Embankment Dam

✓ The different types of embankment dams
✓ Causes of failure of embankment dams
✓ Design procedure for earthen embankment dams
✓ Seepage control measures for embankment dams and their foundations
✓ Computation of seepage through embankment dams
✓ Stability calculations for embankment dams
Embankment Dam

✓ *Earthfill embankments:* An embankment may be categorized as an earth fill dam if *compacted soils account for over 50%* of the placed volume of material.

✓ *Rock fill embankments:* The designation ‘rock fill embankment’ is appropriate where *over 50% of the fill material may be classified as rockfill*.

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**Table: Earthfills vs. Rockfills**

<table>
<thead>
<tr>
<th>Earthfills</th>
<th>Rockfills</th>
</tr>
</thead>
<tbody>
<tr>
<td>clays/silts</td>
<td>nature</td>
</tr>
<tr>
<td>cohesive-frictional: low - medium</td>
<td>particle range</td>
</tr>
<tr>
<td>very low</td>
<td>strength</td>
</tr>
<tr>
<td>&lt; 0.1 mm</td>
<td>permeability</td>
</tr>
<tr>
<td>&lt; 2 mm</td>
<td>gravels and coarse rocks</td>
</tr>
<tr>
<td>frictional: medium</td>
<td>crushed rock</td>
</tr>
<tr>
<td>medium</td>
<td>2 – 600 mm</td>
</tr>
</tbody>
</table>

- **Homogeneous earthfill**
- **Zoned earthfill: central core**
- **Zoned earthfill with rockfill**
- **Zoned rockfill: central core**
- **Rockfill with membrane**
Principal variants of earth fill

(a) Homogenous with toe drain: small secondary dams
   \( m = 2.0 - 2.5 \)

(b) Modern homogenous with internal chimney drain
   \( m = 2.5 - 3.5 \)

(c) Slender central clay core: 19th-century ‘Pennines’ type – obsolete post 1950
   \( m = 2.5 - 3.5 \)

(d) Central concrete core: smaller dams – obsolescent
   \( m = 2.5 - 3.5 \)

(e) Wide rolled clay core: zoned with transitions and drains: note base drain
   \( m = 2.5 - 3.5 \)

(f) Earthfill/rockfill with central rolled clay core: zoned with transitions and drains
   \( m = 1.6 - 2.0 \)

Principal variants of rockfill embankment dams

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

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

(c) Decked: upstream asphaltic or concrete membrane
   \( m = 1.6 - 2.0 \)

(d) Central asphaltic membrane
   \( m = 1.6 - 2.0 \)
Key Words: rockfill, transition ............... pervious zone, to have structural strength
core, facing ....................... impervious zone, to keep water tight
filter .................................. to prevent loss of soil particles
drain ................................. to pass water from upstream to downstream
core trench, grouting ......... (to dissipate pore water pressure)

(a) Homogeneous Earth Dam

(b) Rockfill Dam with a Centrally Located Core

(c) Rockfill Dam with an Inclined Core

(d) Rockfill Dam with a Facing
In principle, larger embankment dams required two component elements.

- An *impervious water-retaining element* or *core of very low permeability of soil*, for example, soft clay or a heavily remoulded ‘puddle’ clay—*Core fill*

- Supporting shoulders of coarser *earth fill (or of rock fill)*, to provide *structural stability*—*Shell fill*

- *Filters /Drains*
• As a further enhancement to the design, the shoulders were frequently subject to a degree of simple zoning, with finer more cohesive soils placed adjacent to the core element and coarser fill material towards either face.

• Compacted fine grained silty or clayey earthfills, or in some instances manufactured materials, like asphalt or concrete, are employed for the impervious core element.
Requirements for core, shoulder and drainage blankets, filters

- **Core fill**: should have *low permeability* and ideally be of *intermediate to high plasticity* to accommodate a limited degree of deformation without risk of cracking.

- **Shoulder fill**: requires sufficiently high shear strength to permit the economic construction of stable slopes of the steepest possible slope angle.

  ➢ It is preferable that the shoulder fill *has relatively high permeability* to assist in *dissipating pore water pressures*.

  ➢ The shoulder need not be homogeneous.

- **Drain/filter**: material must be clean, free draining and not liable to chemical degradation.
### Indicative engineering properties for compacted earth fills

<table>
<thead>
<tr>
<th>Fill type (BS 5930)</th>
<th>Compaction characteristics</th>
<th>Shear strength (effective stress)</th>
<th>Coefficient of compressibility, ( \gamma_{c} \times 10^{-4} \text{ kN} \text{m}^{-2} )</th>
<th>Coefficient of horizontal permeability, ( k_h \text{ (m/s)} )</th>
<th>Drainage characteristics (relief of ( u_w ))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravels (GW–GC)</td>
<td>Unit weight, ( \gamma_{d} \text{ (kN/m}^3 )</td>
<td>18–22</td>
<td>5–10</td>
<td>0</td>
<td>35–40</td>
</tr>
<tr>
<td>Sands (SW–SP)</td>
<td>Water ( w_{opt} ) (%)</td>
<td>16–20</td>
<td>10–20</td>
<td>0</td>
<td>35–40</td>
</tr>
<tr>
<td>Silts (ML–MH)</td>
<td>Cohesion, ( c' ) (kN/m²)</td>
<td>16–20</td>
<td>15–30</td>
<td>&lt;10</td>
<td>25–35</td>
</tr>
<tr>
<td>Clays (CL–CH)</td>
<td>Friction, ( \phi' ) (degrees)</td>
<td>16–21</td>
<td>15–30</td>
<td>&lt;20</td>
<td>20–30</td>
</tr>
<tr>
<td>Crushed rockfill (2–600mm size range)</td>
<td>N/A</td>
<td>17–21</td>
<td>N/A</td>
<td>0</td>
<td>40–55</td>
</tr>
</tbody>
</table>
Embankment dams may be classified into three main types:

- Homogeneous type
- Zoned type
- Diaphragm type
A purely homogeneous type of dam is composed entirely of a single type of material.

Fully homogeneous section might be found convenient where the slopes are required to be flat because of a weak foundation.
Zoned type

- A central impervious **core is flanked by shells** of materials considerably more pervious
- The shells enclose and protect the core
- The **upstream shell affords stability** against rapid drawdown and the **downstream shell acts as a drain** that controls the line of seepage.
Contd

- Diaphragm type

- This type the entire dam is *composed of pervious material* i.e. sand, gravel or rock and *thin diaphragm of impervious materials* is provided to retain water.

- The *position of the impervious diaphragm* may vary to extreme *limits-upstream face* on one side and *central core* on the other end.

- If the *base width of the core is less than the height of the embankment*, the dam is considered as diaphragm type.
Contd

Diaphragm type earth dam

Inclined diaphragm type earth dam
Embankment dam and appurtenant structures-
basic types and typical layouts

• An embankment dam, whether made of earth completely or of rock in-filled with earth core, has a trapezoidal shape with the shoulder slopes decided from the point of stability against the various possible modes of failure.

• In order to check the seepage through the body of the dam, a number of variations are provided:
  - Homogeneous dam with toe drain
  - Homogeneous dam with horizontal blanket
  - Homogeneous dam with chimney drain and horizontal blanket
Zoned dam with central vertical core and toe drain

Zoned dam with central vertical core, chimney filter and horizontal blanket

Zoned dam with inclined core, chimney filter and horizontal blanket
Contd

General Shape of Embankment Dam
Homogeneous dam with toe drain
Homogeneous dam with horizontal blanket
Homogeneous dam with chimney drain and horizontal blanket
Zoned dam with central vertical core and toe drain
Zoned dam with central vertical core, chimney filter and horizontal blanket
Contd

Zoned dam with inclined core, chimney filter and horizontal blanket
For the **rock fill embankment dams**, the following variants are common:

- Central vertical clay core
- Inclined clay core with drains
- Decked with asphalt or concrete membrane on upstream face with drains

All are provided with a *chimney filter* connected to a horizontal blanket.
Rock fill dam with Central vertical clay core, chimney filter and horizontal blanket
Rock fill dam with Inclined clay core, chimney drain and horizontal blanket
Decked rock fill dam with U/S asphaltic or concrete membrane with chimney drain and horizontal blanket. The phreatic line is for small amount of water that leaks through the cracks of the U/S membrane.
Contd

Cross section of Embankment dam and concrete gravity dam spillway
Contd

- Causes of failure of earth dams

- The various modes of failures of earth dams may be grouped under three categories:
  - Hydraulic failures
  - Seepage failures, and
  - Structural failures
Contd

- **Hydraulic failures**

  ✓ This type of failure occurs by the *surface erosion* of the dam by water. This may happen due to the following reasons:

  - **Overtopping of the dam** which might have been caused by a flood that exceeded the design flood for the spillway

    ✓ Overtopping may also be caused insufficient freeboard
Contd

• **Erosion of upstream face** and shoulder by the action of continuous wave action
Contd

• Erosion of downstream slope by rain wash
- Erosion of downstream toe of dam by tail water
• Erosion of downstream toe of dam by tail water
Contd

- Seepage failures

  - Piping through dam and its foundation

    - Piping is the *progressive erosion* which develops *through the dam* or *within its foundation* by the water seeping from upstream to the downstream

    - If forces *resisting erosion* i.e. *cohesion, inter-locking effect, weight of soil particles, action of downstream filter* etc. are less than those’ which tend to cause, the soil particles are washed away causing piping failure
Internal erosion and piping through dam body and foundation
Seepage through Foundation and Hydraulic Fracturing

(a) Seepage Through Pervious Foundation
Uplift pressure acts vertically on downstream side, so that counterweight fill or relief well are recommended to prevent heaving and local slide.

(b) Hydraulic Fracture
Concentration of flow lines at downstream toe leads to the increase in hydraulic gradient. Upward seepage force causes reduction in effective stresses in foundation, and quick sand and piping take place when counterweight loading is not enough.
Contd

• Conduit leakage

  – Caused due to seepage taking place by the surface of a conduit due to cracks enclosed within an embankment dam

  – Cracks that might have developed due to unequal settlement of dam or by overloading from the dam

Seepage by the outer surface of conduit: may lead to progressive piping
• Sloughing of downstream face:

  – Failure due to sloughing takes place where *downstream portion of the dam becomes saturated* either *due to choking of filter toe drain*, or due to the *presence of highly pervious layer* in the body of the dam.
Contd

- Structural failures

- Sliding due to weak foundation

  - Due to the *presence of faults and seams* of weathered rocks, shales, soft clay strata, the foundation may not be able to withstand the pressure of the embankment dam

  - The *lower slope moves outwards* along with a part of the foundation and *the top of the embankment subsides* causing large mud waves to form beyond the toe.
Contd

Instability of upstream or downstream slopes caused by failure of weak foundation.
Contd

• Sliding of upstream face due to sudden drawdown

  – If the *reservoir water is suddenly depleted*, the pore pressure cannot get released, which causes the upstream face of the dam to slump

[Diagram of upstream slope failure due to rapid drawdown]
Contd

Upstream slope failure due to rapid drawdown
Contd

• Sliding of the downstream face due to slopes being too steep

  – Slope being *too high and / or too steep* in relation to the *shear strength of the shoulder material* and causes a sliding failure of the downstream face of the dam.
Contd

- Flow slides due to liquefaction

  - Triggered by a *shock or a movement, as during an earthquake*, some portion of the dam or foundation may destabilize due to the phenomena called liquefaction.

  - When the *effective stress drops to zero*, which means the soil loses all its shear strength, it behaves like a *dense liquid and slides down (liquefaction)*, and the dam slumps.
Contd

- Embankment and foundation settlement
  - Excess settlement of the embankment and/or the foundation causes loss of free board.

Excessive settlement of dam and foundation.
Contd

Damages in Embankment and Foundation

© Damage of embankment
- Sliding (by pore-water pressure, earthquake)
- Deformation (settlement and lateral deflection)
- Leakage
- Hydraulic fracture (quick sand and piping)

© Damage of foundation
- Bearing capacity
- Settlement
- Leakage
- Hydraulic fracture
- Liquefaction
Embankment seepage control

- The conventional types of seepage control and drainage features generally adopted for the embankment dam are:

  a) Impervious core
  b) Inclined/vertical filter with horizontal filter
  c) Network of inner longitudinal drain and cross drains
  d) Horizontal filter
  e) Transition zones/transition filters
  f) Intermediate filters
  g) Rock toe
  h) Toe drain
Section of **homogenous dam** showing seepage control features
Section of **Zoned dam** showing seepage control features

Zones:
1. Central impervious core
2. Transition zone / Transition filter
3. Shell / Rockfill
4. Inclined filter
5. Horizontal filter
6. Rock toe
7. Toe drain
Horizontal intermediate Filter layer at downstream face
Horizontal intermediate Filter layer at Upstream face
Contd

- Details of **rock toe/pitching protection** and **toe drains** are illustrated for various combination of **Tail Water Level (TWL)** and stripped **Ground Level (SGL)**.

  - Rock toe when TWL is higher than SGL
  - Pitching when TWL is higher than SGL
  - Rock toe + pitching when TWL is higher than rock toe
  - Pitching when SGL is above TWL
  - Pitching and lined toe drain
Contd

-Rock toe when TWL is higher than SGL
Contd

-Pitching when TWL is higher than SGL
Contd

-Rock toe + pitching when TWL is higher than rock toe
Contd

-Pitching when SGL is above TWL
Contd

-Pitching and lined toe drain

[Diagram showing the components of a toe drain with labels such as "RIPRAP TURFING", "THICK STONE PITCHING", "THICK GRAVEL/METAL LAYER", "THICK SAND LAYER", "HORIZONTAL FILTER", "TOE DRAIN", "STRIPPED SURFACE", "150 THICK LINING", and "600"

All dimensions in mm]
Contd

- Foundation seepage control

- Seepage flows and pressure within the foundation are controlled by *cutoffs* and *by drainage*.
Contd

(c) Diaphragm cut-off (need not penetrate to impervious horizons)

(d) Upstream blanket (may employ underdrain with relief wells)

Cut-offs and control of under seepage
### Upstream Slope Protection

- Upstream slope shall be protected by riprap, which is a layer of rock fragments, against wave action.

- The riprap or pitching should be underlain with two layer of filters to prevent the water from eroding or washing out of the underlying embankment material.
Pitching of rip rap (a) With berm below MDDL; (b) Without berm below MDDL; (c) Terminating at rock surface; (d) Terminating at stripped ground level

MDDL=Minimum Draw Down Level
Contd

• Downstream slope protection

  – embankment should be protected by **turfing**, that is **growing of grass** on the surface against erosion by rain wash

  – if affected, upon by tail water, should be protected by placing riprap over proper filter layers

Typical downstream face surface drainage arrangement
<table>
<thead>
<tr>
<th>Defect</th>
<th>Characteristics</th>
<th>Causes</th>
<th>Preventive–corrective measures</th>
</tr>
</thead>
<tbody>
<tr>
<td>External</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Overtopping</td>
<td>Flow over dam and possible washout;</td>
<td>Inadequate spillway and/or freeboard</td>
<td>Adequate spillway capacity and initial freeboard, and/or reinforced grass surface to slope</td>
</tr>
<tr>
<td></td>
<td>less cohesive soils most at risk; most</td>
<td>Settlement reducing freeboard; spillway obstructed</td>
<td>Restoration of settlement; crest protection; good maintenance</td>
</tr>
<tr>
<td></td>
<td>serious if localized</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wave erosion</td>
<td>Damage to upstream face and shoulder</td>
<td>Face protection disturbed or damaged</td>
<td>Proper design and maintenance</td>
</tr>
<tr>
<td>Toe erosion</td>
<td>Flood discharge damaging toe</td>
<td>Spillway channel badly designed and/or located</td>
<td>Good hydraulic design; training walls</td>
</tr>
<tr>
<td>Gullying</td>
<td>Local concentrated erosion of downstream face by precipitation</td>
<td>Poor surface drainage</td>
<td>Vegetation, surface reinforcement and/or drainage</td>
</tr>
<tr>
<td>Defect</td>
<td>Characteristics</td>
<td>Causes</td>
<td>Preventive-corrective measures</td>
</tr>
<tr>
<td>--------------------------------</td>
<td>-----------------------------------------------------------</td>
<td>------------------------------------------------</td>
<td>---------------------------------------------------</td>
</tr>
<tr>
<td><strong>Internal seepage</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Loss of water</td>
<td>Increased seepage loss and/or irregularities in phreatic surface; soft spots on slopes or downstream</td>
<td>Pervious dam and/or foundation; cut-off inadequate Internal cracking</td>
<td>Cut-off and core grouting</td>
</tr>
<tr>
<td>Seepage erosion (concealed internal erosion)</td>
<td>Turbid seepage through drainage system</td>
<td>Internal cracking</td>
<td>Careful design; grouting</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Leakage along perimeter of culverts, tunnels, pipework etc.</td>
<td>Internal drainage; filters; careful zoning of fill Detail design; use of collars; grouting</td>
</tr>
<tr>
<td>Defect</td>
<td>Characteristics</td>
<td>Causes</td>
<td>Preventive-corrective measures</td>
</tr>
<tr>
<td>---------------</td>
<td>------------------------------------------------------</td>
<td>------------------------------------------------------------------------</td>
<td>---------------------------------------------------------</td>
</tr>
<tr>
<td>Instability</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Foundation slip</td>
<td>Surface displacement of ground near toe of slope</td>
<td>Soft or weak foundation and/or high porewater pressures</td>
<td>Consolidate soil; drainage; ground improvement</td>
</tr>
<tr>
<td>Face slopes</td>
<td>Change in morphology; bulging and deformation, leading to rotational or translational slip</td>
<td>High porewater pressure; slopes too steep; rapid drawdown on upstream slope</td>
<td>Drainage; flatten slope or construct stabilizing berms</td>
</tr>
<tr>
<td>Flowslide</td>
<td>Sudden liquefaction, rapid flow mechanism</td>
<td>Triggered by shock or movement; silty soils at risk</td>
<td>Adequate compaction/ consolidation or toe berm added</td>
</tr>
<tr>
<td>Deformation</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Settlement</td>
<td>Loss of freeboard; local low spots</td>
<td>Deformation and consolidation of dam and/or foundation; result of internal erosion etc.</td>
<td>Restoration of freeboard; good internal detailing to reduce risk of cracking, e.g. protective filters</td>
</tr>
<tr>
<td>Internal</td>
<td>External profile deformation; internal cracking</td>
<td>Relative deformation of zones or materials</td>
<td>Good detailing, with wide transition zones etc.</td>
</tr>
</tbody>
</table>
Seepage Analysis

- Two important seepage calculations are required in embankment dams, which are as follows

  1. Location of the phreatic line
  2. Quantity of seepage discharge

- Quantity of seepage discharge

- Seepage through the embankment can be computed from the flow net, and/or Darcy’s equation.
Contd
• For a *homogeneous earth dam* of properly prepared flow net the seepage discharge can be determined as:

  – If \( h \) is the total hydraulic head and \( N_d \) is the number of potential drops, then the drop in potential head (\( \Delta h \)) per drop is given as:

    \[
    \Delta h = \frac{h}{N_d}
    \]

  – If along the flow path, the length of the side of a flow net box between *one potential and the other* is \( l \), then the hydraulic gradient across the square is \( \Delta h/l \)

  – The discharge passing through two streamlines of the field (\( \Delta q \)) is given as

    \[
    \Delta q = K \cdot \frac{\Delta h}{l} \cdot b
    \]
Contd

\[ \Delta q = K \cdot \frac{\Delta h}{l} \cdot b \]

– where \( K \) is the coefficient of permeability and
– \( b \) is the width of one flow channel, that is, the distance between two stream lines.

- If \( N_f \) is the total number of flow channels, then the seepage per unit width of the embankment (\( q \)) is given as

\[ q = \sum \Delta q = K \cdot \frac{h}{N_d} \left( \frac{b}{l} \right) \cdot N_f = K \cdot h \cdot \left( \frac{b}{l} \right) \cdot \left( \frac{N_f}{N_d} \right) \]

- The equation can be used to find the \textit{water seepage in the dam}. It can also be used to find quantity of seepage in the foundation with \textit{proper flow net construction}.
Contd

- Location and Determination of Phreatic lines

  - The seepage or phreatic line may be defined as the line within a dam section *below* which there are *positive hydrostatic pressures* in the dam
  - the hydrostatic pressure is zero on the phreatic line

- The phreatic line *represents the top flow line* or the boundary condition for drawing the flow net

- the phreatic line for the homogeneous fill section is a *basic parabola* except at the ingress and egress points

- The presence of a pervious stratum below the dam does not influence the position of the phreatic line
Contd

- Stepwise procedure for locating phreatic line
  1. With horizontal drainage filter
Contd
Contd
a) The horizontal distance between **upstream toe A and the point ‘B’** where water surface meets the upstream face is calculated or measured (say L). The point Bo is then located on the water surface at a distance **0.3 L from B**.

b) The basic parabola has to pass **through B0** and have its **focus at F** which is the starting point of the horizontal drainage. With these points known the **basic parabola** may be constructed graphically.

c) With centre Bo and radius BoF,
   a) **Draw an arc to meet the water line at C.**
   b) **Draw the vertical line CD which is the directrix.**
   c) **Let FD, the focal distance = Yo. Bisect the distance FD to get the point E, the vertex of the parabola.**
   d) **Draw FG parallel to CD and equal to yo. Knowing Bo,G and E the basic parabola can be drawn**
• The focal distance $y_o$ can also be determined on the consideration that if $(x \ y)$ is one point on the parabola,

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

Since the point $B_0$ of coordinates $d, h$ lies on the equation,

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

d) The ingress portion of phreatic line is joined to the base parabola from point B, keeping the starting end normal to the upstream face.
2. With inclined discharge face

- For embankments **with no drainage measures** the **base parabola cuts the discharge face at point Go** at a distance \((a + \Delta a)\) along the discharge face from point F, and extends beyond the limits of the embankments.
2. With inclined discharge face (without horizontal filter)

- For embankments with no drainage measures the base parabola cuts the discharge face at point Go at a distance \((a + \Delta a)\) along the discharge face from point F, and extends beyond the limits of the embankments.
-The actual seepage line meets the discharge face (at point G) at a distance \( a \) below the point Go. The value of ‘\( a \)’ can be worked out from the curve after Cassagrande.

<table>
<thead>
<tr>
<th>( \alpha ) in degrees</th>
<th>( \frac{\Delta a}{a + \Delta a} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>30(^\circ)</td>
<td>0.36</td>
</tr>
<tr>
<td>60(^\circ)</td>
<td>0.32</td>
</tr>
<tr>
<td>90(^\circ)</td>
<td>0.26</td>
</tr>
<tr>
<td>120(^\circ)</td>
<td>0.18</td>
</tr>
<tr>
<td>135(^\circ)</td>
<td>0.14</td>
</tr>
<tr>
<td>150(^\circ)</td>
<td>0.10</td>
</tr>
<tr>
<td>180(^\circ)</td>
<td>0.0</td>
</tr>
</tbody>
</table>
• The value of \((a + \Delta a)\) can either be measured directly on the face when the parabola has been drawn or its value determined from the equation,

\[
a + \Delta a = \frac{y_o}{1 - \cos \alpha}
\]

\(\alpha = 180^\circ\) for a horizontal filter and \(\alpha < 90^\circ\) when no drainage is provided
Contd

3. **With rock toe**

- The basic parabola may be drawn in a similar way taking F as focus. As already shown **this parabola it self is the seepage line** for a horizontal filter. (For a horizontal filter $\alpha = 180^\circ$).
• **For a rock toe**, an appropriate value of \( \alpha \), 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.

• If \( \alpha \), less than 30°, the distance ‘a’ or the point of emergence of the phreatic line at the downstream slope may be determined with the help of Schaffernak’s equation.

\[
a = \frac{d}{\cos \alpha} - \sqrt{\frac{d^2}{\cos^2 \alpha} - \frac{h}{\sin^2 \alpha}}
\]

\[
a = \sqrt{d^2 + h^2} - \sqrt{d^2 - h^2 \cot^2 \alpha}
\]

where \( d \) and \( h \) are the coordinates of the initial point B.
• Stability of Slopes
  • Methods of Investigating Stability of Slopes

• For an embankment dam the most important cause of failure is sliding

• Every soil mass which has slope at its end is subjected to shear stresses on internal surfaces in the soil mass, near the slope. This is due to the force of gravity which tries to pull down the portions of the soil mass, adjoining the slope

• If the shearing resistance of the soil is greater than the shearing stress induced along the most severely stressed or critical internal surface, the slope will remain stable
• Failure Of slope

(a) Toe failure

(b) Slope failure

(c) Base failure

Toe failure, in which the failure occurs along the surface that passes through the toe

Slope failure, in which the failure occurs along a surface that intersects the slope above the toe

Base failure, in which the failure surface passes below the toe
Investigating stability of slope Using Swedish method

- In this method, the *curved slip surface* is taken to be an arc of circle with a certain centre.

- There will be a number of such likely slip circles.

- It is necessary to pick up the *most dangerous of critical slip circle* i.e. that circle along which the soil has the least shear resistance.

- The centre of this circle is located by trial and error.
• Let AB be a circular surface with radius r and center O. The *trial failure wedge* above the slip surface is *divided into vertical slices* by drawing vertical lines. The slices are usually of equal width, but not necessarily.

• In the case of *non-homogeneous slopes* where the slip surface passes through more than one type of material, a *vertical line is always located at the point* where the slip surface passes from one material to the other.

• Considering the equilibrium of one slice (no 4). The slice is in equilibrium under the following forces.
Contd
(1) Weight \((W)\) acting vertically through its centre of gravity.

(2) Cohesive force \((C)\) acting along the curved surface in the direction opposite to the direction of probable movement of the wedge.

(3) Reaction \((R)\) at the base inclined at angle \(\phi\) to the normal, assuming the slippage is imminent.

(4) Reactions on the two vertical sides of the slice due to adjacent slices. However, in the Swedish circle method, it is assumed that the reactions on the two sides are equal and opposite and are, therefore, in equilibrium and do not affect the stability of the slice. Accordingly, only the first three forces are considered for the analysis.
The weight $W$ is resolved into its *normal component* ($N$) and the tangential component ($T$).

- Taking the moments about the center of rotation $O$, for all the forces:

- Actuating or overturning moment, $M_o = T \cdot r$

- The moment due to $N$-Components is zero, as these components always pass through $O$

- Resisting Moment, $M_r = (C \cdot \Delta L) \cdot r + R(r \sin \varphi)$
  - Where $\Delta L$ is the length of the curved surface of the slice
Contd

\[ N = R \cos \phi \text{ or } R = \frac{N}{\cos \phi} \]

Or \[ R \sin \phi = N \tan \phi \]

\[ Mr = (C \Delta L) r + N r \tan \phi \]

- The factor of safety for the slice is equal to the ratio of the resisting Moment (Mr) and overturning moment (Mo)

\[
F_s = \frac{r(C \Delta L + N \tan \phi)}{T \times r} = \frac{C \Delta L + N \tan \phi}{T}
\]

\[
F_s = \frac{\sum C \Delta L + \sum N \tan \phi}{\sum T} \quad \text{(For the entire wedge)}
\]
Contd

(a) Relatively homogeneous embankment and soil foundation

(b) Embankment on rock or ‘stiff’ foundation

(c) Soft, compressible clay layer in foundation; drawdown-type failure

(d) Wedge-type active mass; sliding in part on soft horizon

Stability analysis: failure surface schematics
Contd

• The most critical condition in stability analysis
  
  – Reservoir drawdown
  – Steady Seepage Condition
  – End of Construction’ Condition
  – Earthquake

• Reservoir Drawdown

• To take account of this fact in stability computations, the resisting forces are calculated for submerged weight of the material below water surface, and the actuating forces are calculated for the saturated weight of the material below water surface.
Contd
Contd

• All materials **below drawdown level are submerged**, and therefore, resisting and actuating force below the drawdown level are calculated on the **basis of the submerged weight** of the materials.

• **Steady Seepage Condition**

• For the analysis of this condition, **reservoir is assumed to be at the normal storage level** in which case the phreatic line is assumed to have fully developed

• The **upstream slope will be subjected to water pressure** from the reservoir while **the rest of dam will be subjected to pore pressure**
Steady Seepage Conditions.
Contd

\[ F = \frac{cL_a + \tan \phi \sum (N - U)}{\sum T} \]

Or

\[ F_s = \frac{\sum c \Delta L + \sum (N - ul) \tan \phi}{\sum T} \]

where \( u \) is the average pore pressure on the slice and \( l \) is the curved length of the base of the slice. (Note. \( l = \Delta L = b \sec \alpha \).)
Pore Pressures in drawdown and steady conditions

- Pore pressures on *any slice of a failure are taken to act normally on the arc surface* and are equivalent to the *weight of water* transmitted by that slice.

Consider a slice *below phreatic line and above dead storage level* or tail water level. If *h is the average height* and *d its width*, force due to pore pressure.

\[
\begin{align*}
    u &= h \cdot w \times \text{arc length} \\
    &= h \times w \times d \times \sec \theta.
\end{align*}
\]
The resisting force due to pore pressure will be reduced to $u \tan \phi$.

\[ = (h \text{d.w. sec } \theta) \tan \phi \]

\[ = (h \times \text{arc length}) \times w \tan \phi \]

If the length for a small slice be taken approximately equal to the width of the slice, the contribution of pore pressure becomes $(h \cdot b \cdot w) \tan \phi$.
Contd

• ‘End of Construction’ Condition

• Stability at the end of construction is most critical for homogeneous embankments constructed of plastic materials.

• on completion of embankment there would be construction pore pressure due to consolidation of fill under the embankment load.
• ‘Earth quack

  – Practice to account the horizontal acceleration caused by earthquake

  – It is recommended that applicable values of seismic coefficient may be taken into account in stability computation for:
    • steady seepage conditions,
    • while half of its values be adopted in case of sudden drawdown and end of construction conditions
### Table 2.7  Guideline factors of safety: effective stress stability analysis

<table>
<thead>
<tr>
<th>Design loading condition</th>
<th>Factor of safety, $F_{\text{min}}$</th>
<th>Downstream slope</th>
<th>Upstream slope</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1) Under construction; end of construction</td>
<td></td>
<td>1.25</td>
<td>1.25</td>
</tr>
<tr>
<td>(2) Long-term operational; reservoir full</td>
<td></td>
<td>1.5</td>
<td>1.5</td>
</tr>
<tr>
<td>(3) Rapid drawdown</td>
<td></td>
<td>–</td>
<td>1.2</td>
</tr>
<tr>
<td>(4) Seismic loading with 1, 2 or 3 above</td>
<td></td>
<td>1.1</td>
<td>1.1</td>
</tr>
</tbody>
</table>
Contd

- **Location of critical slip surface**

  Various circles with center on this line are tried until the one with minimum factor of safety is found.
• The line joining 01 and 02 points is the line on which the centre of the critical circle lies.

**Recommended values of α and β - Fellenius construction**

<table>
<thead>
<tr>
<th>Slope hor: vertical</th>
<th>α</th>
<th>β</th>
</tr>
</thead>
<tbody>
<tr>
<td>1:1</td>
<td>27.5°</td>
<td>37°</td>
</tr>
<tr>
<td>2:1</td>
<td>25°</td>
<td>32°</td>
</tr>
<tr>
<td>3:1</td>
<td>25°</td>
<td>35°</td>
</tr>
<tr>
<td>4:1</td>
<td>25°</td>
<td>36°</td>
</tr>
<tr>
<td>5:1</td>
<td>25°</td>
<td>37°</td>
</tr>
</tbody>
</table>
The filter material used for **drainage system** shall satisfy the following criteria:

- Filter materials shall be more pervious than the base materials.
- Filter materials shall be of such gradation that particles of base material do not totally migrate through to clog the voids in filter material; and
- Filter material should help in formation of natural graded layers in the zone of base soil adjacent to the filter by readjustment of particles.
Contd

- The uniformity of the soil is expressed qualitatively by a term known as uniformity coefficient: $C_u$, expressed as

$$C_u = \frac{D_{60}}{D_{10}}$$

where $D_{60} = \text{particle size such that } 60\% \text{ of the soil is finer than this size}$, and $D_{10} = \text{particle size such that } 10\% \text{ of the soil is finer than this size}$. 
A widely employed empirical approach to defining appropriate filter material grading envelopes is given in the form of ratios for specified particle passing sizes:

\[
\frac{D_{15}(\text{filter})}{D_{85}(\text{soil})} \leq 5, \\
\frac{D_{15}(\text{filter})}{D_{15}(\text{soil})} \geq 5 \\
\text{and} \\
\frac{D_{50}(\text{filter})}{D_{50}(\text{soil})} \leq 25
\]

Set out piping and permeability criteria respectively

Defines permeability ratio