COFFER DAM

A cofferdam is a temporary structure built to enclose an area surrounded by water for excavation of foundations.

It is generally required for foundations of structures such as bridge piers, docks, locks, and dams, which are built in open water.

Cofferdams are also used for laying foundations in open land where there is a high ground water table.

It is a type of watertight construction built around the periphery of the proposed excavation to prevent flow of water into the excavation, so as to facilitate the foundation to be laid in dry condition.

Working inside a cofferdam can be hazardous if it is installed improperly or not safely pressurized to prevent the entry of water

It is also used as a watertight chamber attached to the side of a ship to facilitate repairs below the water line.

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SHEET PILE USED AS COFFERDAM

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SHEET PILE WALL USED AS COFFERDAM

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OTHER USES OF COFFERDAM

The cofferdam is also used on occasion in the shipbuilding and ship repair industry, when it is not practical to put a ship in drydock for repair or alteration.

An example of such an application is certain ship lengthening operations. In some cases a ship is actually cut in two while still in the water, and a new section of ship is floated in to lengthen the ship.

Shipwrights and repair yards also use a form of portable cofferdam, which can be attached to the side of a ship to enact repairs below the waterline. At sea, this can be a useful way to quickly address potential problems until the ship is taken into dry dock for more long-term repair. Minor repairs can be undertaken with a portable cofferdam in a shipyard to avoid the expense of hauling the ship into dry dock for the work to be completed.

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Types of Coffer Dams

The following are different types of coffer dam commonly used in practice

1. Earth Coffer Dams
2. Rockfill Coffer Dams
3. Single-Sheet Pile Coffer Dams
4. Double-Wall Sheet Piling Coffer Dams
5. Braced Coffer Dams
6. Cellular Coffer Dams
   a) Diaphragm type
   b) Circular type
   c) Cloverleaf type

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1. **Earth Coffer Dams:** These are simplest type of coffer dams and are used upto 3 m depth of water. Earth embankments are constructed around the area to be dewatered.

   The earth coffer dams are built of local soils, preferably fine sand. These usually have a clay core or a vertically driven sheet piling in the middle. The upstream slope of the bank is covered with a rip rap.

   A successful coffer dam need not be completely watertight. For reasons of economy, it is not possible to make it watertight and hence some seepage of water into the excavation is usually tolerated. The water collected is pumped out of the excavation. The embankment should be provided with a minimum free board of 1m to prevent overtopping by waves.

   Sand – bag coffer dams are an alternate used in case of emergency.

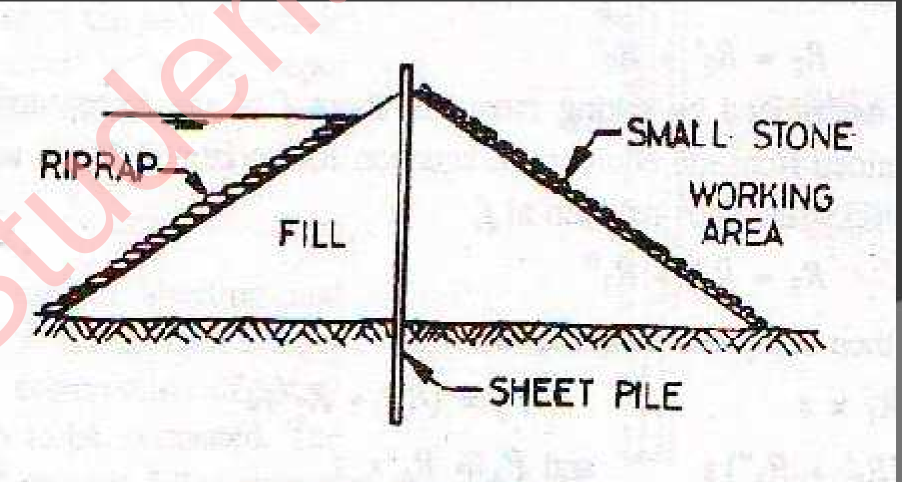

Fig. 21.10. Earth Coffer Dam.

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2. Rockfill Coffer Dams: Rockfill coffer dams made of rockfill are sometimes used to enclose the site to be dewatered. These are permeable and are usually provided with an impervious membrane of soil to reduce seepage. The crest and the upper part of the impervious membrane are provided with rip rap to provide protection against wave action. Overtopping does not cause serious damage in case of rockfill coffer dams. The slopes of a rockfill coffer dams can be made as steep as 1 horizontal to 1.5 vertical.

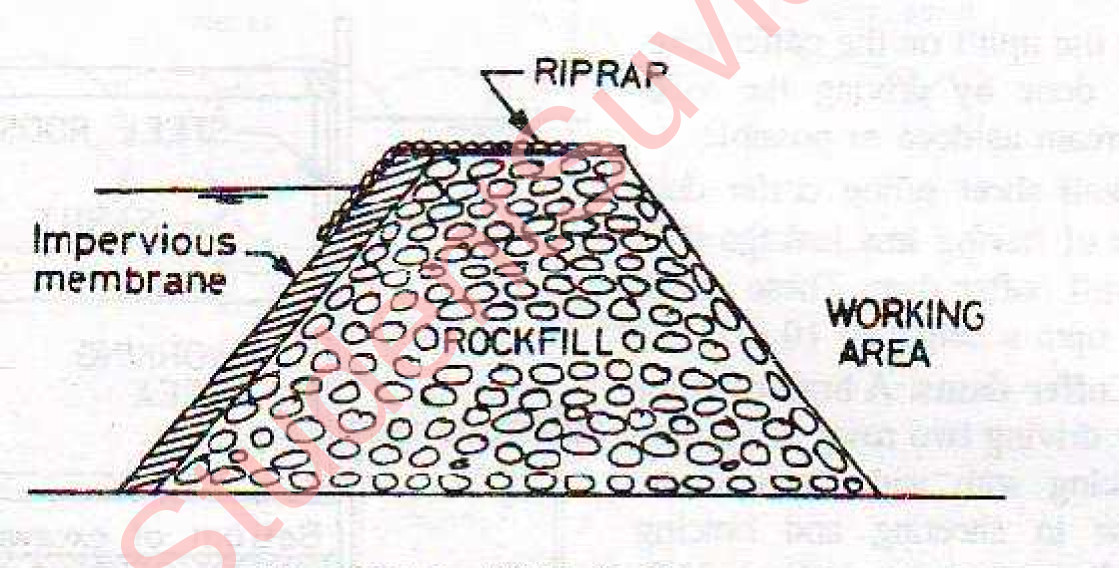

Fig. 21.11. Rockfill Coffer Dam.

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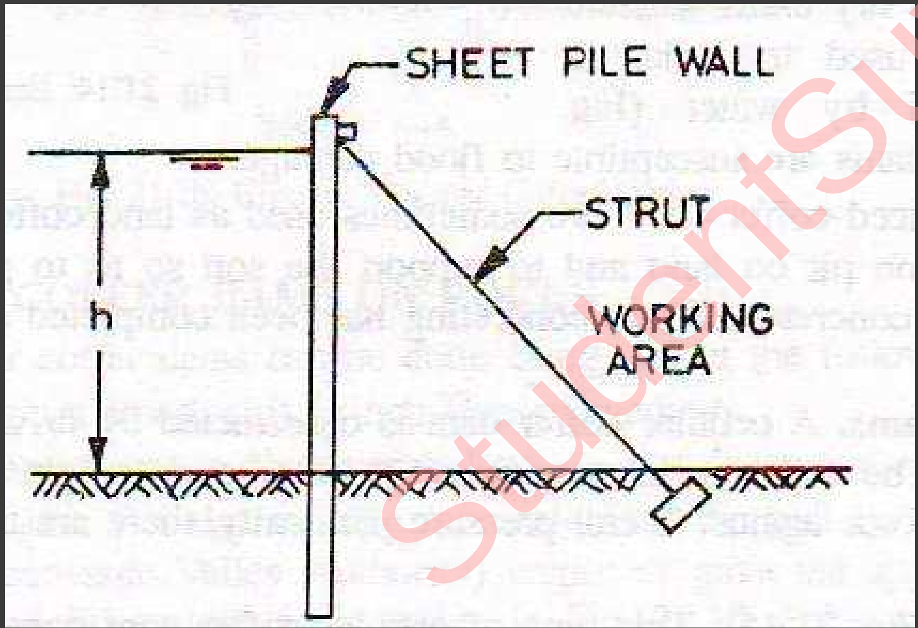
3. **Single-Sheet Pile Coffer Dams:** Single-sheet piling coffer dams are generally used to enclose small foundation sites in water for bridges at a relatively shallow depth. In this type of coffer dams, there is a single row of cantilever sheet piles. The piles are sometimes heavily braced. Joints in the sheet piles are properly sealed. This type of coffer dams are suitable for moderate-flow velocities of water and for depths upto 4m. The depth of penetration below ground surface is about 0.25 h for coarse sand and gravels, 0.50 h for fine sand and 0.85 h for silts, where h is the depth of water. Sometimes, single-sheet pile coffer dams are provided with earth fills on one or both sides to increase the lateral stability.



Single sheet pile cofferdam

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4. Double-Wall Sheet Piling Coffer Dams: A double-wall sheet piling coffer dam consists of two straight, parallel vertical walls of sheet piling, tied to each other and the space between walls filled with soil. The width between the parallel piles is empirically set as (h/2+1.50m), where h is height of water. Double-wall sheet piling coffer dams higher than 2.5m should be strutted. Sometimes, an inside berm is provided to keep the phreatic line within the berm.

The fill material should have a high coefficient of friction and unit weight so that it performs as a massive body to give the coffer dam stability against sliding and overturning. Suitable measures should be adopted to reduce the uplift on the coffer dam. This is generally done by driving the sheet piling on the upstream as deep as possible.

The double-wall sheet piling coffer dam has the advantage of having less leakage than that in a single-wall coffer dam. These coffer dams are suitable upto a height of 10m

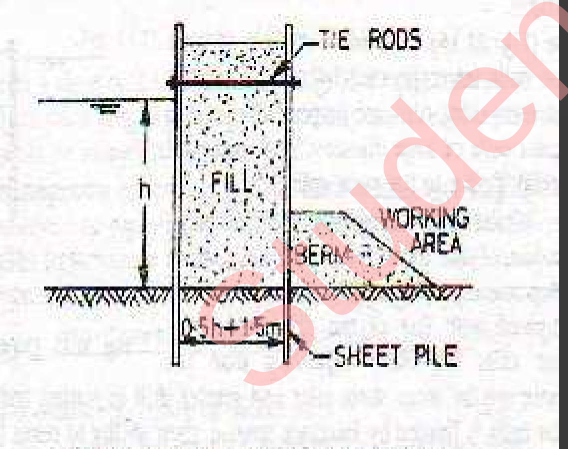

Fig. 21.13. Double-Wall Sheet Piling Coffer Dam.

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5. **Braced cofferdam:** A braced coffer dam is formed by driving two rows of vertical sheeting and bracing with wales and struts. These are similar to sheeting and bracing system discussed for braced cuts in sect.21.3, with one basic difference that braced cuts are required for excavations in dry areas whereas braced coffer dams are used to isolate a working area surrounded by water. The braced coffer dams are susceptible to flood damage.

Land coffer dams: Braced coffer dams are sometimes used as land dams to prevent ground water from entering the foundation pit on land and to support the soil so as to prevent cave in. After the pit is dewatered, the structure is concreted. When concreting has been completed above the water level, the coffer dam is removed.

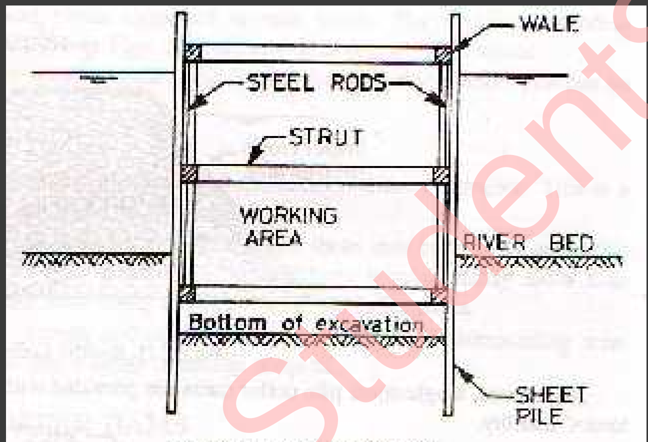

Fig. 21.14. Braced Coffer Dam.

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6. Cellular Coffer Dams: A cellular coffer dam is constructed by driving sheet piles of special shapes to form a series of cells. The cells are interconnected to form a watertight wall. These cells are filled with soil to provide stabilizing force against lateral pressure. Basically, there are two types of cellular coffer dams that are commonly used.

i. Diaphragm type ii. Circular type

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i. Diaphragm type: This type of cellular coffer dam consist of Circular arcs on the inner and outer sides which are connected by straight diaphragm walls. The connection between the curved parts and the diaphragms are made by means of a specially fabricated Y element. The coffer dam is thus made from inter connected steel sheet piles. The cells are filled with coarse grained soils which increase the weight of the coffer dam and its stability. The leakage through the coffer dam is also reduced. To avoid rupture of diaphragms due to unequal pressure on the two sides, it is essential to fill all the cells at approximately the same rate. One advantage of the diaphragm type is that the effective length of the coffer dam may be increased easily by lengthening the diaphragm.

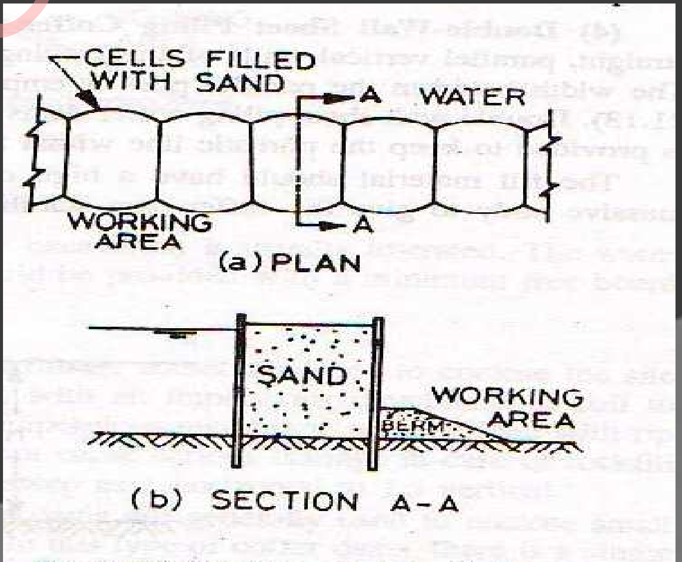

Fig. 21.15. Diaphragm Cellular Coffer Dam.

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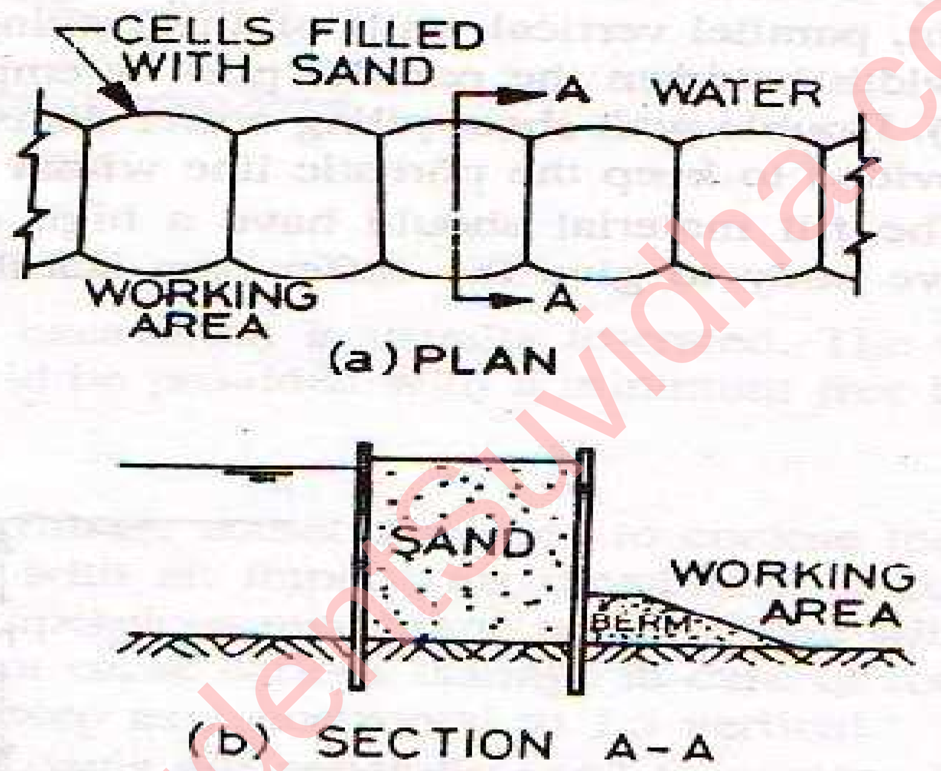

Fig. 21.15. Diaphragm Cellular Coffer Dam.

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ii. Circular type: It consist of a set of large diameter main circular cells interconnected by arcs of smaller cells. The walls of the connecting cells are perpendicular to the walls of the main circular cells of large diameter. The segmental arcs are joined by special T-piles to the main cells.

The circular-type cellular coffer dams are self-sustaining, and therefore independent of the adjacent circular cells. Each cell can be filled independently. The stability of such cells is much greater as compared with that of the diaphragm type. However, the circular cells are more expensive than the diaphragm type, as these require more sheet piles and greater skill in setting and driving the piles. Because the diameter of circular cells is limited by interlock tension, their ability to resist large lateral pressure due to high heads is limited.

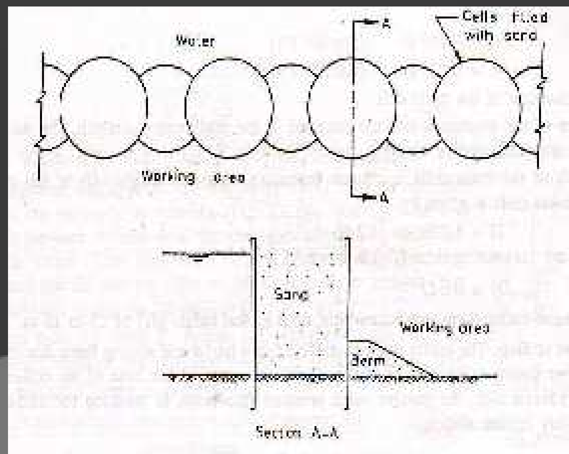

Fig. 21.16. Circular Cellular Coffer Dam.

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Design of Cellular Coffer Dams

1. Dams on rocks
2. Dams on soil

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1. Design of Cellular Coffer Dams on Rocks

- The design of circular, cellular coffer dams can be done using one of the following methods.
  - TVA (Tennessee Valley Authority) method
  - Cumming's method
  - Brinch-Hansen method

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The TVA method is more common and is discussed below. The following design aspects are dealt.

1. Location of Saturation Line
2. Average Width
3. Safety Against Sliding
4. Safety Against Overturning
5. Safety Against Slipping
6. Safety Against vertical shear
7. Stability Against bursting

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1. Location of Saturation Line: For determining the weight of the soil in the cell, it is required to locate the saturation line. TVA (Tennessee Valley Authority) engineers gave the approximate location of the saturation line for different types of fill materials. For a perfectly draining fill, the saturation line is shown in fig. a. The lower half may be assumed saturated for analysis. For other type of fill, the saturation line at a slope of 2:1 is assumed. In case of a berm, the saturation line drops down to the top of the berm. For stability analysis, two extreme locations (marked 1 and 2) of saturation line should be investigated in this case.

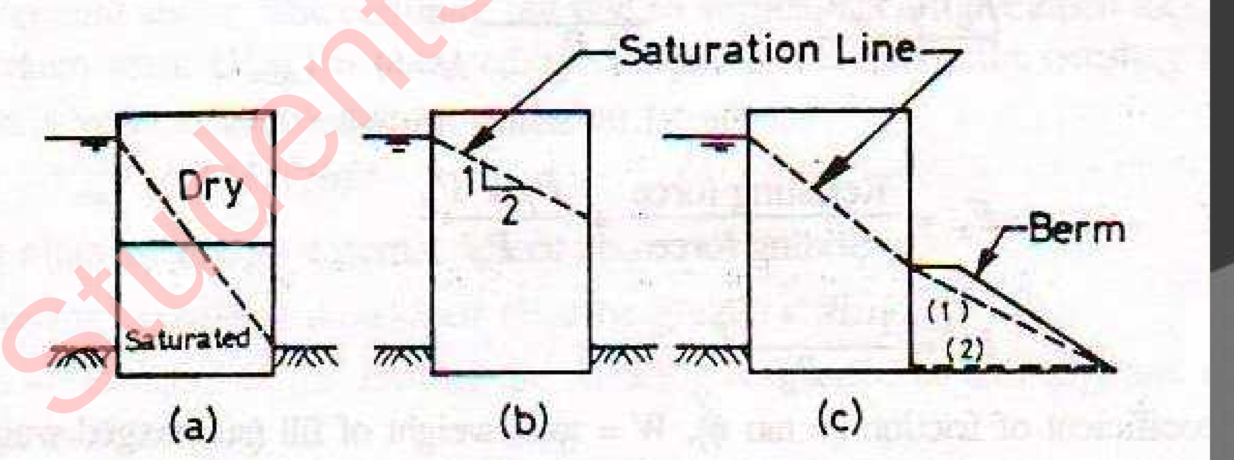

Fig. 21.17. Location of Saturation Line.

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2. Average Width: the design of a coffer dam is made for a section 1m long of uniform, average width. The average width is obtained by equating the section modulus of the equivalent rectangular section to the actual section modulus. An approximate value of the average width may, however, be obtained by equating the equivalent rectangular area to the actual area of the coffer dam between center to center.

Thus

$$\text{Average width (b)} = \frac{\text{area of one main cell} + \text{area of one connecting cell}}{2L}$$

Where 2L = distance between center to center of main cells.

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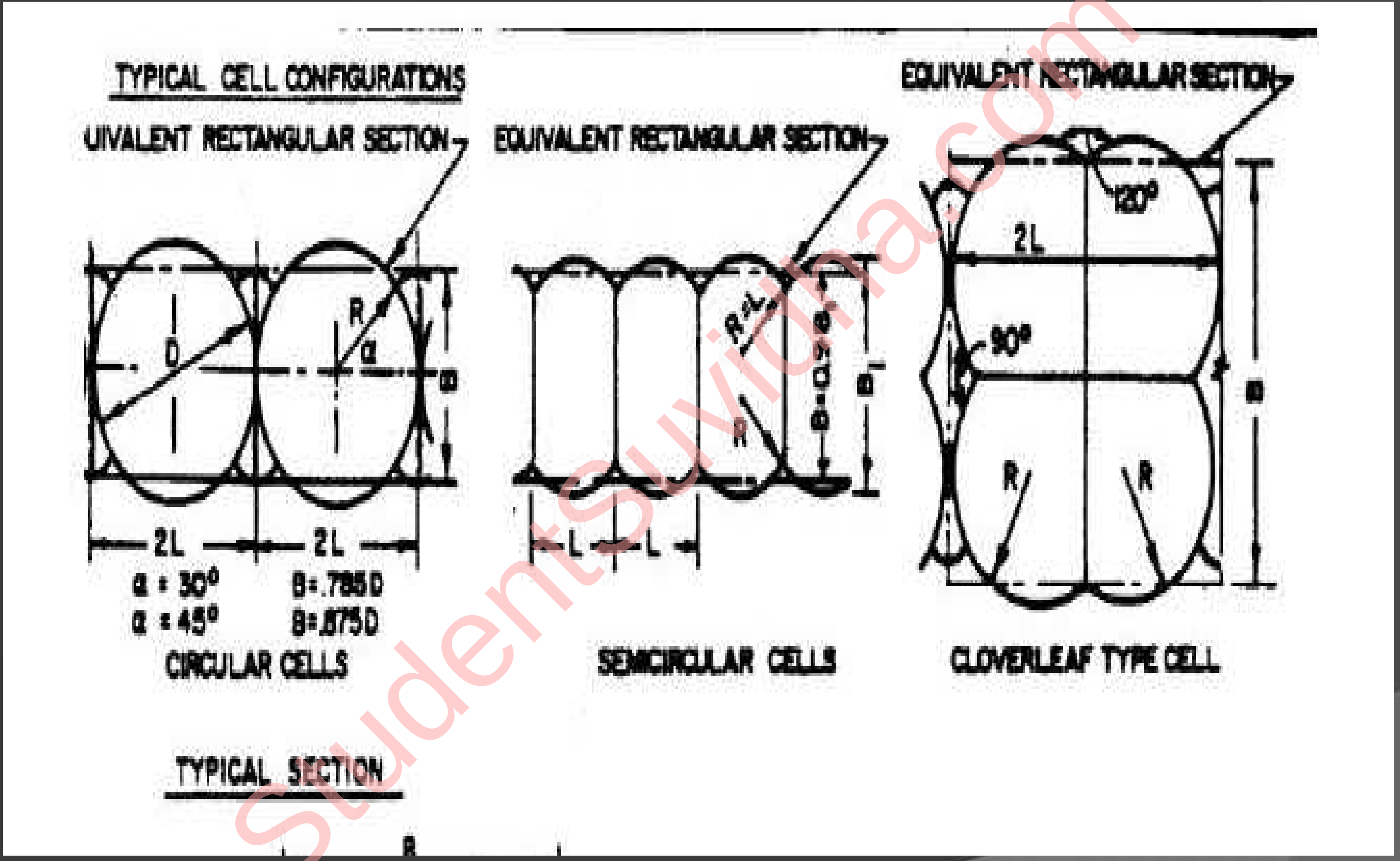

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- The TVA engineers gave the following relation for computing the average width.

  $b = 0.785\ D$ (For 90^{o} T's)

  And $b = 0.875\ D$ (For 60^{o} T's)

  Where D is the diameter of the main cell.
- The above values of average width are assumed in the preliminary analysis. The actual width to be provided is obtained after the stability analysis.
- The diameter (D) of the main cells is chosen depending upon the height (H) of the coffer dam. The diameter (D) of the main cells is given by

  D = 1.0 H to 1.2 H
- The diameter of the connecting cells (D_1) is given by

  $D_1 = 0.6\ D$
- The circular, cellular coffer dams are economical upto a total height (H) of 15 to 18m.

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3. Safety Against Sliding: the coffer dam is subjected to a horizontal sliding force due to water pressure and earth pressure. The sliding is resisted by the frictional resistance at the base of the coffer dam. If berms are provided on the interior side, the passive earth pressure also helps in resisting the sliding. Thus the factor of safety against sliding,

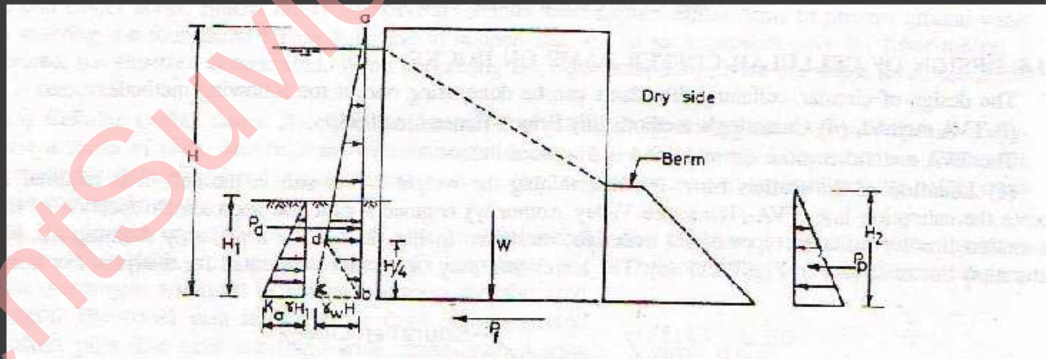

Fig. 21.18. Sliding Analysis.

$$F_s = \frac{\text{Resisting force}}{\text{Sliding force}} = \frac{P_f + P_p}{P_d} \quad \ldots(21.12)$$

or

$$F_s = \frac{\mu W + P_p}{P_d} \quad \ldots(21.13)$$

where μ = coefficient of friction (= tan ϕ), W = total weight of fill (submerged weight below saturation line),
P_p = passive resistance of the berm on dry side.
P_d = driving force due to water and soil on the water side.
P_f = resisting force at the base.

A factor of safety of at least 1.25 is generally recommended.

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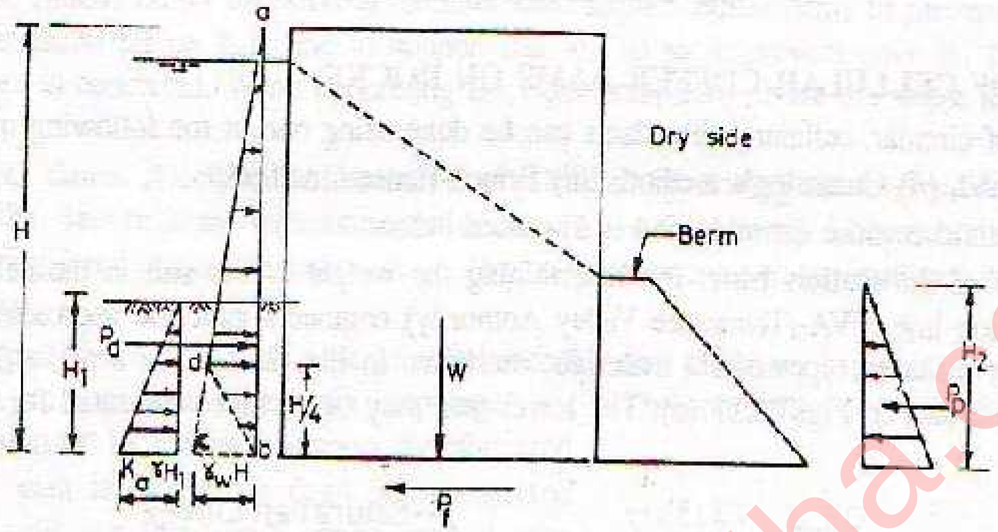

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P_d = driving force due to water and soil on the water side.

P_f = resisting force at the base.

A factor of safety of at least 1.25 is generally recommended.

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4. Safety Against Overturning: The coffer dam should be safe against failure due to overturning at toe. Neglecting the passive resistance of the berm, the factor of safety against overturning is given by

$$F_o = \frac{\text{Resisting moment}}{\text{Overturning moment}}$$

The factor of safety (F_o) should be greater than 2.0.

Further, as the soil cannot resist tension, the resultant of the forces must lie within the middle third. The ecentricity (e) is determined after locating the point where the resultant strikes, as the case of retaining walls. Thus.

$$e \le b/6$$

$$b \ge \sqrt{\frac{6P_d \bar{Z}}{¥H}}$$

Where $\bar{z}$ = height of the line of action of P_d (driving force) above the base.

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Design of Cellular Coffer Dams on Rocks

5. Safety Against Slipping: As the cell tends to tip over the toe, the fill material has the tendency to run out. The piles on the water side creep upward as one unit, but the piles on the dry side slip relative to each other. This behavior occurs because the frictional resistance between the fill and the piles on the water side is smaller than the frictional resistance along the interlock.

   On the water side, the force P_d pushes the pile against the fill. The frictional resistance between the pile and the fill is equal to $\mu' P_d$

The factor of safety against slipping is given by

$$F_{sp} = \frac{\text{Frictional resistance against upward movement}}{\text{Upward force}}$$

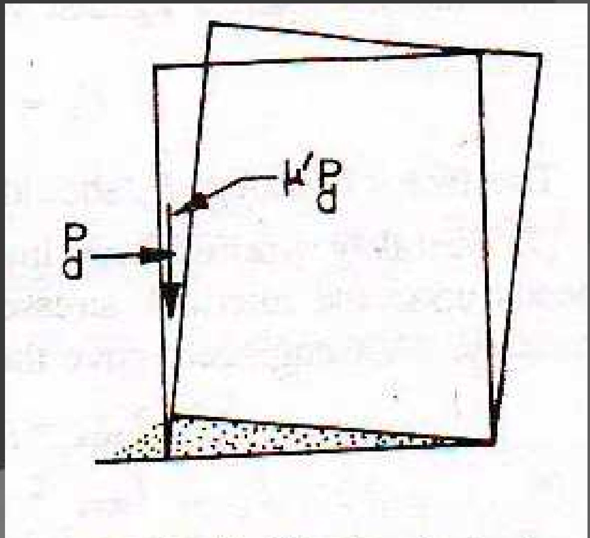

Fig. 21.19. Slipping Analysis.

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The value of friction coefficient μ' is equal to tan δ ,where δ is the angle of friction between the fill and the pile.
The minimum width b required can be obtained by taking moments about the toe. Thus

$$P_d \times \bar{Z} \times F_{sp} = P_d \tan \text{¢} \times b$$

Or

$$b = \frac{\bar{Z} F_{sp}}{\tan \text{¢}}$$

The value of F_{sp} is usually taken as 1.25.

If the sheet pile is embedded in the rock for a substantial depth, the effect of the active pressure and the passive pressure should also be considered when summing up the moments about the toe.

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6. Safety against vertical shear:

The cell may fail due to vertical shear developed along a plane through its centre line. The maximum shear (V_{max}) is obtained by computing the maximum bending moment acting on the cell, considering it is as a vertical cantilever.

Thus

$$V_{max} = 3M/(2b)$$

Where M = bending moment due to external forces above the base.

For stability, the shearing resistance developed must be greater than V_{max}.

The shearing resistance is equal to the sum of the shearing resistance of soil (S_1) and the resistance due to interlock (S_2) obtained as follows:

$$S_1 = \frac{1}{2}\gamma_a H^2 \left[\frac{\cos^2\phi}{2-\cos^2\phi}\right]\tan\phi$$

or

$$S_1 = \frac{1}{2}\gamma_a H^2 K \tan\phi$$

Where K = coefficient of earth pressure having a value greater than that for the active pressure,

ϕ = angle of internal friction of the cell fill,

γ_a = average unit weight of soil.

The resistance S_2 is equal to the interlock tension T multiplied by the coefficient of friction (f). $S_2 = f \cdot T$

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The maximum pressure is developed at a height of (3/4) Hf above the base, where Hf is the height of cell above the point of fixity.

Thus

$$T = \frac{1}{2} ¥_a H (3/4 H_f) K_a$$

$$S_2 = \frac{3}{8} ¥_a H H_f K_a f$$

The coefficient of interlock friction f is generally taken as 0.30.
The factor of safety against vertical shear is given by

$$Fv = \frac{S_1 + S_2}{V_{max}}$$

The factor of safety *Fv* should not be less than 1.25.

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7. Stability against bursting:

The cell should be safe against bursting. The stability against bursting depends upon the interlock stresses. The interlock stresses are quite complicated and cannot be determined accurately. TVA engineers gave the following expression for the ring tension (T_{max}) for 90^{o} – Tees.

$$T_{max} = pL\sec\theta$$
$$T_{max} \approx pD/2$$

Where p = horizontal pressure due to cell fill, given by

$$p = \gamma_a H' K_a + \gamma_w H_w$$

In which H’ = depth of soil upto that level, H_w = water depth,

θ = angle which the T makes with the axis.

L = one-half the distance between centers of main cells.

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The maximum ring tension occurs at the lower quarter point (i.e. Hf/4 above the rock or ground surface).

The computed maximum interlock tension should not exceed the allowable tension (T_{all}). The factor of all safety against bursting is given by

$$F_b = \frac{T_{all}}{T_{max}}$$

The value of T_{all} is usually taken as 1500kN/m.

The value of F_b should be at least 1.25.

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Design of Cellular Coffer Dam on Soil

The procedure for the design of a coffer dam embedded in deep soil is similar to that for a coffer dam resting on rock, as discussed in sect. 21.6. However, the following additional requirements must be satisfied.

The sheet pile in sand must be driven to such a depth that the bearing capacity at that level is greater than the vertical force acting on the pile. A minimum factor of safety of 1.50 is generally recommended.

The maximum vertical force per unit length (Q) developed is equal to the frictional resistance between the fill and the pile and is given by

$$Q = \frac{1}{2}\gamma H^2 K_a \tan\delta$$

Where H = height of cell above top of the stratum,

K_a = coefficient of active earth pressure

δ= angle of friction between fill and pile

γ = unit weight of cell fill

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$$\text{Factor of safety} = \frac{Q_{ult}}{Q}$$

Where, Q_{ult} =ultimate load capacity against bearing capacity failure.

If the coffer dam is embedded in clay, the ultimate load capacity is given by
Q_{ult} = (5.7c)b
Where c= unit cohesion

The ultimate load capacity should be greater than the fill load. The factor of safety is given by,

$$F = \frac{Q_{ult}}{Fillload} = \frac{5.7cb}{¥bH}$$

$$F = \frac{5.7c}{¥H}$$

$$H = \frac{5.7c}{¥F}$$

A minimum factor of safety of 1.50 is recommended.

If the coffer dam is embedded in soft to medium clay, it should be safe against caused by unequal settlement. The tilting can be estimated from the compressibility characteristics of the soil.

Cellular coffer dams on a deep sand deposit should have sufficient factor of safety against piping failure.

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Figure shows a coffer dam founded on deep sand bed. Water percolates under the base of the coffer dam and rises up in front of the toe. The flow net can be drawn as shown. The flow lines are almost vertical in front of the toe. If the seepage pressure is equal to or greater than the submerged unit weight, quick (boiling) conditions may develop. The factor of safety against boiling is given by..

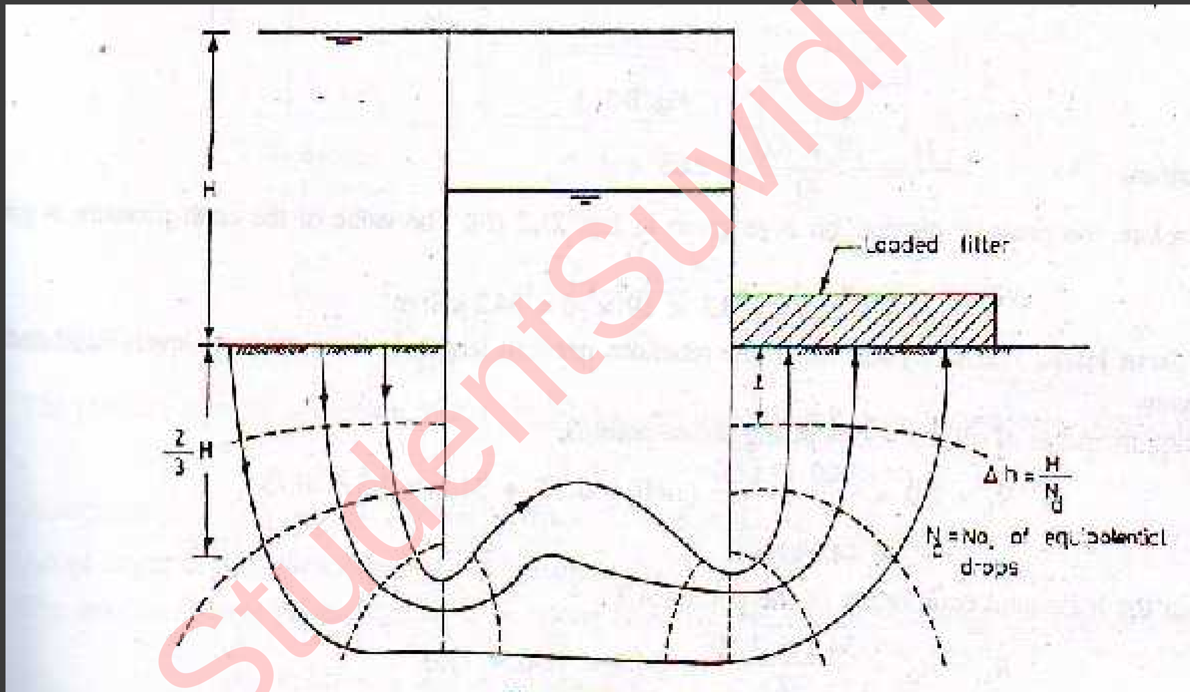

Fig. 21.20. Flow-Net

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$$F = \frac{\gamma'}{i\gamma_w}$$

Where i= hydraulic gradient at exit (= Δh / l), Δh = drop between last two equipotential lines and l = length of last flow field.

If the factor of safety is less than 1.50, a loaded filter is provided as shown to increase the downward force without increasing the seepage pressure.
The factor of safety can also be increased by reducing the gradient 'i' by driving the sheet pile deeper by reducing the effective head by permitting some water depth on the inner side.
The depth of the sheet pile below the ground surface is generally kept at least equal to two-thirds of the height of the coffer dam.

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INTER-LOCK STRESSES

Increase in interlock stresses may lead to the failure of the interlocks. Also, interlock stresses are important in estimating the stability of the cell against bursting The cell should be safe against bursting. The interlock stresses are quite complicated and cannot be determined accurately. TVA engineers gave the following expression for the ring tension (Tmax) for 90º – Tees.

$$T_{\max} = pL\sec\theta$$

$$T_{\max} \approx pD/2$$

Where p = horizontal pressure due to cell fill, given by

$$p = \gamma_a H' K_a + \gamma_w H_w$$

In which H' = depth of soil upto that level, H_w = water depth,
θ = angle which the T makes with the axis.
L = one-half the distance between centers of main cells.

Interlock stresses are developed due to

Hard driving through dense or excessively deep overburden
Distortions present in the connecting arcs
Splicing of new and used sheet piling of different manufacturer

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