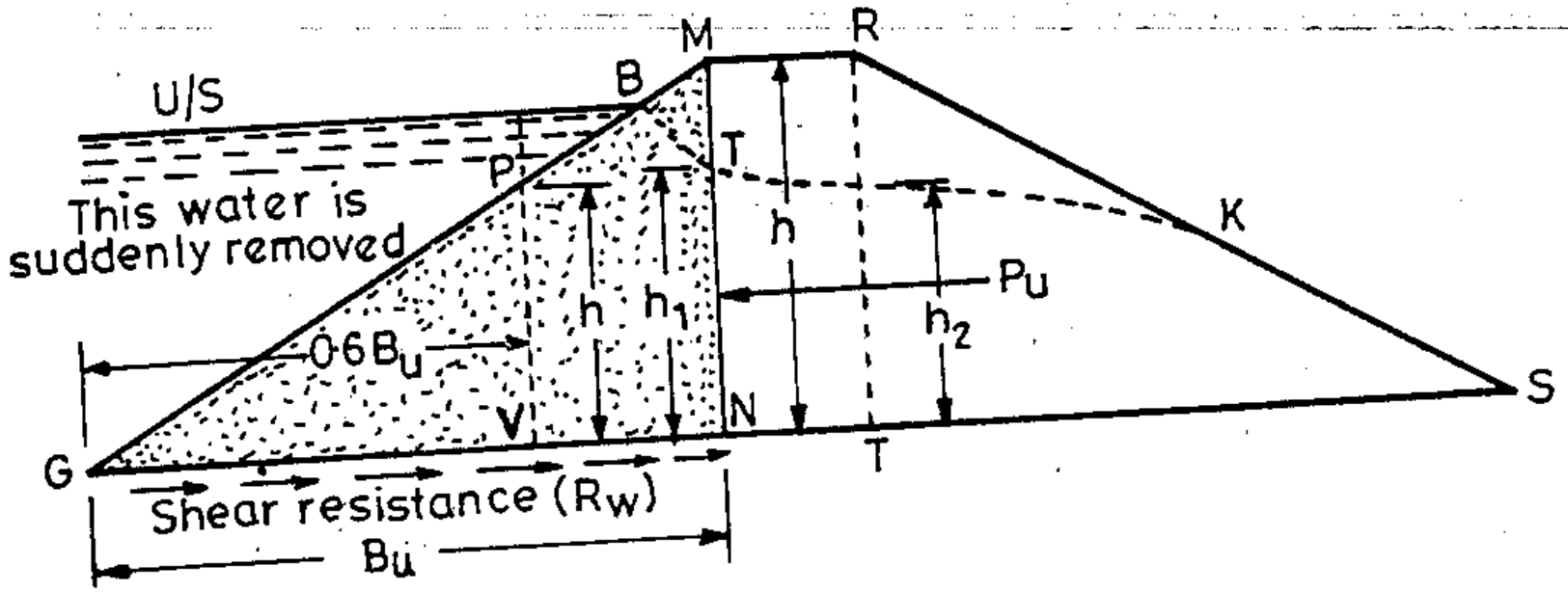
- a horizontal force pu is exerted by the saturated fill in u/s or d/s shoulder by the soil and the water contained with in the soil.

- a resistant shear force Ru is provided by the shear resistance developed at the base.

- safety against sliding is ensured by counterbalancing Pu by Ru.

- $Fs = Ru/Pu > 1.5$

Stability of u/s slope during sudden drawdown of reservoir



Considering a unit length of the dam, the horizontal force P_u is given by the equation

$$P_u = \left[\frac{\gamma_1 h^2}{2} \tan^2\left(45 - \frac{\phi}{2}\right) + \gamma_w \cdot \frac{{h_1}^2}{2}\right] \text{(say in kN)} \qquad \ldots(20.27)$$

where γ_1 is the weighted density at the centre of triangular shoulder upstream and is given by

$$\gamma_1 = \frac{\gamma_{sub} \cdot h_1 + \gamma_{dry} \cdot (h - h_1)}{h} \qquad \ldots(20.28)$$

Shear resistance (R_u) of u/s slope portion of the dam developed at base GN is given by

$$R_u = C + W \tan \phi \qquad \ldots(20.29a)$$

where W = the weight of the u/s triangular shoulder of dam

C = The total cohesive force developed at base GN.

If c is the unit cohesion of the dam soil, then $C = c \times (B_u \times 1)$ where B_u = length GN.

$$W = [\text{Area of } \Delta GMN] \cdot \gamma_{sub} = \gamma_{sub} \cdot \left(\frac{1}{2} \cdot Bu \cdot h\right)$$

$$R_u = C + W \tan \phi$$

$$= c \cdot (B_u \times 1) + (\gamma_{sub} \cdot \frac{1}{2} \cdot B_u \cdot h) \tan \phi$$

Now P_u and R_u are known, the factor of safety against sliding can be easily calculated, using

$$\text{F.S.} = \frac{R_u}{P_u} \qquad \ldots(20.30)$$

It should be more than 1.5.

The factor of safety calculated above is w.r. to average shear (τ_{av}), which will be equal to

$$\tau_{av} = \left(\frac{P_u}{B_u \times 1} \right) \qquad \text{...(20.31)}$$

It has been found by photoelastic studies, that the maximum intensity of shear stress occurs at a distance 0.6 B_u from the heel (*i.e.* 0.4 B_u from the top shoulder) and is equal to 1.4 times the average shear intensity.

∴ Maximum shear stress induced

$$= \tau_{max} = 1.4 \left(\frac{P_u}{B_u} \right)$$

which is developed at the point V (Fig. 20.26) such that $GV = 0.6\, B_u$.

The unit shearing resistance developed at this point V is give by

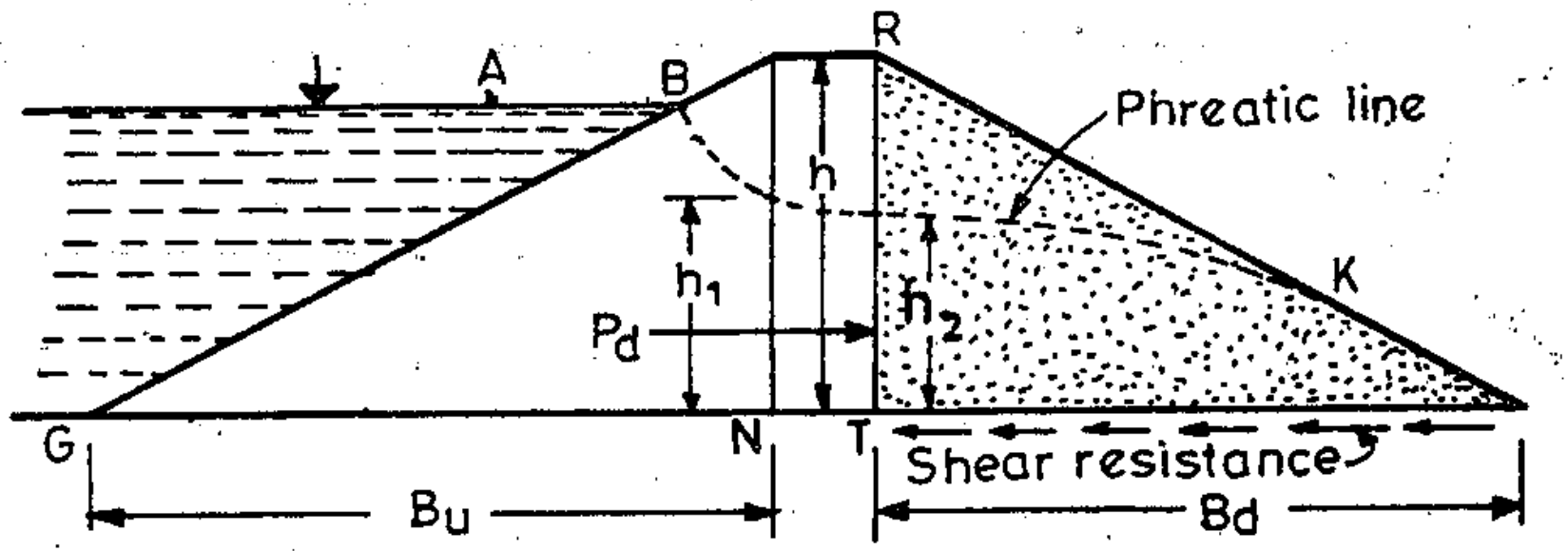
$$\tau_f = c + h' \gamma_{sub} \cdot \tan \phi$$
$$= c + 0.6\, h \cdot \gamma_{sub} \cdot \tan \phi$$

F.S. at the point of the maximum shear

$$= \frac{\tau_f}{\tau_{max}}$$

It should be more than 1.

Stability of d/s slope under steady seepage



The horizontal shear force P_d, in this case, is given by

$$P_d = \left[\frac{\gamma_2 \cdot h^2}{2} \tan^2 \left(45 - \frac{\phi}{2} \right) + \frac{\gamma_w \cdot h_2^2}{\cdot 2} \right] \qquad \text{...(20.35)}$$

where γ_2 is the weighted density at the centre of the triangular shoulder downstream, given by

$$\gamma_2 = \frac{\gamma_{sub} \cdot h_2 + \gamma_{dry} (h - h_2)}{h} \qquad \text{...(20.36)}$$

Shear resistance R_d of d/s slope portion of dam, developed at base is given by

$$R_d = C + W \tan \phi,$$

where W = The weight of the d/s slope portion of dam. (*i.e.* ΔRTS)

$C = c \times (B_d \times 1)$, where c is the unit cohesion.

$$W = \gamma_{sub} \cdot (\tfrac{1}{2} \cdot B_d \cdot h)$$

Knowing P_d and R_d, the factor of safety against shear can be easily determined as

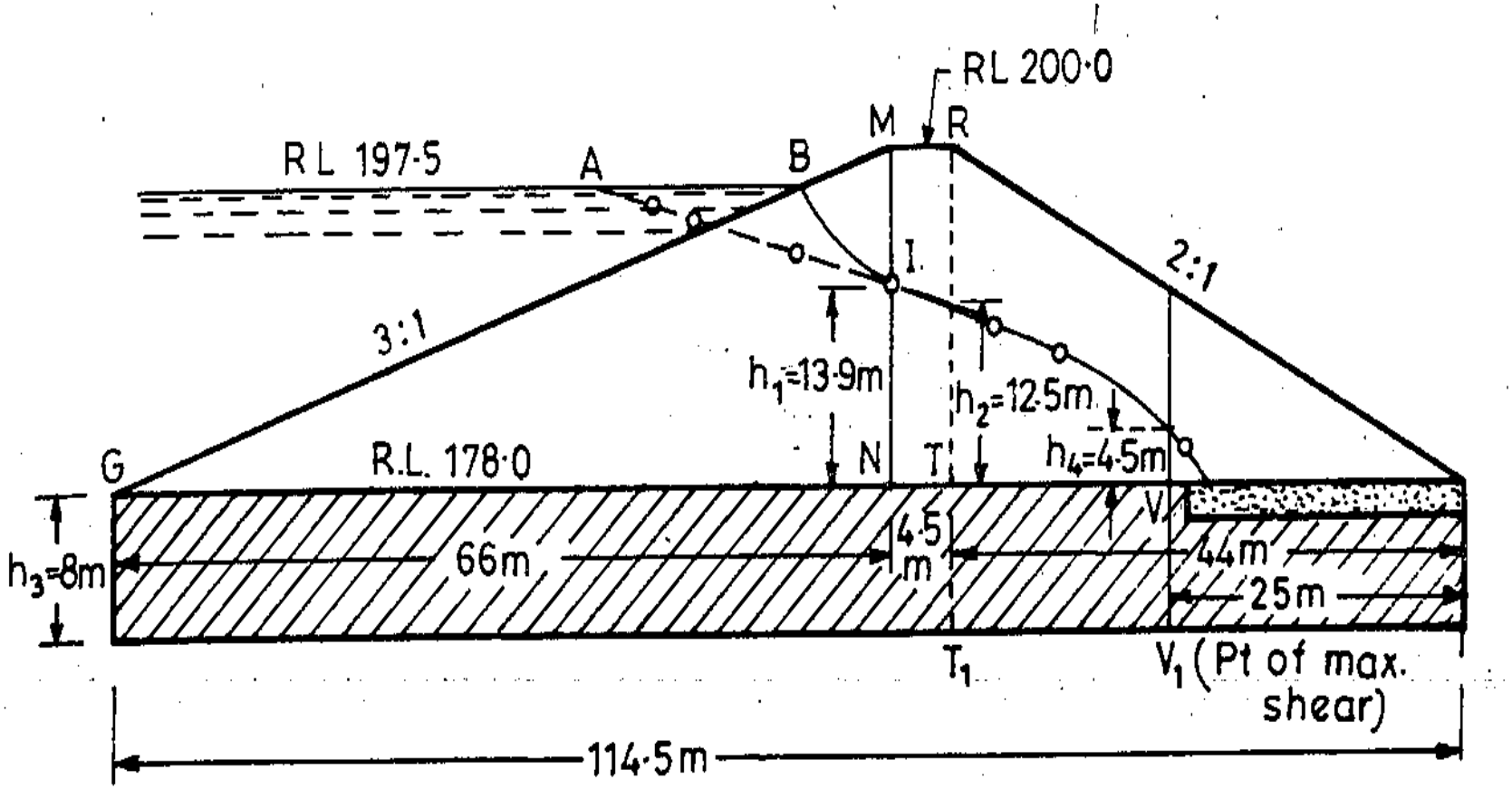
$$\text{F.S.} = \frac{R_d}{P_d} \qquad \ldots(20.38)$$

The factor of safety at the point of maximum shear can also be determined in the same manner as was explained for the u/s slope portion.

Example 20.4. *An earthen dam made of homogeneous material has the following data :*

Level of the top of the dam	$= 200.00$ *m*
Level of deepest river bed	$= 178.0$
H.F.L. of reservoir	$= 197.5$ *m*
Width of top of dam	$= 4.5$ *m*
Upstream slope	$= 3 : 1$
Downstream slope	$= 2 : 1$
Length of the horizontal filter from d/s toe, inwards	$= 25$ *m*
Cohesion of soil of dam	$= 24\ kN/m^2$
Cohesion of soil of foundation	$= 54\ kN/m^2$
Angle of internal friction of soil in the dam	$= 25°$
Angle of internal friction of soil in the foundation	$= 12°$
Dry weight of the soil in the dam	$= 18\ kN/m^3$
Submerged weight of the soil in the dam	$= 12\ kN/m^3$
Dry unit weight of the foundation soil	$= 18.3\ kN/m^3$
Coefficient of permeability of soil in the dam	$= 5 \times 10^{-6}$ *m/sec.*

The foundation soil consists of 8 m thick layer of clay, having negligible coefficient of permeability. Check the stability of the dam and its foundations.


RL 200·0
M
R
RL 197·5
A
B
I
2:1
3:1
h_1=13·9m
h_2=12·5m
h_4=4·5m
G
R.L. 178·0
N
T
V
h_3=8m
66m
4·5 m
44m
25m
T_1
V_1 (Pt of max. shear)
114·5m

1) u/s slope stability

Horizontal force (P_u) acting on the ΔGMN is given by equation (20.27) as :

$$P_u = \left[\frac{\gamma_1 h^2}{2}\tan^2\left(45 - \frac{\phi}{2}\right) + \gamma_w \cdot \frac{h_1^{\,2}}{2}\right]$$

where γ_1 = the weighted density at the centre of triangular shoulder upstream (ΔGMN) and is given by equation (20.28) as :

$$\gamma_1 = \frac{\gamma_{sub} \cdot h_1 + \gamma_{dry}(h - h_1)}{h}$$

$$= \frac{12 \times 13.9 + 18\,(22.0 - 13.9)}{22.0}$$

$$= 14.7\ \text{kN/m}^3$$

$$\therefore \quad P_u = \frac{14.7 \times (22.0)^2}{2}\tan^2\left(45° - \frac{25°}{2}\right) + 9.81 \times \frac{(13.9)^2}{2} = 2391\ \text{kN}$$

Shear resistance R_u of the u/s slope portion of dam developed at the base GN is given by equation (20.29) as :

$$R_u = C + W \tan \phi$$

$$= c\,(B_u \times 1) + (\gamma_{sub} \tfrac{1}{2} B_u\, h) \tan \phi\,;$$ neglecting the small dry soil area BMI, as it is very small and this neglection is on a safer side.

$$B_u = 66 \text{ m}$$

$$\therefore \quad R_u = 24 \times 66 + (12 \cdot \tfrac{1}{2} \cdot 66 \cdot 22.0) \tan 25°$$

$$= 1584 + 4062 = 5646 \text{ kN}$$

Factor of safety against horizontal shear along base under u/s slope

$$= \frac{R_u}{P_u} = \frac{5646}{2391} = 2.36 > 2.0 \qquad (\therefore \text{ safe})$$

Horizontal shear stress induced in the u/s slope portion of dam at base.

$$\tau_{av} = \frac{P_u}{B_u \times 1} = \frac{2391}{66}\ \text{kN/m}^2 = 36.23\ \text{kN/m}^2$$

$$\tau_{max} = \text{Maximum shear}$$

$$= 1.4\,\tau_{av} = 1.4 \times 36.23 = 50.72\ \text{kN/m}^2$$

The maximum shear is developed at a point $0.6\,B_u$

$$= 0.6 \times 66 = 39.6\ \text{m away from point } G$$

The unit shear resistance developed at this point

$$\tau_f = c + 0.6\,\gamma_{sub} \tan\phi$$

$$= 24 + 0.6 \times 22.0 \times 12 \tan 25° = 97.9\ \text{kN/m}^2$$

$$\therefore \quad \text{F.S.} = \frac{\tau_f}{\tau_{max}} = \frac{97.9}{50.72} = 1.93 > 1 \qquad (\therefore \text{ safe})$$

2) D/s slope stability

Horizontal force P_d acting on the portion of downstream dam (RTS) during steady seepage is given by equation (20.35) as :

$$P_d = \left[\frac{\gamma_2 h^2}{2} \tan^2\left(45° - \frac{\phi}{2}\right) + \gamma_w \frac{h_2^2}{2}\right]$$

where γ_2 is the weighted density at the centre of the triangular shoulder RTS and given by equation (20.36) as :

$$\gamma_2 = \frac{\gamma_{sub}\, h_2 + \gamma_{dry}\,(h - h_2)}{h}$$

$$= \frac{12 \times 12.5 + 18 \times (22.0 - 12.5)}{22.0}$$

$$= 14.6 \text{ kN/m}^3$$

$$P_d = \frac{14.6\,(22)^2}{2} \tan^2\left(45° - \frac{25°}{2}\right) + 9.81 \cdot \frac{(12.5)^2}{2} = 2200 \text{ kN}$$

Shear resistance R_d of the d/s slope portion of dam developed at base is given as :

$$R_d = C + W \tan \phi$$

$$W = \gamma_{sub} \cdot (\tfrac{1}{2} \cdot B_d \cdot h)$$

$$R_d = cB_d + \left[\gamma_{dry} A_1 + \gamma_{sub} A_2\right] \tan \phi$$

$$= 24 \times 44.0 + [18 \times 300 + 12 \times 184] \tan 25° = 4604 \text{ kN.}$$

F.S. against horizontal shear along base under d/s slope

$$= \frac{R_d}{P_d} = \frac{4604}{2200} = 2.09 > 2 \ (\therefore \text{ Safe})$$

Average shear induced at base

$$= \frac{P_d}{B_d} = \frac{2200}{44} = 50 \text{ kN/m}^2$$

Maximum shear stress induced

$$\tau_{max} = 1.4 \times 50 = 70 \text{ kN/m}^2$$

The maximum shear stress is developed at a point $0.6\ B_d$

$$= 0.6 \times 44 = 26.4 \text{ m away from toe}$$

This unit shear resistance developed at this point

$$\tau_f = c + 0.6h\,\gamma_{sub}\tan\phi$$

(assuming the entire height as submerged as it will give safer results)

$$= 24 + 0.6 \times 22 \times 12 \tan 25° = 97.9\ \text{kN/m}^2$$

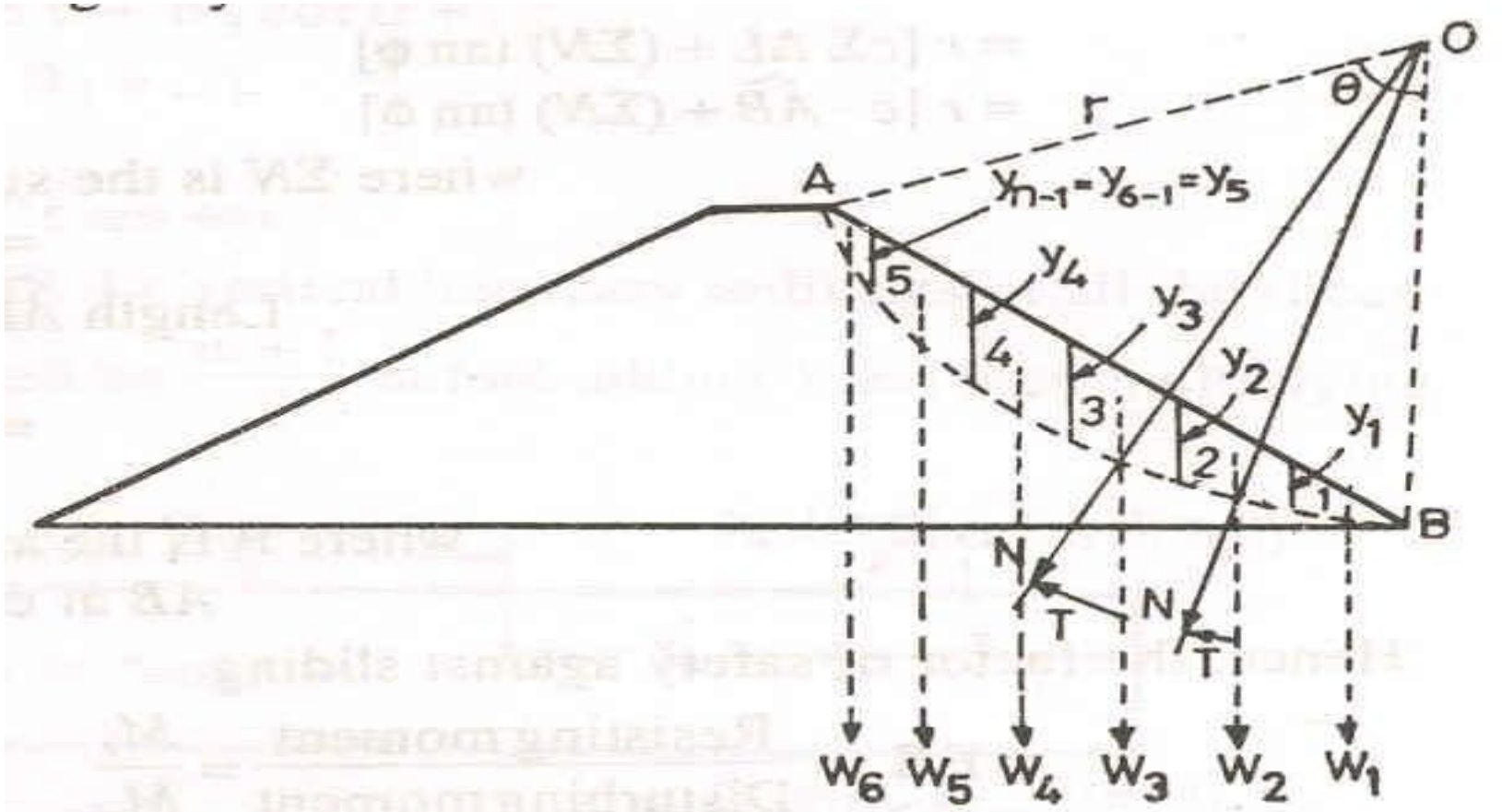
$$\therefore \quad \text{F.S.} = \frac{\tau_f}{\tau_{max}} \frac{97.9}{70} = 1.40 > 1 \qquad (\therefore \text{ Safe})$$

2) Swedish slip circle method:

- assumes that u/s and d/s earthen slopes will be slide along a cylindrical arc.

- the earth mass subtended by the arc is termed as slip circle and divided in to a number of vertical segments called slices.

- O is the center and r is the radius of the possible failure.


O
θ
r
A
$y_{n-1}=y_{6-1}=y_5$
y_4
y_3
y_2
y_1
5
4
3
2
1
B
N
T
N
T
W_6
W_5
W_4
W_3
W_2
W_1

- The weight of each slices are acted at the center and calculated by unit weight of soil * volume of slice.
- W is resolved in to normal and tangential component
- The normal component (N) = W*cos(a) is resisting force.
- The tangential component (T) =W sin(a) is disturbing force.

- The total disturbing forces will be summation of disturbing forces for all slices;

$$T = T1+T2+T3+\ldots\ldots\ldots+Tn$$

- The total disturbing moments over the sliding surface will be equal to:

$$Md = r*\sum Ti$$

- The magnitude of shear strength developed in each slice will depend upon the normal components of that slice.

Its magnitude will be:

$$= c\Delta L + N* \tan\emptyset$$

Where; c is the unit cohesion of the soil

ΔL is curved length of the slice

Φ is the angle of internal friction

N is equal to Wcosα

The total resisting force is given by:

$= \sum c^* \Delta L + \sum N^* \tan\emptyset$

$= c \sum \Delta L + \sum N^* \tan\emptyset$

The resulting resisting moment is:

$Mr = r^*(c \sum \Delta L + \sum N^* \tan\emptyset)$

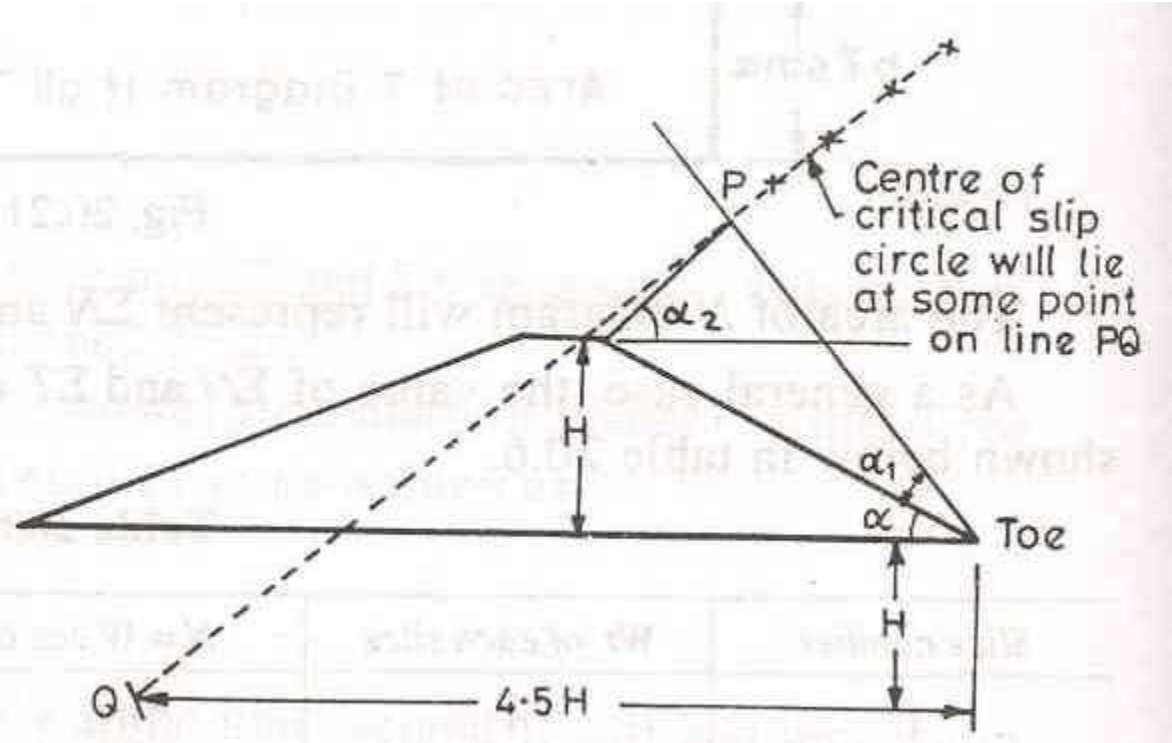
Hence the factor of safety against sliding:

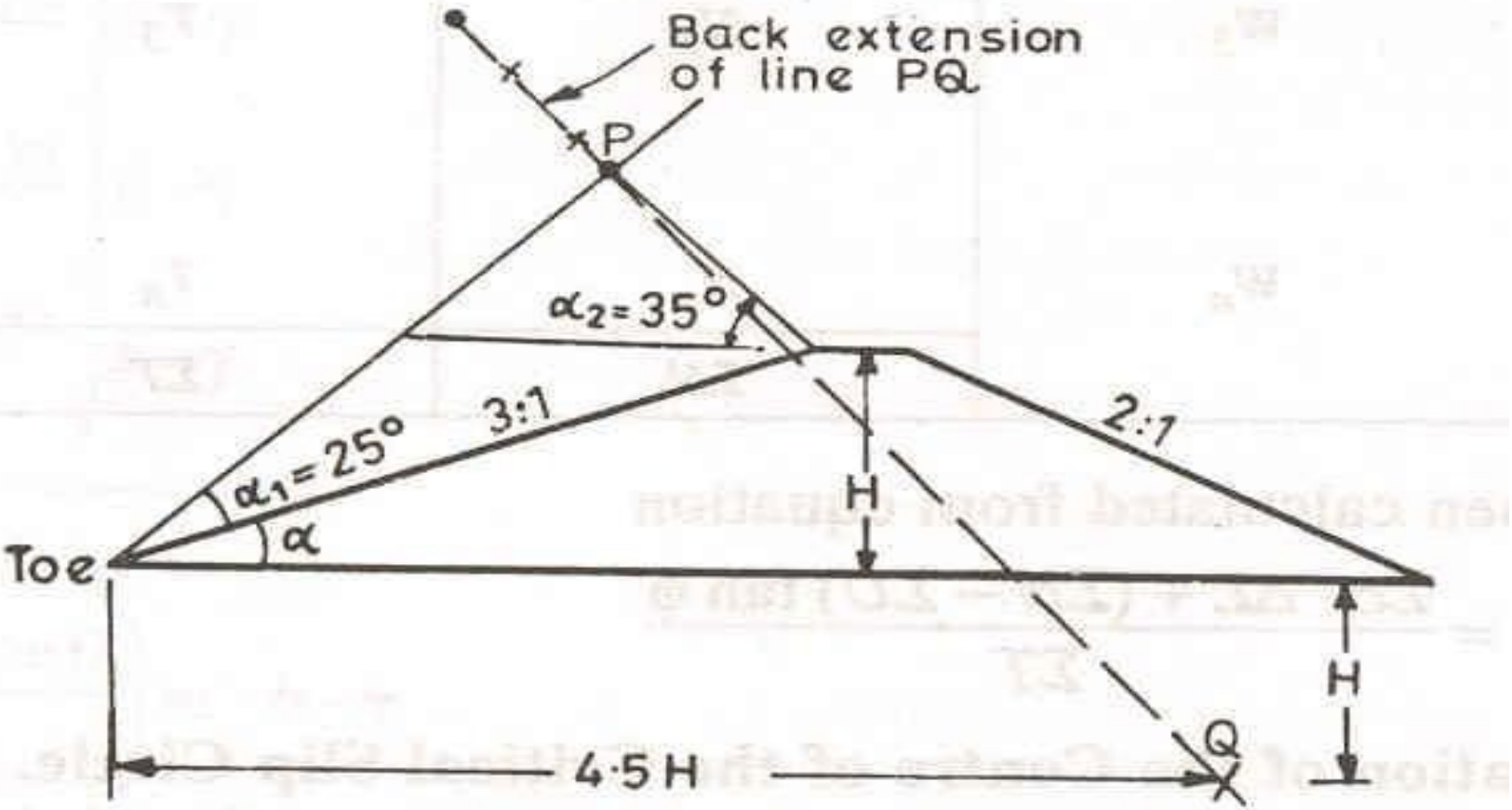
$$Fs = Mr/Md$$

$$= (c \sum \Delta L + \sum N * \tan \emptyset)/\sum T$$

Location of the center of circle

- Fellenius has suggested a method of drawing a line representing the locus of the critical slip circle.
- The determination of this line PQ is obtained with the help of directional angles α1 and α2


P
Centre of critical slip circle will lie at some point on line PQ
α_2
H
α_1
α
Toe
H
Q
4·5 H


Back extension
of line PQ
P
$\alpha_2 = 35°$
3:1
2:1
$\alpha_1 = 25°$
α
Toe
H
H
Q
4·5 H

Slope	Directional angles	
	α_1 *in degrees*	α_2 *in degrees*
1 : 1	27.5	37
2 : 1	25	35
3 : 1	25	35
4 : 1	25	35
5 : 1	25	35

Chapter Four- River Diversion

- - A technique of diverting or changing the direction of water a way from the construction site in order to make the construction site dry
- - this process has been taking place for two purpose
- 1) for construction purpose
- 2) for water use purpose

Continued…..

River diversion for construction purpose:

- during construction of concrete/earthen dam, the dam materials can not be placed on the construction site unless the site is free from water (dry).

- so the river water has to be diverted away from the site by any means.

- the above process can be accomplished by providing the following temporary structures

Continued……

- 1) coffer dam:

  - a low head dam usually built of earthen materials (for economic purpose)

  - has to be provided at u/s and d/s side of the main dam

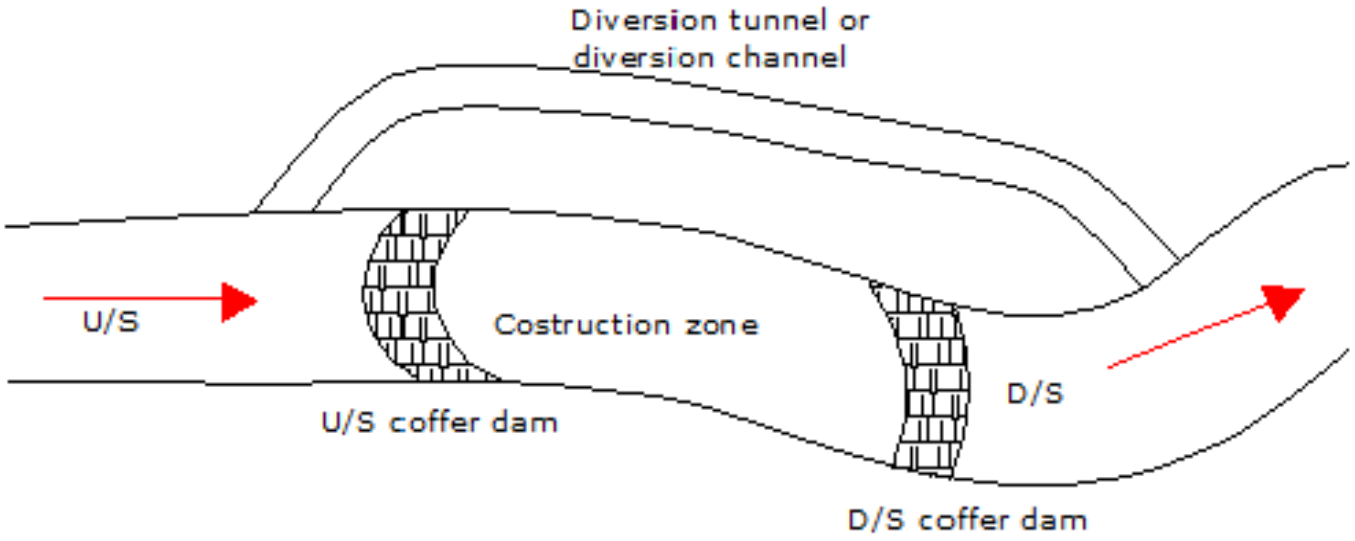
  2) Canal/Channel and Diversion Tunnel:

  - open channel or closed conduit that joins the u/s and d/s

Method of river diversion

- The diversion process can be accomplished in either of the following ways:
- *a) provision of diversion tunnel:*
- - this method is the most commonly applied method of diversion
- - Accomplished by providing coffer dam and tunnel

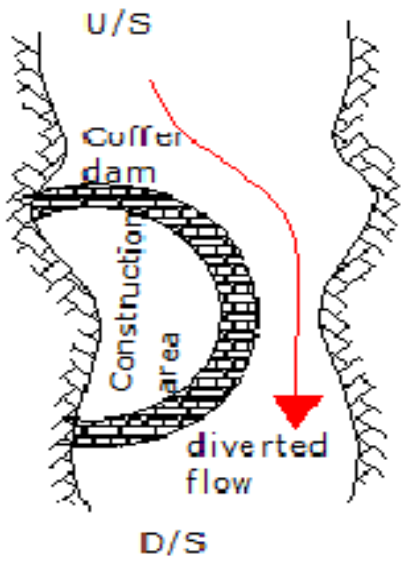
Continued.....


Diversion tunnel or
diversion channel
U/S
Costruction zone
D/S
U/S coffer dam
D/S coffer dam

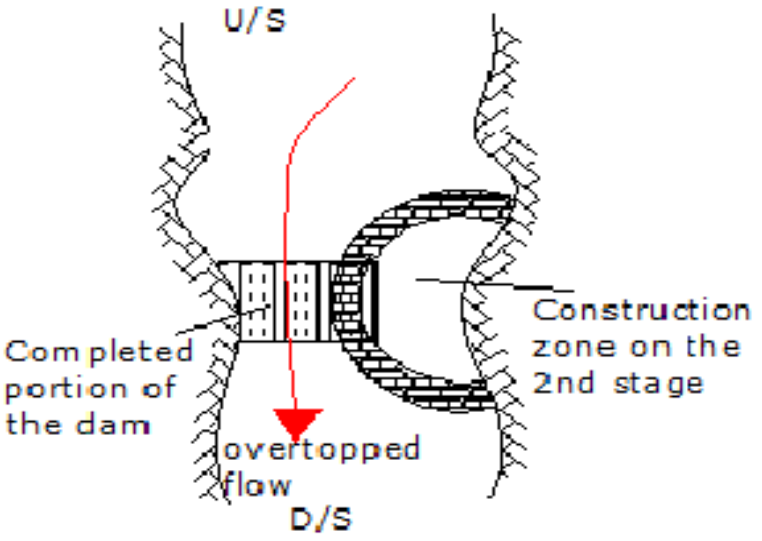
Continued……..

- B) two stage construction:
- - the diversion process has been taking place by provide a semi circular coffer dam to confine one side of the channel.
- - sometimes economical
- - on the next stage the diverted water is allowed to overtop the main dam or to pass through the dam outlet which is already provided in body of dam
- - suitable for only concrete dam

Continued....



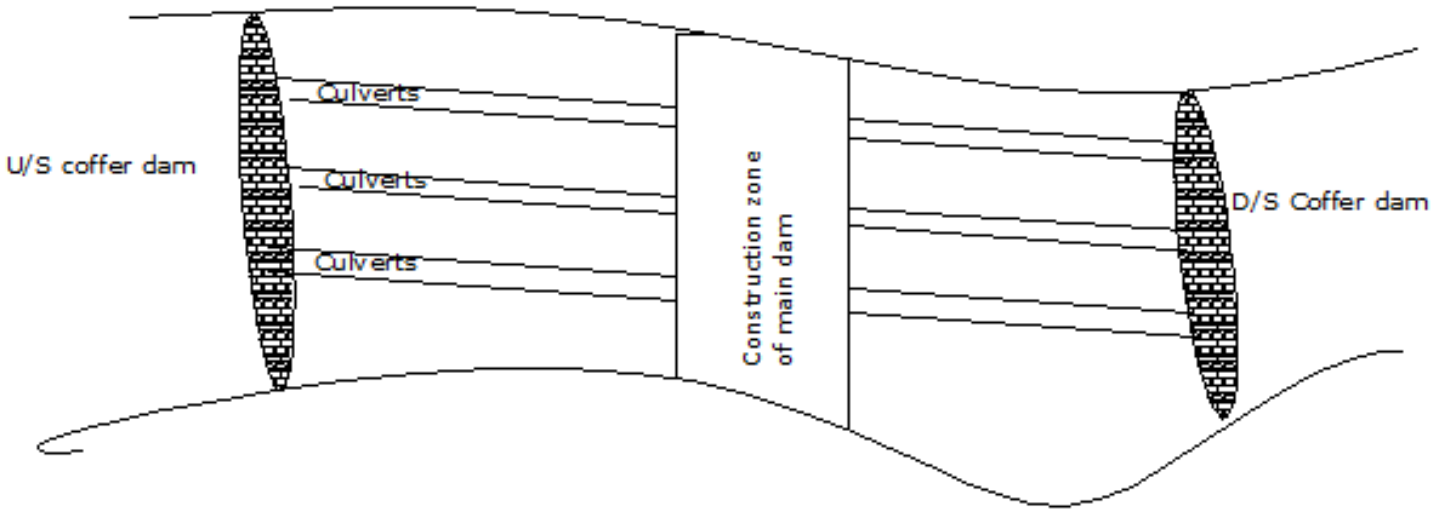
i. 1st stage diversion



ii.2nd stage diversion

Continued……

- C) River diversion through the construction site:
- - culverts are provided to join the u/s and d/s coffer dam through which water passes to d/s


U/S coffer dam
Culverts
Culverts
Culverts
Construction zone
of main dam
D/S Coffer dam

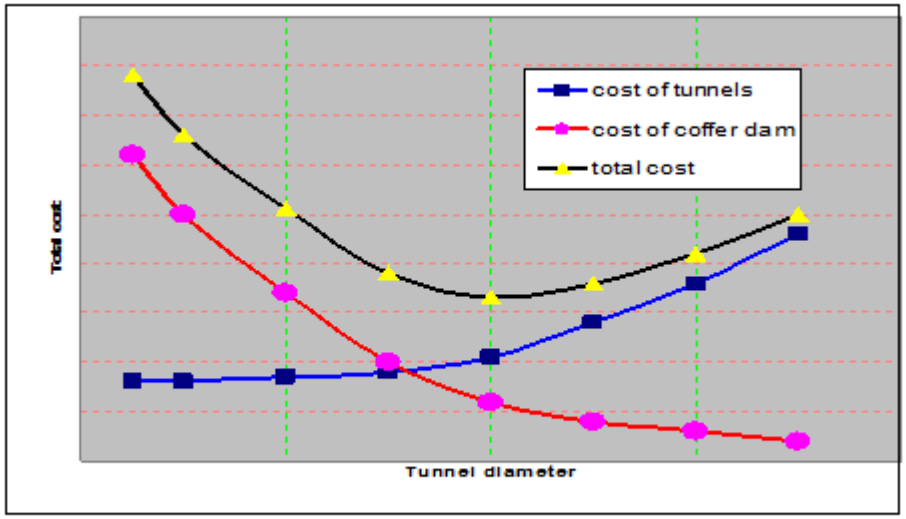
Choice of design flood

- - flood occurs when the flow depth exceeds the limit depth(hl) and the corresponding limit discharge (Ql)
- - the probability of flooding for a return period of T with in 1 year of construction period is given by :
- P = 1/T
- - the probability of non flooding for a return period of T with in 1 year of construction period is given by:
- q = 1-P.....> q = 1-1/T

Continued……

- - for m Years construction period:-
- $q = (1-1/T)^m$
- The probability of flooding for m years of construction period will be:
- $P = 1-q$
- $P = 1- (1-1/T)^m$ is called Hydrologic risk of the construction site

Economic combination of coffer dam height and tunnel Diameter



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- THANK U !!!!!