Module 4

Hydraulic Structures for Flow Diversion and Storage

Lesson 7

Design and Construction of Concrete Gravity Dams

Instructional objectives

On completion of this lesson, the student shall learn:

1. The different types of embankment dams
2. Causes of failure of embankment dams
3. Design procedure for earthen embankment dams
4. Seepage control measures for embankment dams and their foundations
5. Computation of seepage through embankment dams
6. Stability calculations for embankment dams
7. Construction process for embankment dams
8. Instrumentation in embankment dams

4.7.0 Introduction

An embankment dam, as defined earlier, is one that is built of natural materials. In its simplest and oldest form, the embankment dam was constructed with low-permeability soils to a nominally homogeneous profile. The section featured neither internal drainage nor a cutoff to restrict seepage flow through the foundation. Dams of this type proved vulnerable associated with uncontrolled seepage, but there was little progress in design prior to the nineteenth century. It was then increasingly recognized that, in principle, larger embankment dams required two component elements.

1. An impervious water-retaining element or core of very low permeability of soil, for example, soft clay or a heavily remoulded ‘puddle’ clay, and
2. Supporting shoulders of coarser earthfill(or of rockfill), to provide structural stability

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. Present embankment dam design practice retains both principles. Compacted fine grained silty or clayey earthfills, or in some instances manufactured materials, like asphalt or concrete, are employed for the impervious core element. Subject to their availability, coarser fills of different types ranging up to coarse rockfill are compacted into designated zones within either shoulder, where the characteristics of each can best be deployed within an effective and stable profile.

Although the loads acting on the concrete gravity dam (discussed in lesson 4.6) is the same acting on the embankment dam, the method of design and analysis of the two differ considerably. This is mostly because the gravity dam acts as one monolithic structure, and it has to resist the destabilizing forces with its own self weight mainly. Failure to do so may lead to its topping, sliding or crushing of some of the highly stressed regions. An embankment dam, on the other hand, cannot be considered monolithic. It is actually a conglomerate of particles and on the action of the various modes, which are much different from those of a gravity dam. Hence, the design of an embankment dam is done in a different way than that of a gravity dam. In fact, the design procedures are targeted towards resisting the failure of an embankment dam under different modes, which are explained in the next section.

4.7.1 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, discussed in section 4.7.2. The top crest is kept wide so as to accommodate roadway (Figure1). In order to check the seepage through the body of the dam, a number of variations are provided. For earthen embankment dams, these range from the following types:

1. Homogeneous dam with toe drain (Figure 2)
2. Homogeneous dam with horizontal blanket (Figure 3)
3. Homogeneous dam with chimney drain and horizontal blanket (Figure 4)
4. Zoned dam with central vertical core and toe drain (Figure 5)
5. Zoned dam with central vertical core, chimney filter and horizontal blanket (Figure 6)
6. Zoned dam with inclined core, chimney filter and horizontal blanket (Figure 7)

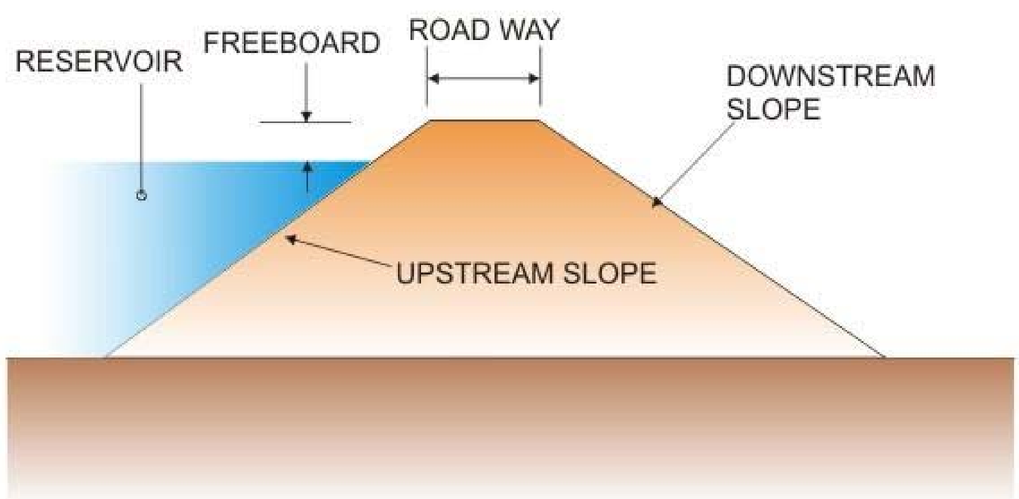


FIGURE 1. General shape of an embankment dam

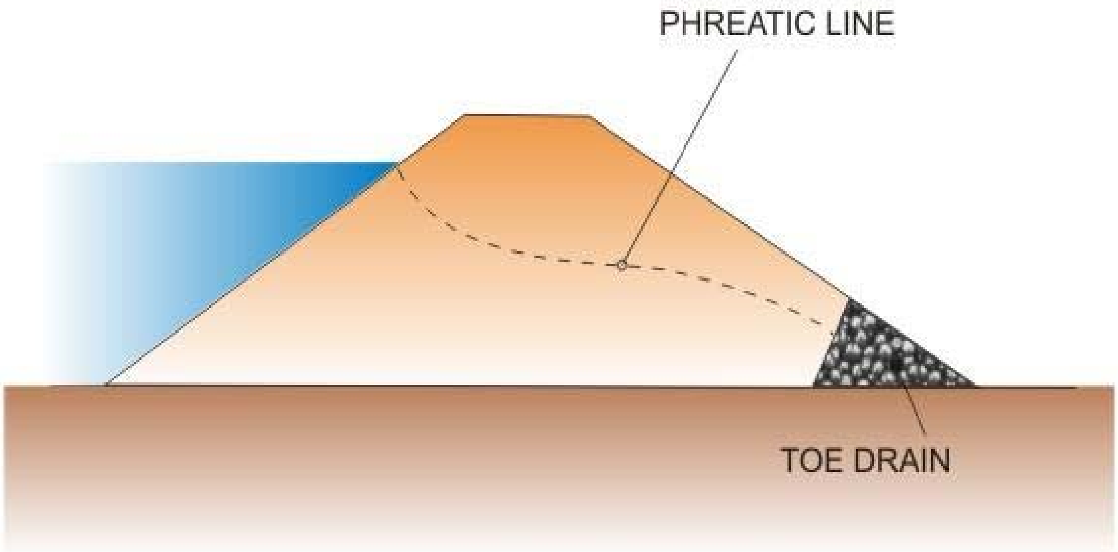


FIGURE 2. Homogeneons earthen embankment dam with toe drain

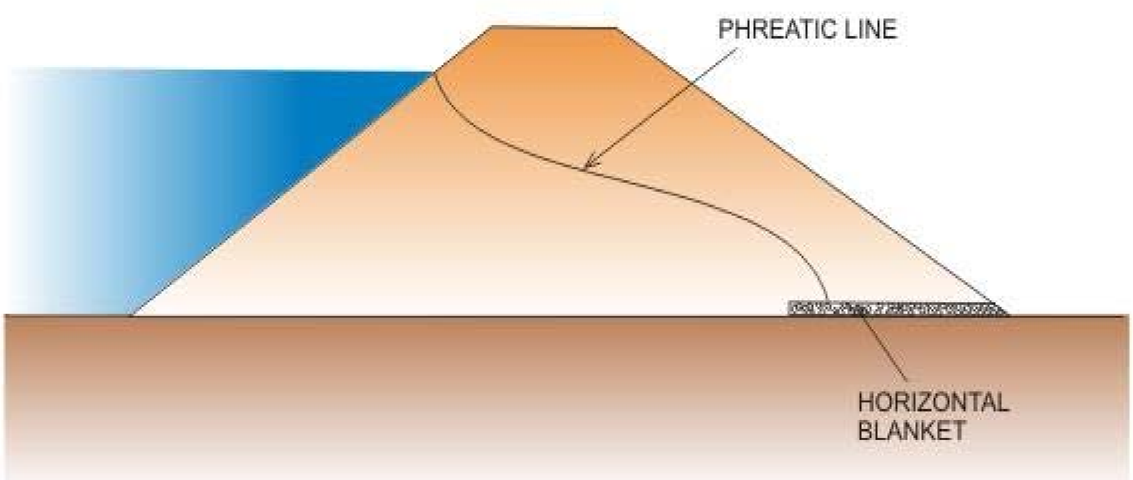


FIGURE 3.Homogeneous dam with horizontal blanket

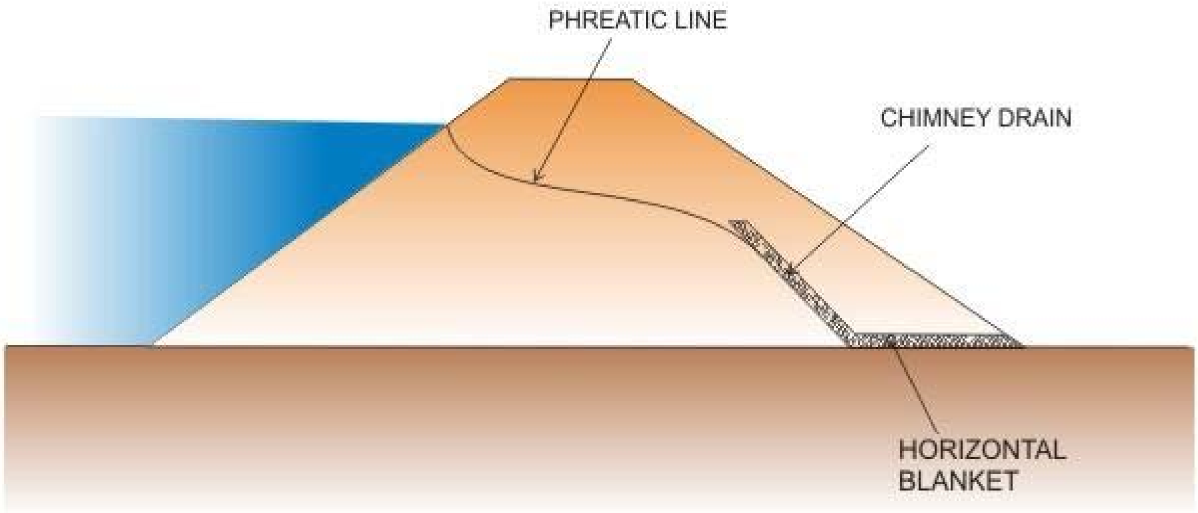


FIGURE 4. Homogeneous dam with inclined chimney drain connected to horizontal blanket

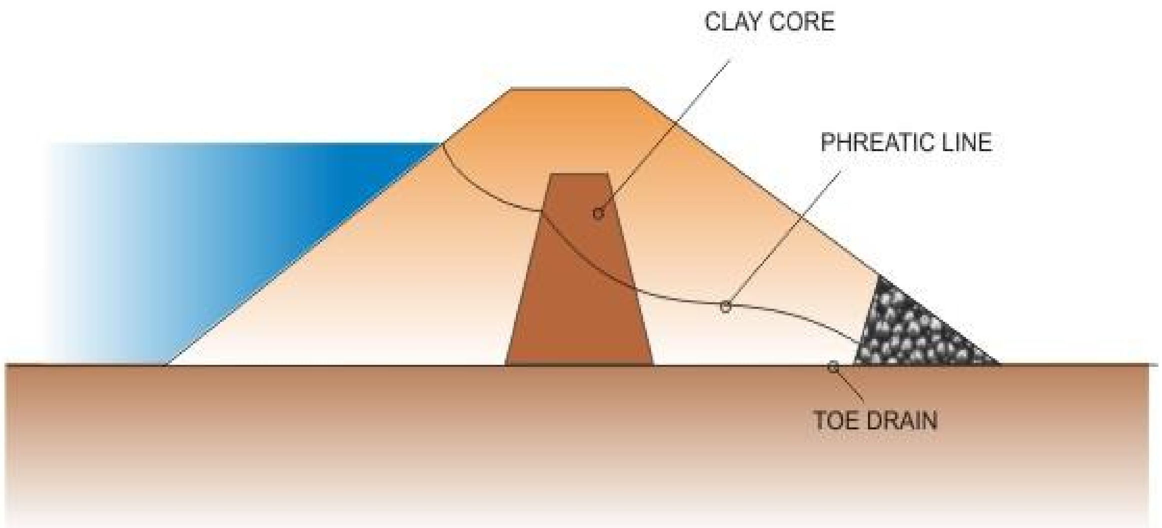


FIGURE 5. Zoned dam with central vertical clay core and toe drain

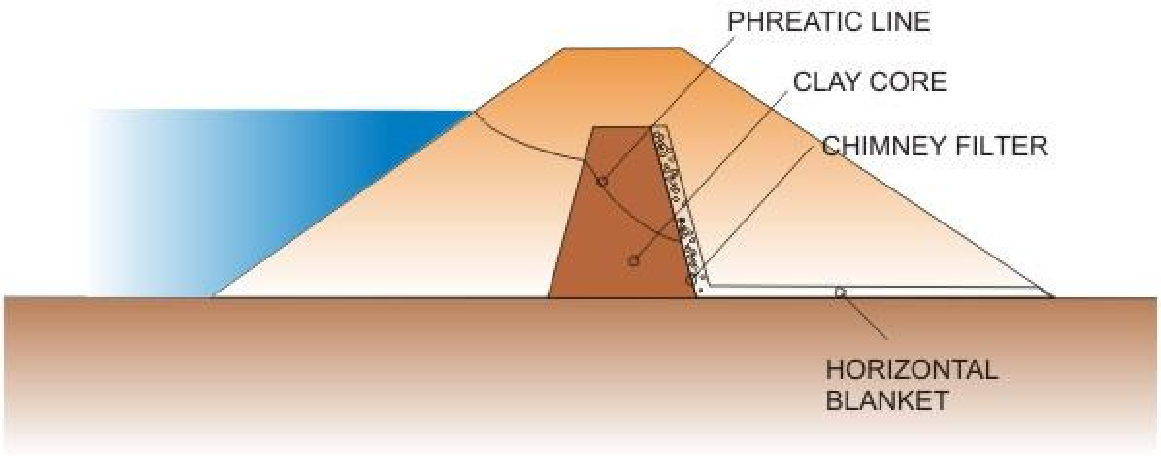


Figure 6. Clay core zoned dam with central vertical core and chimney filter with horizontal blanket

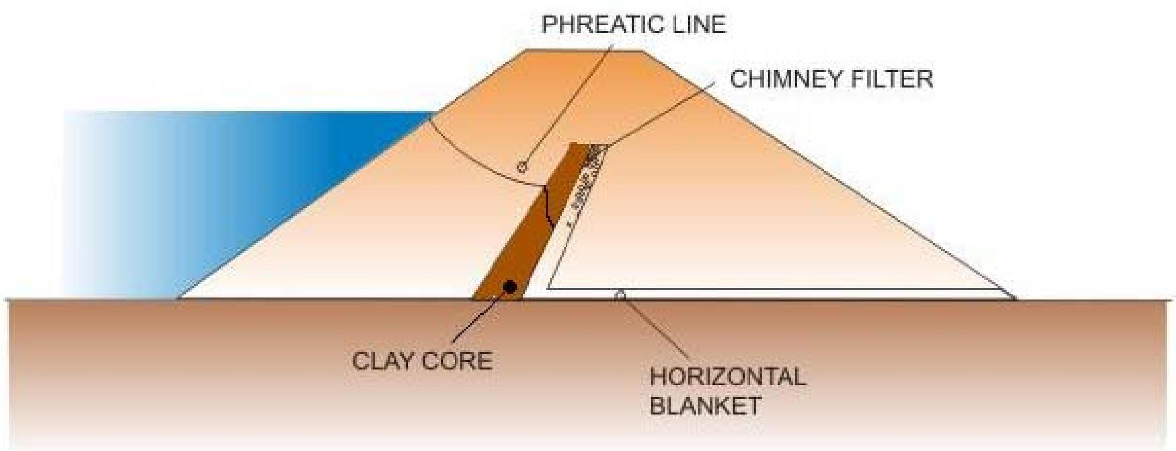


FIGURE 7. Inclined Clay core zoned dam with chimney filter and horizontal blanket.

An embankment dam is not as impervious as a concrete dam and water continuously seeps through the dam body. In Figures 2 to 7, the position of the phreatic line (that is, the line corresponding to the phreatic surface lying above the saturated zone when seen in a vertical plane) has been marked in the respective figures. It may be noticed how the phreatic line is forced to remain within the dam body by providing the clay core (with relatively less permeability) to reduce the amount of seepage water and the chimney filter with the horizontal blanket, composed of materials with very high permeability, that is used to drain out the seepage water safely through the body of the dam.

For the rockfill embankment dams, the following variants are common:

1. Central vertical clay core (Figure 8)
2. Inclined clay core with drains (Figure 9)

3. Decked with asphalt or concrete membrane on upstream face with drains (Figure 10)

All are provided with a chimney filter connected to a horizontal blanket.

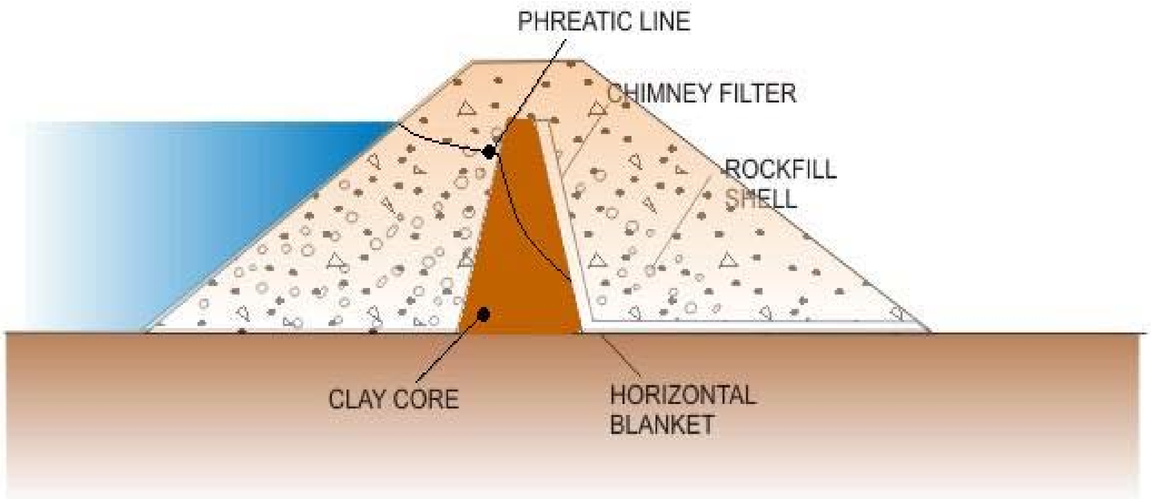


FIGURE 8. Rockfill dam with vertical clay core, chimney filter and horizontal blanket.

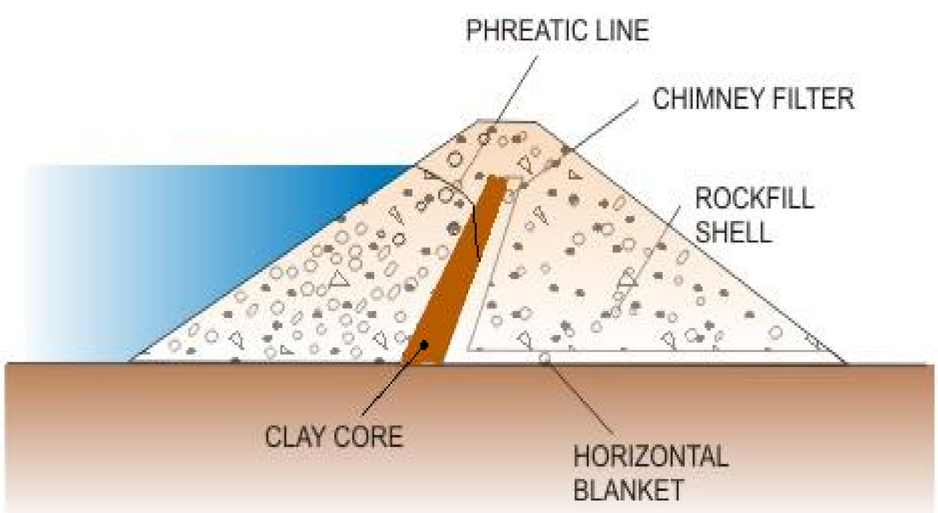


FIGURE 9 . Rock fill dam with inclined clay core, chimney drain and horizontal blanket

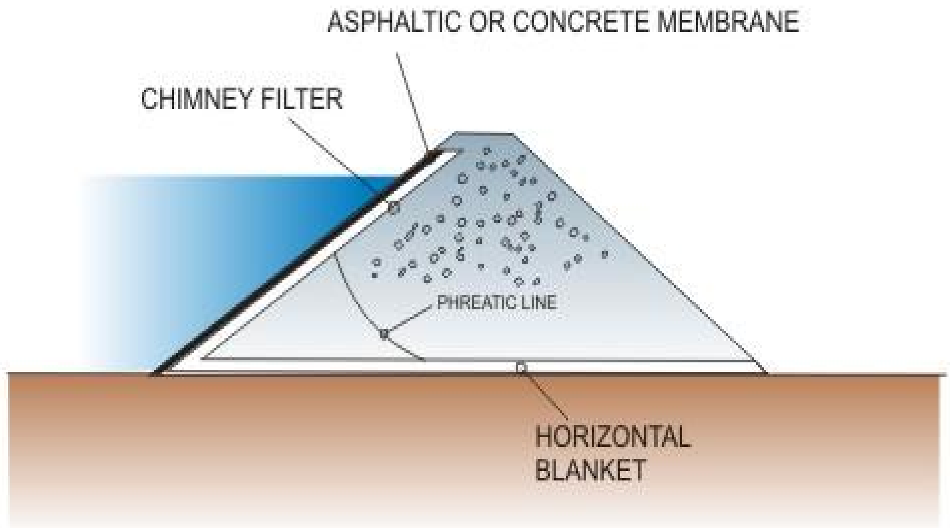


FIGURE 10. Decked rock fill dam with upstream asphaltic or concrete membrane with chimney drain and horizontal blanket. The phreatic live is for the small amount of water that leaks through the cracks of the upstream membrane.

Here, too it may be observed that the clay core with relatively very low permeability or the asphaltic and concrete membranes serve to reduce the quantity of seepage water. The rockfill shell only serves as a support to the core or membrane. The phreatic line through the rock fill is very gently inclined due to the materials of high permeability.

Since both the upstream and downstream faces of an embankment are inclined, usually varying in the ratio of 1V:1.5H to 1V:2.0H, the plan view of such a dam in a river valley would look as shown in Figure 11.

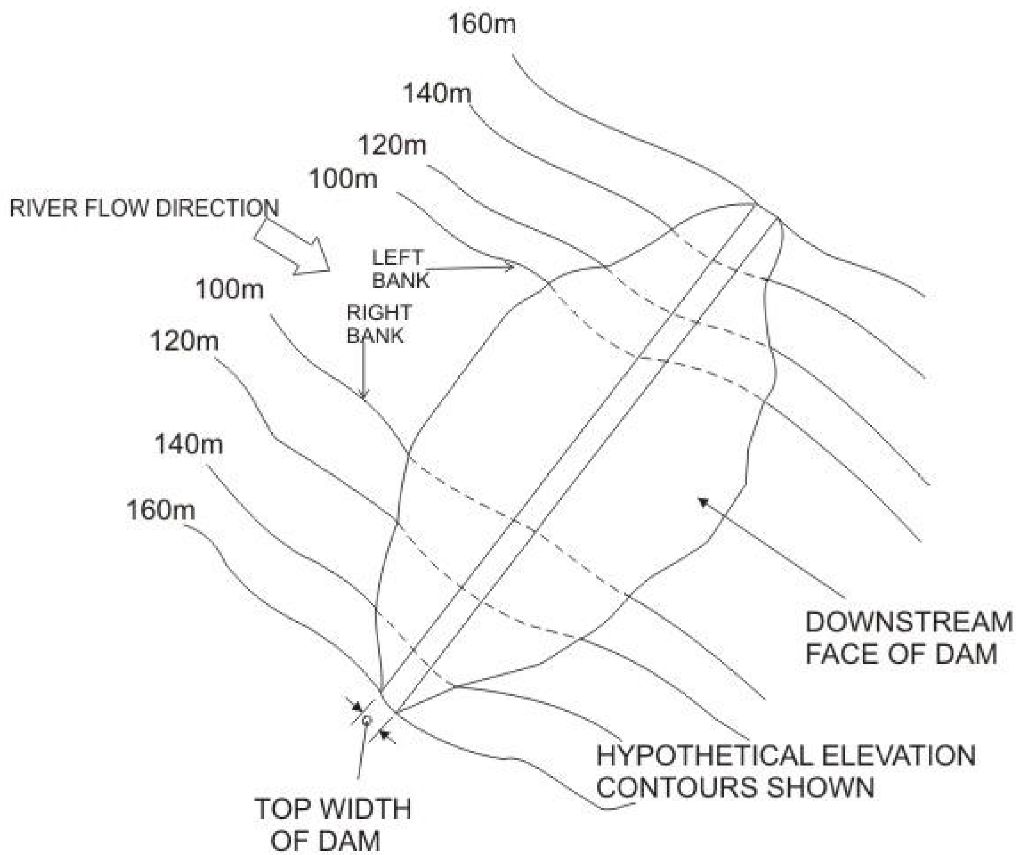


FIGURE 11. Layout of an embankment dam within a river valley

Though this is entirely satisfactory, the problem remains in providing a spillway to let out flood flows. This cannot be done through an embankment dam, since that would lead to its washout. Instead the spillway portion is usually made as a concrete gravity dam section, as shown in Figure 12 for the layout of Konar Dam on Konar river, in Jharkhand.

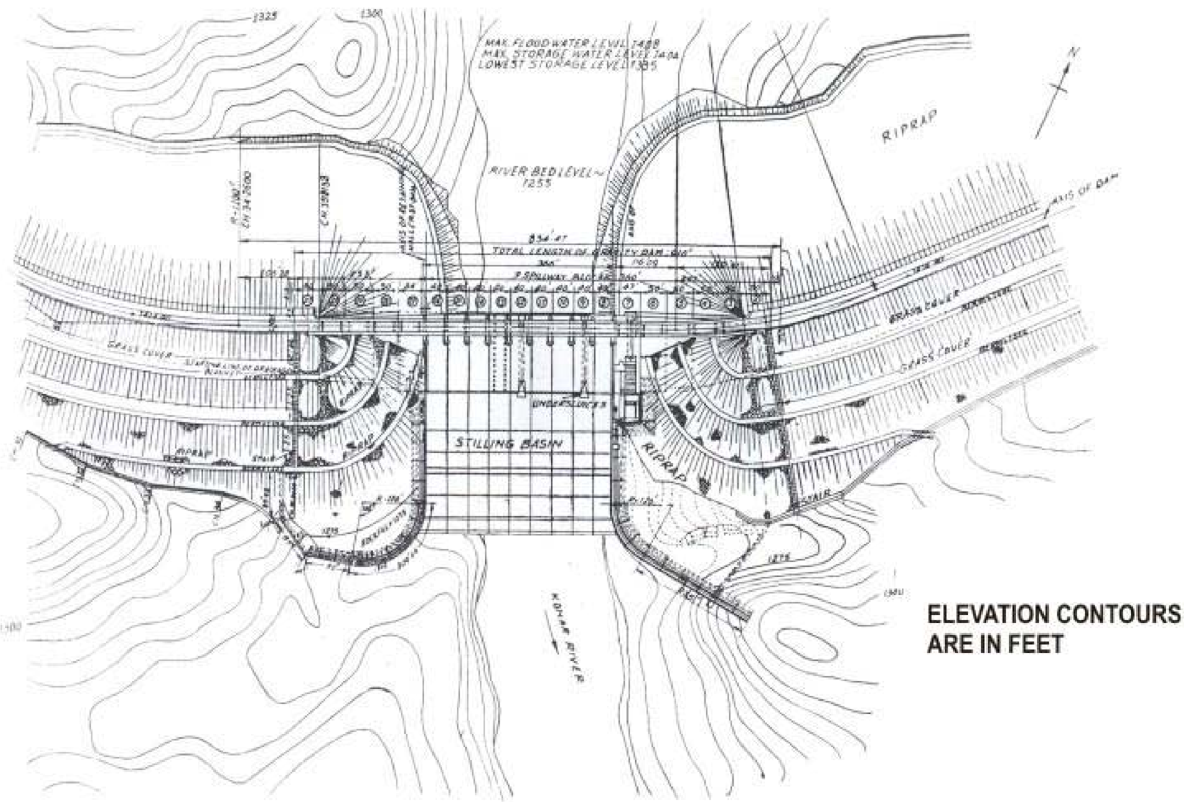


FIGURE 12. Layout of Konar Dam on River Konar, and located in Jharkhand (Drawing courtesy: CBIP Publication 138, volume II)

Typical cross section through the embankment portion and the spillway portion are shown in Figure13.

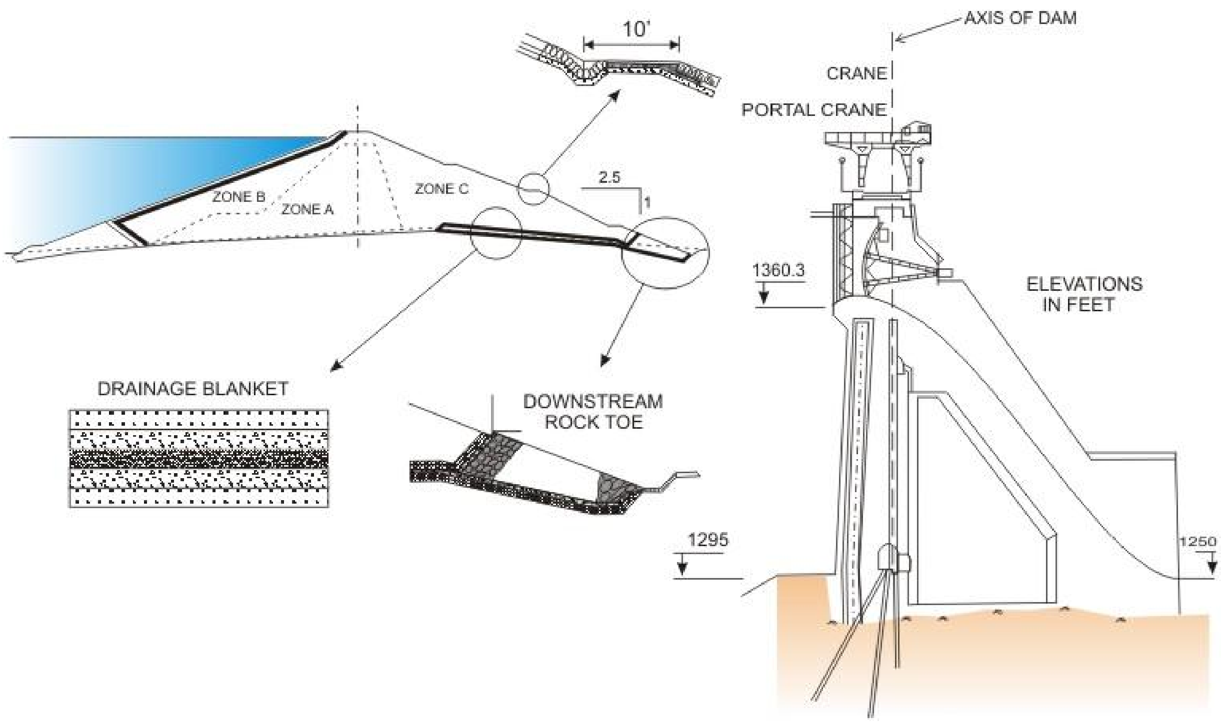
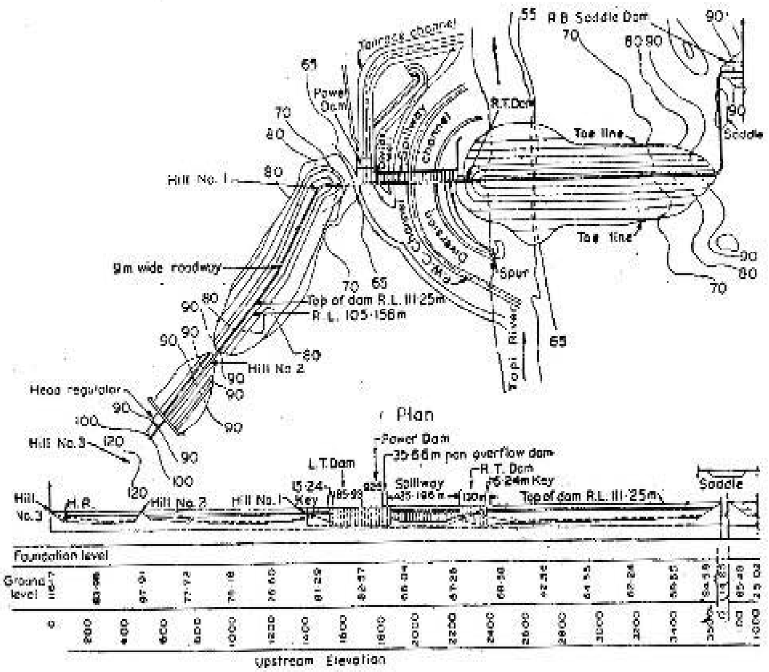


FIGURE 13. Cross sections of embankment dam and concrete gravity dam spillway of Konar dam shown in FIGURE 12

Another layout of the combination of an earthen embankment dam and a concrete spillway is shown in Figure14, for Ukai dam on Tapi river, in Gujarat.

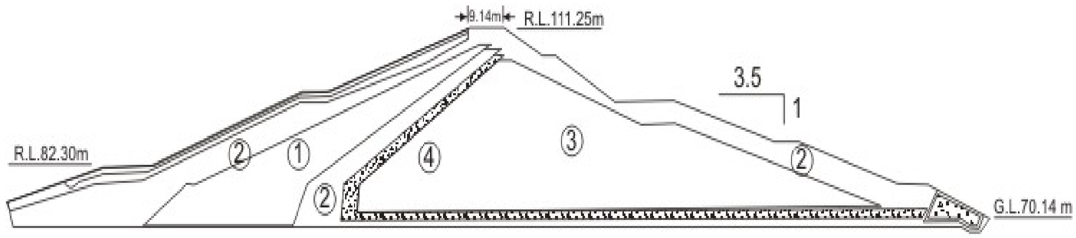


ELEVATIONS
IN METRES

FIGURE 14 . Layout of Ukai dam- showing concrete spillway and embankment dam

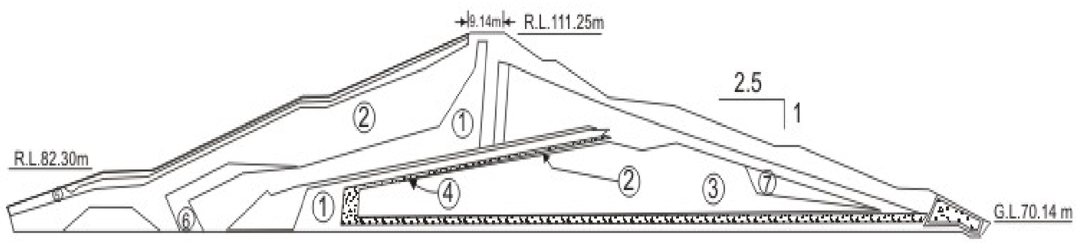
(Drawing courtesy: CBIP Publication 138, Volume I)

Typical cross sections of the two types of embankment dam section used in the project is shown in Figure 15.



Legend

1. Impervious material
2. Semi-previous material
3. Random fill (material available from excavation)
4. Filter



Legend

1. Impervious zone
2. Semi-previous zone
3. Random zone
4. Sand Filter
5. Rock zone
6. Silt zone
7. Sand zone

FIGURE 15. Typical section of embankment dams constructed in Ukai project

Both the spillway section and the power dam section conveying penstock for power houses are made of concrete and are shown in Figures 16 and 17 respectively.

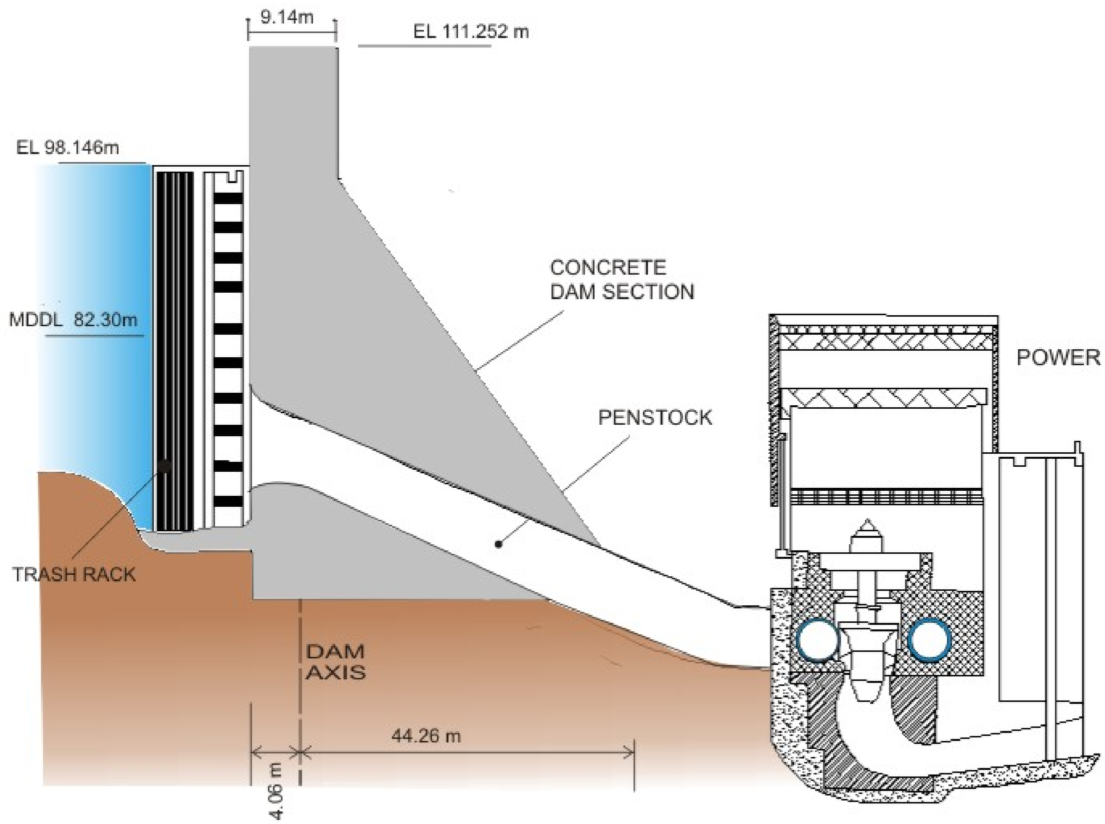


FIGURE 16. Typical power-dam section Ukai project

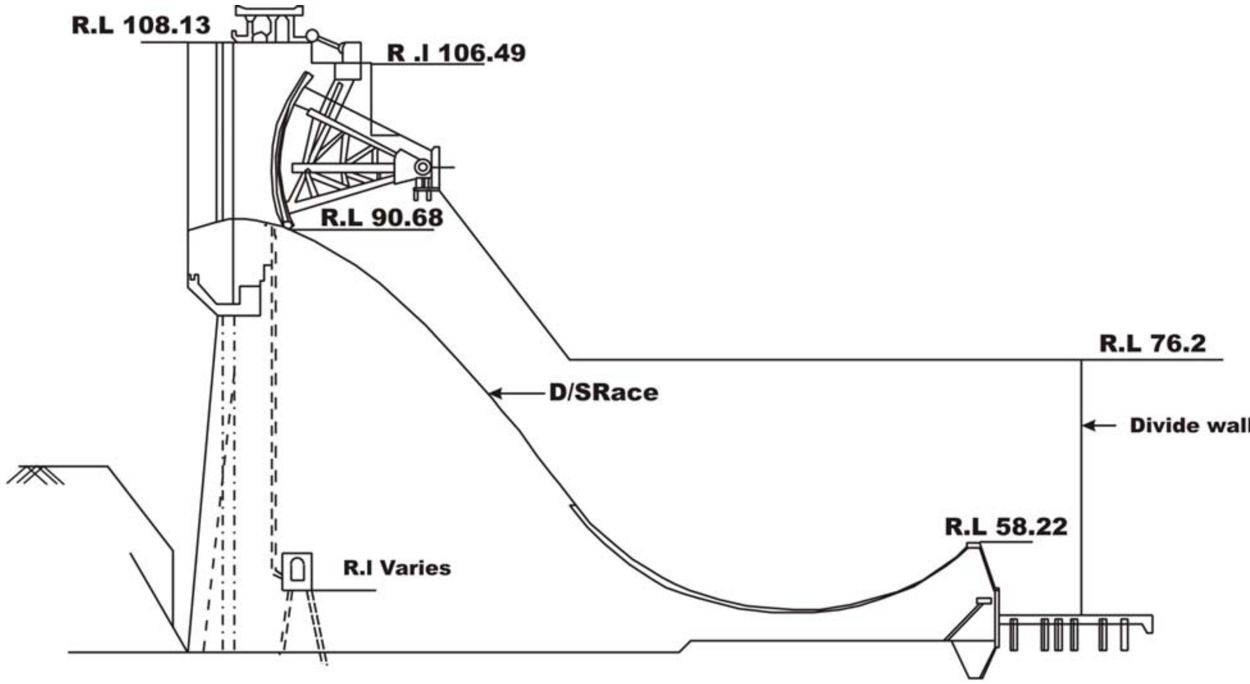


FIGURE 17. CROSS-SECTION OF SPILLWAY SECTION OF UKAI DAM PROJECT

The example of Beas Dam at Pong (Figure 18) may be cited to show a typical layout for outlet work through the embanked section (Figure19).

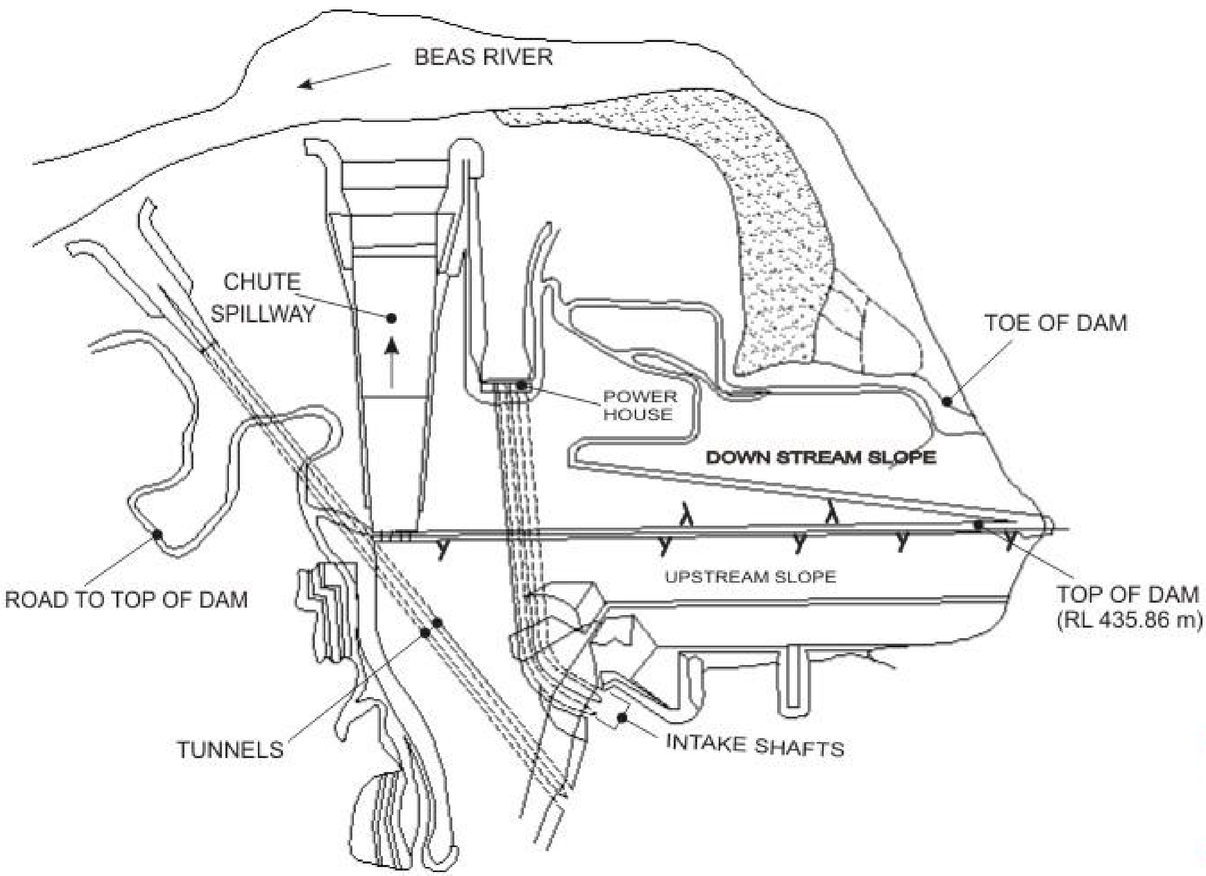


FIGURE 18 . Layout of Beas dam at Pong

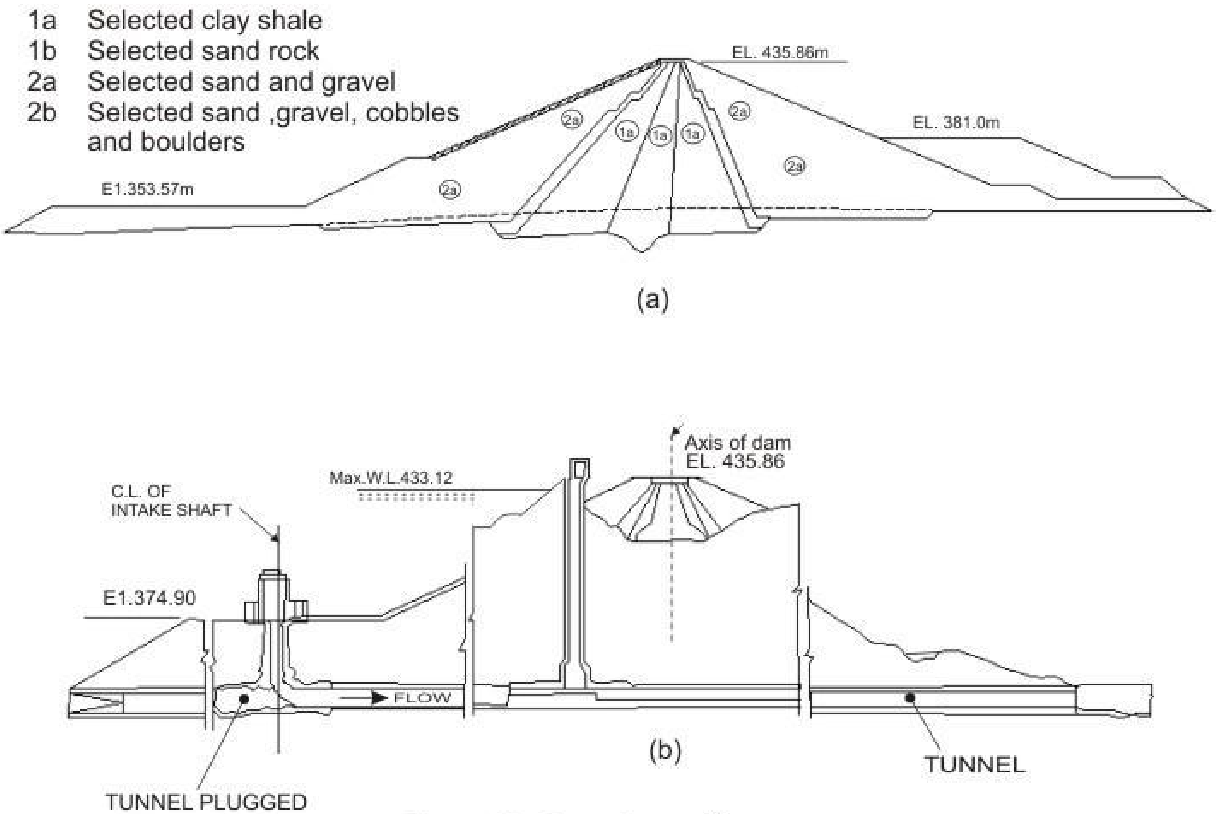
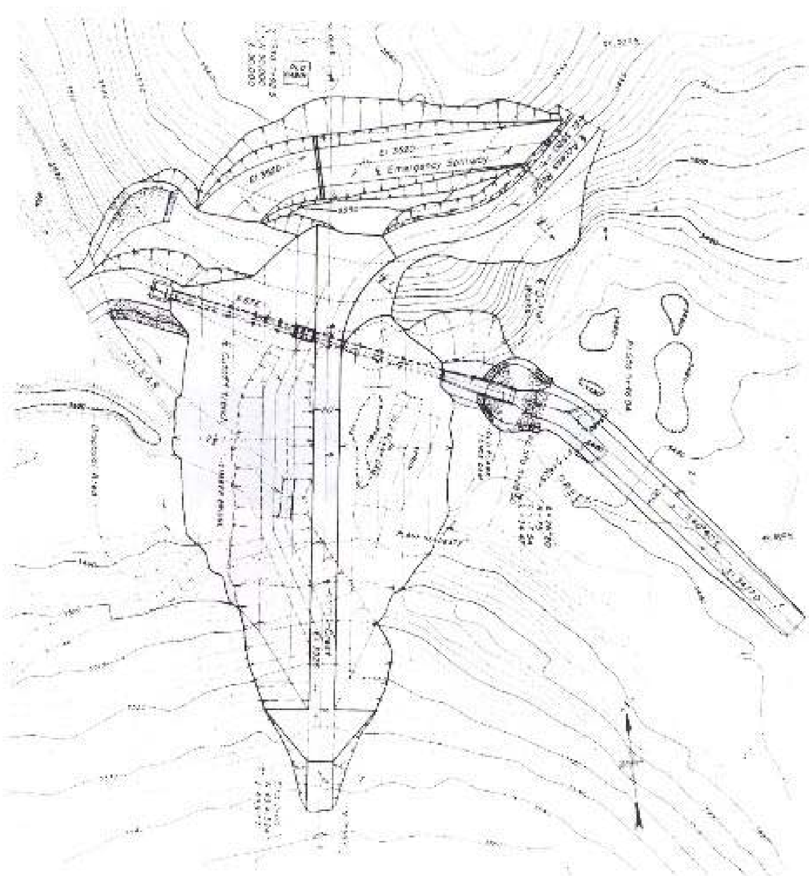


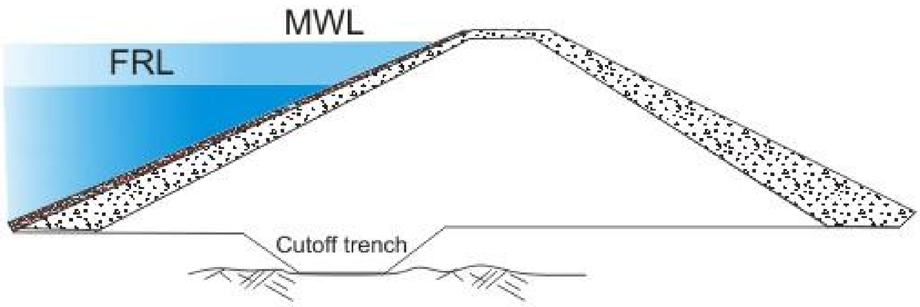
Figure 19. Beas dam at Pong
(a) Maximum section of earth dam
(b) Outlet work for power house through earth dam section

Another example is given in Figure 20 of the Wasco Dam, on Clear Creek, U.S.A, which shows an emergency spillway excavated out of left abutment hills and outlet work using rectangular duct through the bottom of the embankment dam (Figure 21).



ELEVATION CONTOURS IN FEET

FIGURE 20. General layout of Wasco Dam, Clear Creek,USA
Image courtesy: "Design of Small Dams", published by USBR



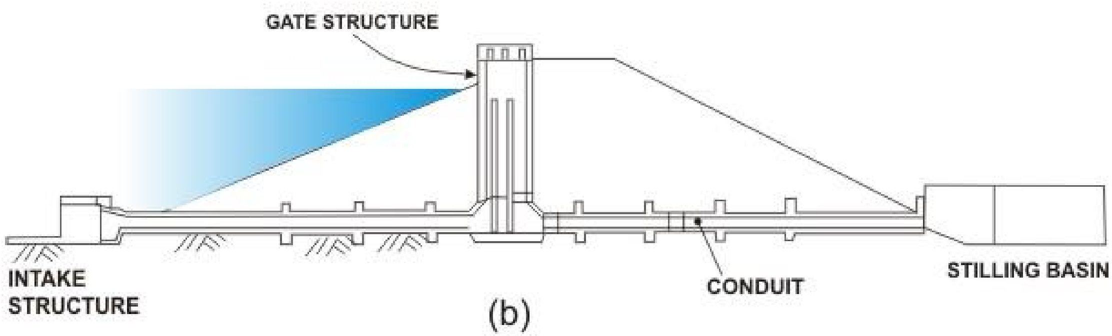


FIGURE 21. Cross sectional details of embankment dam shown in FIGURE 19.
(a) Through maximum section
(b) Through outlet work

4.7.2 Causes of failure of earth dams

The various modes of failures of earth dams may be grouped under three categories:

1. Hydraulic failures
2. Seepage failures, and
3. Structural failures

These modes of failures are explained in some detail in the following paragraphs.

Hydraulic failures

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

1. Overtopping of the dam which might have been caused by a flood that exceeded the design flood for the spillway. Sometimes faulty operation of the spillway gates may also lead to overtopping since the flood could not be let out in time through the

spillway. Overtopping may also be caused insufficient freeboard (the difference between the maximum reservoir level and the minimum crest level of the dam) has been provided. Since earth dams cannot withstand the erosive action of water spilling over the embankment and flowing over the dam's downstream face, either complete or partial failure is inevitable (Figure 22).

2. Erosion of upstream face and shoulder by the action of continuous wave action may cause erosion of the surface unless it is adequately protected by stone riprap and filter beneath (Figure 23).
3. Erosion of downstream slope by rain wash. Though the downstream face of an embankment is not affected by the reservoir water, it may get eroded by heavy rain water flowing down the face, which may lead to the formation of gullies and finally collapse of the whole dam (Figure 24).
4. Erosion of downstream toe of dam by tail water. This may happen if the river water on the downstream side of the dam (which may have come from the releases of a power house during normal operation or out of a spillway or sluice during flood flows) causes severe erosion of the dam base. (Figure 25).

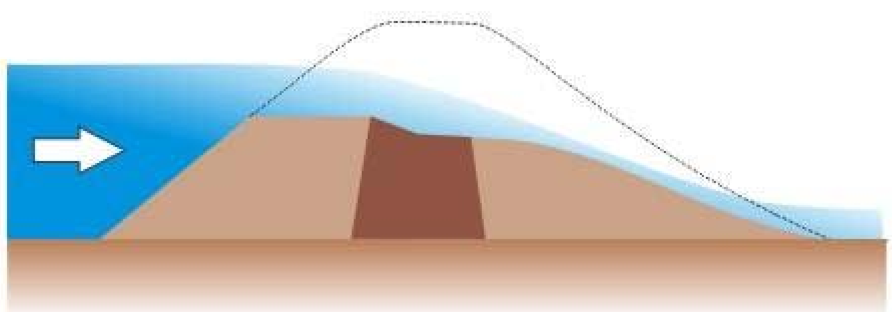


FIGURE 22. Overtopping of dam resulting in washout

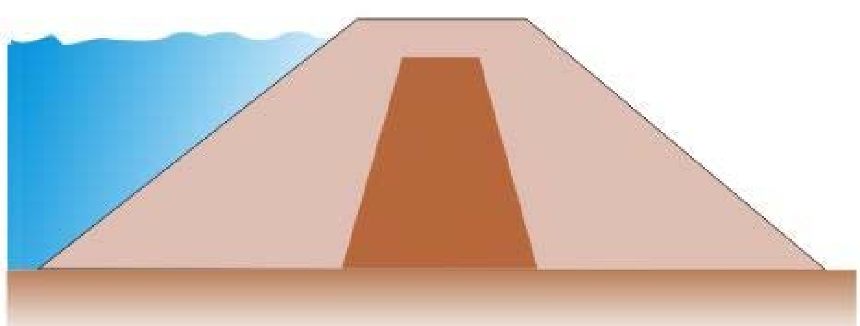


FIGURE 23. Erosion of upstream face by waves breaking on the surface

Figure 24. Scour of downstream face by impact of rain and resulting sheet flow

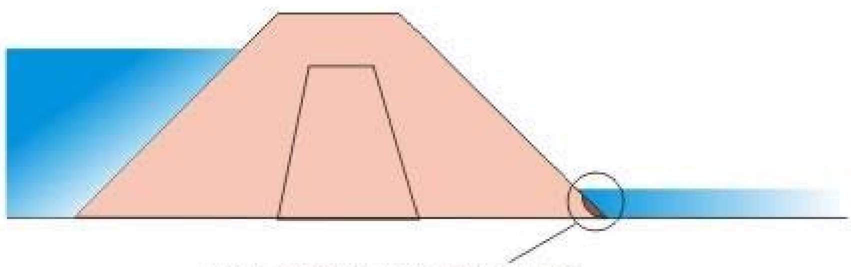


Figure 25. Scour on the downstream toe by tailwater

Seepage failures

The water on the reservoir side continuously seeps through an embankment dam and its foundation to the downstream side. Unless a proper design is made to prevent excessive seepage, it may drive down fine particles along with its flow causing gaps to form within the dam body leading to its collapse. Seepage failures may be caused in the following ways:

1. Piping through dam and its foundation: This is the progressive backward erosion which may be caused through the dam or within its foundation by the water seeping from upstream to the downstream (Figure 26)

2. Conduit leakage: This is caused due to seepage taking place by the surface of a conduit enclosed within an embankment dam (Figure 27). The seepage of water may be from the reservoir to the downstream or due to the water leaking out of the conduit through cracks that might have developed due to unequal settlement of dam or by overloading from the dam. Further, the cracking of a conduit may also be caused when the soil mass lying below it settles and the conduit is not sufficiently strong to support the soil mass lying above.
3. Sloughing of downstream face: This phenomena take place due to the dam becoming saturated either due to the presence of highly pervious layer in the body of the dam. This causes the soil mass to get softened and a slide of the downstream face takes place (Figure 28)

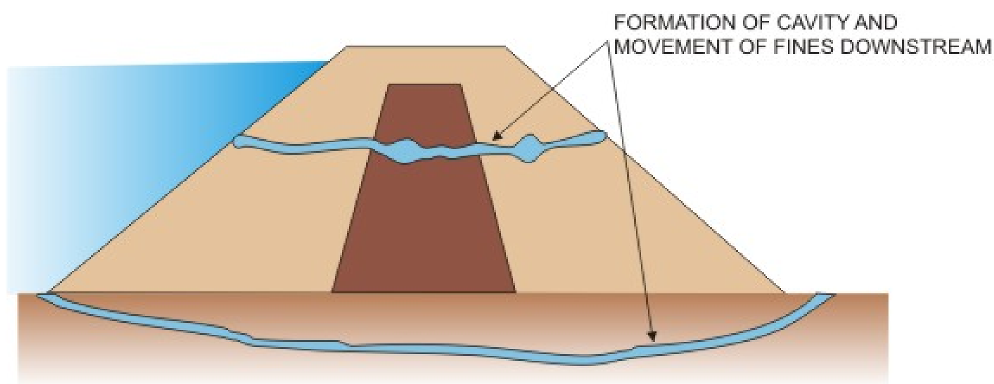


FIGURE 26. Internal erosion and piping through dam body and foundation

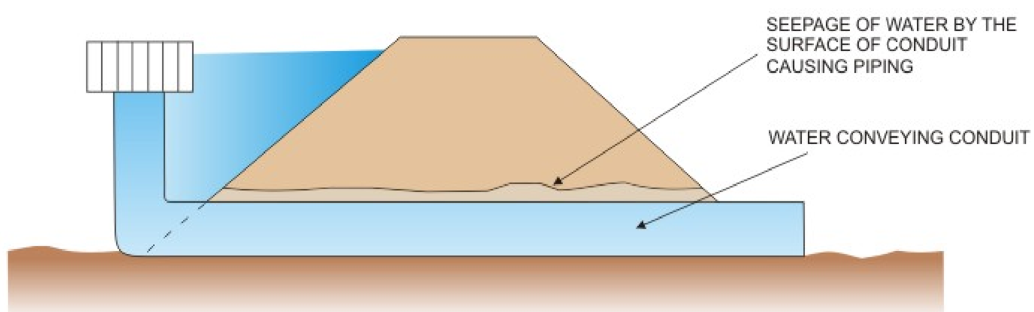


FIGURE 27. Seepage by the outer surface of conduit; may lead to progressive piping

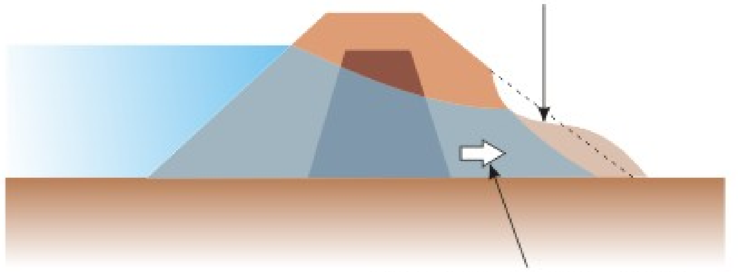


FIGURE 28. Sloughing of downstream face due to high pore water pressure

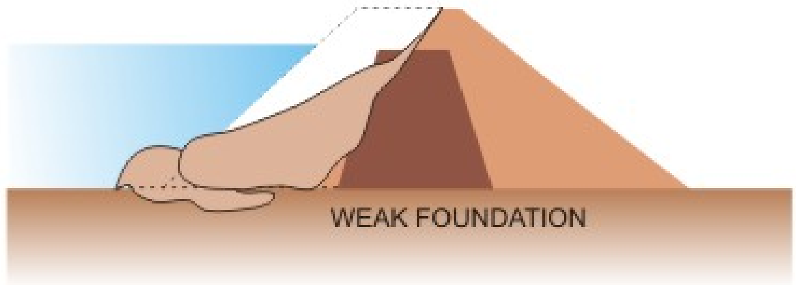
Structural failures

These failures are related to the instability of the dam and its foundation, caused by reasons other than surface flow (hydraulic failures) or sub-surface flow (seepage-failures). These failures can be grouped in the following categories:

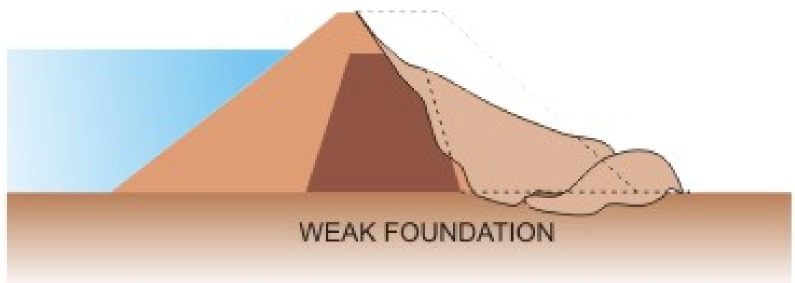
1. Sliding due to weak foundation: Due to the presence of faults and seams of weathered rocks, shales, soft clay strata, the foundation may not be able to withstand the pressure of the embankment dam. The lower slope moves outwards along with a part of the foundation and the top of the embankment subsides (Figure 29) causing large mud waves to form beyond the toe.
2. Sliding of upstream face due to sudden drawdown: An embankment dam, under filled up condition develops pore water pressure within the body of the dam. If the reservoir water is suddenly depleted, say due to the need of emptying the reservoir in expectation of an incoming flood, then the pore pressure cannot get released, which causes the upstream face of the dam to slump (Figure 30).
3. Sliding of the downstream face due to slopes being too steep: Instability may be caused to the downstream slope of an embankment dam due to the slope being too high and / or too steep in relation to the shear strength of the shoulder material. This causes a sliding failure of the downstream face of the dam (Figure 31).
4. 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. Here, even cohesionless soil cannot drain quickly enough as the movements are so sudden that the rate of extra loading on the soil becomes greater than the rate of drainage of the seepage water out of the soil. This causes excess pore water pressure to develop, where both the effective stress and the strength decrease. Under circumstances 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, and the dam slumps.

5. Damage caused by burrowing animals or water soluble materials: some embankment dams get damaged by the burrows of animals which causes the seepage water to flow out more quickly, carrying fine material along with. This phenomena consequently leads to piping failure within the body of the dam, finally leading to a complete collapse. Similarly, water soluble materials within the body of the dam gets leached out along with the seepage flow causing piping and consequent failure.
6. Embankment and foundation settlement: Excess settlement of the embankment and/or the foundation causes loss of free board (Figure 32). The settlement may be more in the deeper portion of the valley, where the embankment height is more.



FAILURE OF UPSTREAM SLOPE



FAILURE OF UPSTREAM SLOPE

FIGURE 29. Instability of upstream or downstream slopes caused by failure of weak foundations

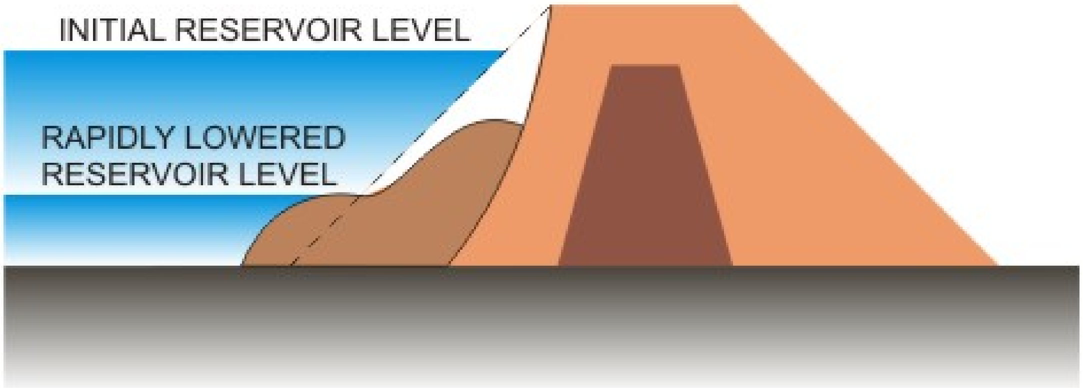


FIGURE 30. Upstream slope failure due to rapid drawdown

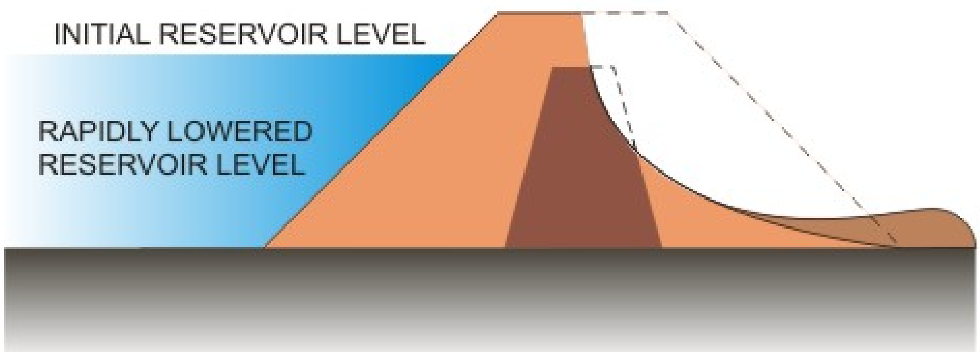


FIGURE 31. Downstream face too steep unable to be resisted by soil shear strength

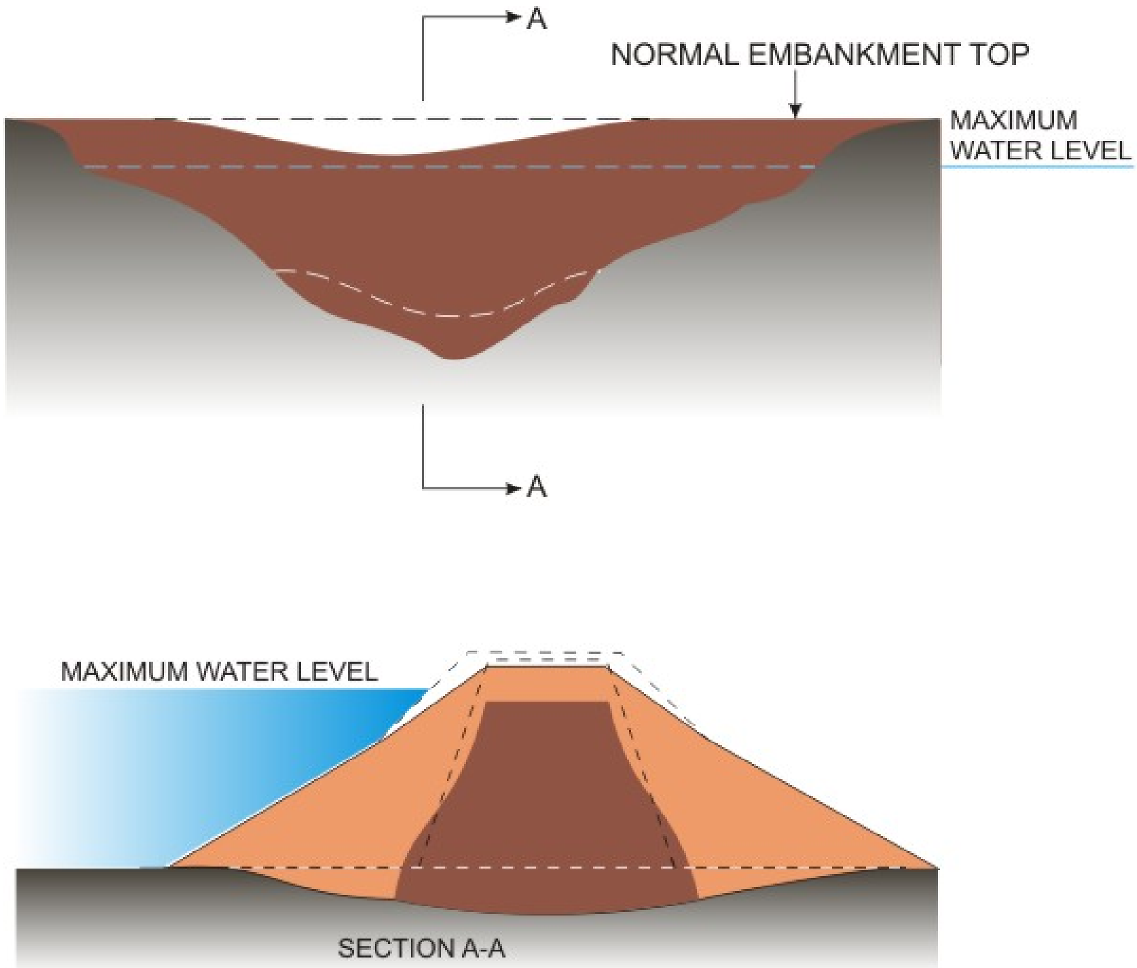


FIGURE 32. Excessive settlement of dam and foundation

4.7.3 Design of earth dams

As may be observed from the modes of failures of earth dams, there are three main types, which have to be prevented in a safe design of embankment dam. These are:

1. Safety against hydraulic failures due to overtopping , rain cuts, wave action or tail water.
2. Safety against seepage failures due to internal erosion and development of pore pressure due to insufficient drainage.
3. Safety against structural instability.
4. Special design requirements.

The features in design that must be incorporated in order to take care of the above criteria are elaborated in the Bureau of Indian Standard codes IS: 12169-1987 " Criteria for design of small embankment dams" and IS: 8826-1978 "Guidelines for design of large earth and rockfill dams" from which the following have been summarized:

1. Safety against hydraulic failures

i) Sufficient spillway capacity should be provided to prevent overtopping of embankment during and after construction.

ii) The freeboard should be sufficient enough to prevent overtopping by waves. Bureau of Indian Standards code IS: 10635-1993" Free board requirement in embankment dams-guidelines" provide methods to determine freeboard. However, for simplicity, the methods to determine free board for concrete dams may be used.

iii) Sufficient height of dam has to be provided to take care of any future settlement and consequent loss of free board

iv) Upstream slope shall be protected by riprap, which is a layer of rock fragments, against wave action. The riprap shall have to be provided from an elevation 1.5m or half of maximum wave height at Minimum Draw Down Level (MDDL), whichever is more below MDDL to the top of the dam. Figure 32 illustrates the recommended methods of terminating the ripraps at the lowest level. As may be observed from Figure 32, the riprap or pitching should be underlain with two layer of filters to prevent the water from eroding washing out of the underlying embankment material.

v) The downstream slope of the embankment should be protected by turfing, that is growing of grass on the surface against erosion by rain wash. A system of open passed drains (chutes) along the sloping surface terminating in longitudinal collecting drains at the junction of berm and slope shall have to be provided at 90m centre-to-centre to drain the rain water. Drains may be formed of stone pitching or with pre-cast concrete sections (Figure 33)

vi) Downstream slope of embankment, if affected, upon by tail water, should be protected by placing riprap of 300mm thickness over proper filter layers up to 1m above the maximum tail water level.

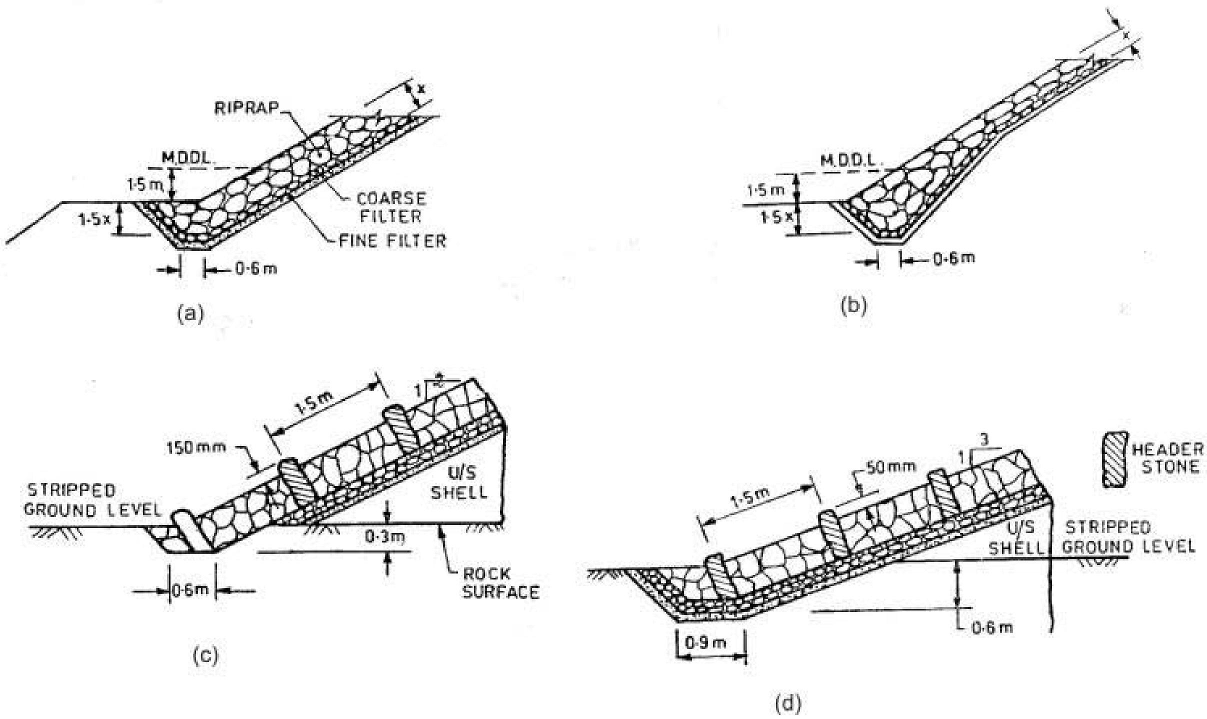


FIGURE 33. Pitching of rip rap (a) With berm below MDDL ; (b) Without berm below MDDL ; (c) Terminating at rock surface ; (d) Terminating at stripped ground level

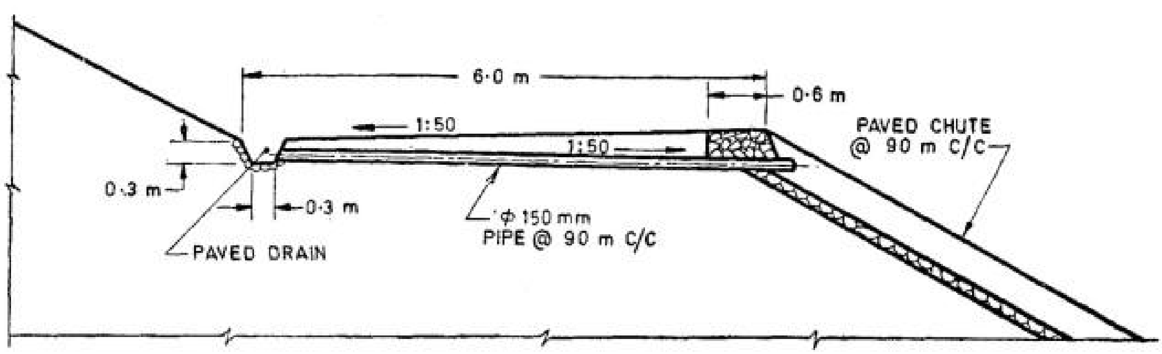


FIGURE 34. Typical downstream face surface drainage arrangement

2. *Safety against seepage failures*

i) Amount of seepage water passing through the embankment and foundation should be limited so as to control piping, erosion, sloughing and excessive loss of water. Seepage control measures are required to control seepage through the dam have to be made according to the recommendations provided in the Bureau of Indian Standards code IS: 9429-1999 “Drainage systems for earth and rockfill dams-code of practice”, which has been discussed in section 4.7.4. Design for control of seepage through foundation have to be made as per IS: 8414-1977 “Guidelines for design of under-seepage control measures for earth and rockfill dams”. Salient features of these are also summarized in section 4.7.4.

ii) The phreatic line should be well within the downstream face of the dam to prevent sloughing. Methods to estimate the location of the phreatic line is discussed in section 4.7.5

iii) Seepage water through the dam or foundation should not be so high that may cause removal of fine materials from the body of the dam leading to piping failures.

iv) These should not be any leakage of water from the upstream to the downstream face, which may occur through conduits, at joints between earth and concrete dam sections, or through holes made by burrowing animals.

3. *Safety against structural instability*

i) The slopes of the embankment on the upstream and downstream should be stable under all loading conditions. Embankment slopes have to be designed in accordance with Bureau of Indian Standards Code IS: 7894-1975 “Code of practice for stability analysis of earth dams”, which has been discussed separately in section 4.7.6.

ii) The embankment slopes should also be flat enough so as not to impose excessive stresses on the foundation, and as much, be within the permissible limits of the shear strength of the material

4. *Special design requirements*

According to IS: 8826-1978, an embankment dam should, in addition to the basic design requirements detailed above, has to satisfy the criteria of control of cracking, stability in earthquake regions, and stability at junctions. These are explained in the following paragraphs:

i) Control of cracking

Cracking of the impervious core results into a failure of an earth dam by erosion, piping, breaching, etc. Cracking occurs due to foundation settlement and/or differential movements within the embankment. Differential moments may occur due to unsuitable or poorly compacted fill materials, different compressibility and stress-strain characteristics of the various fill materials; and variation in thickness of fill over irregularly shaped or steeply inclined abutments. In order to prevent cracking of the core material, the following measures may be adopted:

a) Use of plastic clay core and rolling the core material at slightly more than optimum moisture content. In case of less plastic clay, 2 to 5 percent bentonite of 200 to 300 liquid limit may be mixed to increase the plasticity.

b) Use of wider core to reduce the possibility of transverse or horizontal cracks extending through it.

c) Careful selection of fill materials to reduce the differential movement. To restrict the rockfill in lightly loaded outer casings and to use well graded materials in the inner casings on either side of the core.

d) Wide transition zones of properly graded filters of adequate width for handling drainage, if cracks develop.

e) Special treatment, such as preloading, pre-saturation, removal of weak material, etc, to the foundation and abutment, if warranted.

f) Delaying placement of core material in the crack region till most of the settlement takes place.

g) Arching the dam horizontally between steep abutments.

h) Flattening the downstream slope to increase slope stability in the event of saturation from crack leakage.

i) Cutting back of steep abutment slopes.

ii) Stability in earthquake regions

Embankment dams which are located in earthquake affected regions are likely to be subjected to additional stresses and deformation on account of the sudden vibrations generated by seismic forces. The following factors need to be considered while designing embankment dams in earthquake prone regions:

a) Stability of the slopes should be checked under the additional seismic forces. This has to be done according to IS: 7894-1985 “Code of practice for stability analysis of earth dams”.

b) The settlement of loose or poorly compacted fill or foundation material may lead to loss of free board. Hence, an additional free board may have to be provided to take care of this situation.

c) Cracking of the impervious core leading to possible failure of the dam. In order to take care of this situation, the measures recommended for the control of cracking has to be adopted. In addition, provisions shall have to be made for discharging the maximum anticipated leakage rapidly. For this purpose, downstream zones of large quarried rock or screened gravels and cabbles have to be used. The impervious core may have to be made thicker for resisting the piping action. The top of the dam should be made thicker by increasing the crest width or by using flatter slopes at the top than would be required for dams in non-seismic regions, so as to increase the path of seepage through cracks.

d) Liquefaction of deposits of loose sand in the foundation of the dam, causing cracking, sliding, or actual horizontal movement of the dam. In order to take care of this situation, the foundation should be made as compact as possible, or re-compacted.

iii) Stability at junctions

Junctions of embankment dams with foundation abutments, concrete overflow and non-overflow dam sections, and outlets need special attention with reference to one or all of the following criteria:

a) Good bond between embankment dam and foundation
b) Adequate creep length at the contact plane
c) Protection of embankment dam slope against scouring action, and
d) Easy movement of traffic.

Earth dams may be founded on soil overburden on rock. For foundations on soils or non-rocky strata, vegetation like bushes, grass roots, trees, etc. should be completely removed. The soil containing, organic material or dissoluble salt, should also be completely removed. After removal of these materials, the foundation surface should be moistured to the required extent and adequately rolled before placing embankment material. For rocky foundation, the surface should be cleaned of all loose fragments including semi-detatched and over-hanging surface blocks of rocks. Proper bond should be established between the embankment and the rock surface so prepared.

For junction of earth dams with concrete dam block, the concrete block has to be inclined at 1V:0.5H to meet the impervious core of the earth dam. A wider impervious core and thicker transitions have to be provided at the abutment contacts to increase the length of path of seepage and to protect against erosion.

At the junction of embankment dam with outlet works, proper bond have to be provided. Staunching rings should be added to the outside of the outlet conduits in the impervious zone, at intervals, so as to increase the path of percolation along the contact. Backfilling of the trench for the outlet conduit should be done with concrete up to the top of the rock surface and the portion of the trench above the rock level should be re-filled with impervious materials compacted with moisture content about 2 percent more than optimum.

4.7.4 Seepage control measures in embankment dam and foundation

One of the basic requirements for the design of an earth or rockfill dam is to ensure safety against internal erosion, piping and excessive pore pressure in the dam. A suitably designed drainage system is therefore essential to satisfy these requirements. The seepage of reservoir water through the body of the dam or at the interfaces of the dam with the foundation or abutment creates two main problems, apart from causing excessive water loss and thereby reducing usable storage of reservoir:

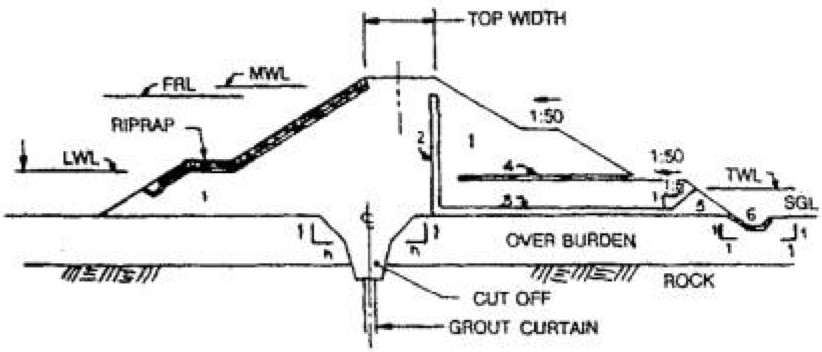
1. Seepage force causing excessive water loss
2. Piping

Inspite of taking all measures in design as well as construction to minimize seepage, it does take place either through the body of the dam or at interfaces. Whatever may be quantum of seepage, if it is not safely drained away from the toe of embankment dam into nearby drainage, valley, etc, it may lead to failure or heavy damage to the embankment, by way of slips of slopes and/or development of internal erosion leading to formation of sink holes, boiling, settlement, etc, besides creating unfriendly environment on downstream faces and areas of embankment dams.

The drainage system should be so devised that it tackles the problems mentioned in section 4.7.4. The design is mostly governed by type and permeability of base materials as well as filter materials, water depth in reservoir, topographical features of dam site, etc. The conventional types of seepage control and drainage features generally adopted for the embankment dam are:

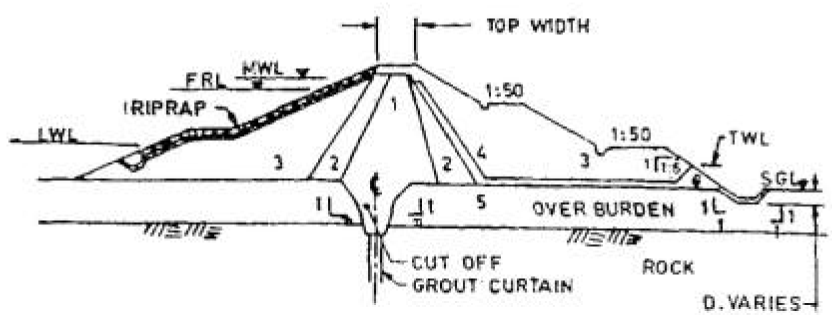
a) Impervious core,
b) Inclined/vertical filter with horizontal filter,
c) Network of inner longitudinal drain and cross drains,
d) Horizontal filter,
e) Transition zones/transition filters,
f) Intermediate filters,
g) Rock toe, and
h) Toe drain.

The drainage system may comprise of either one or a combination of more than one of these drainage features, and typical sections are shown for homogeneous dams, in Figure 35 and for zoned dams, in Figure 36. The functions of each of the components are described in the following paragraphs.



Zones:
1. Impervious fill material
2. Vertical filter
3. Horizontal filter at base
4. Intermediate filter
5. Rock toe
6. Toe drain

FIGURE 35. Section of homogenous 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

FIGURE 36. Section of zoned dam showing seepage control features

Inclined/Vertical Filter

Inclined or vertical filter abutting downstream face of either impervious core or downstream transition zone is provided to collect seepage emerging out of core/transition zone and thereby keeping the downstream shell relatively dry. In the eventuality of hydraulic fracturing of the impervious core, it prevents the failure of dam by piping.

Horizontal Filter

It collects the seepage from the inclined/vertical filter or from the body of the dam, in the absence of inclined/vertical filter, and carries it to toe drain. It also collects seepage from the foundation and minimizes possibility of piping along the dam seat.

Inner Longitudinal and Inner Cross Drains

When the filter material is not available in the required quantity at reasonable cost, a network of inner longitudinal and inner cross drains is preferred to inclined/vertical filters and horizontal filters. This type of drainage feature is generally adopted for small dams, where the quantity of seepage to be drained away is comparatively small. A typical arrangement of longitudinal and cross drains is shown in Figure 37.

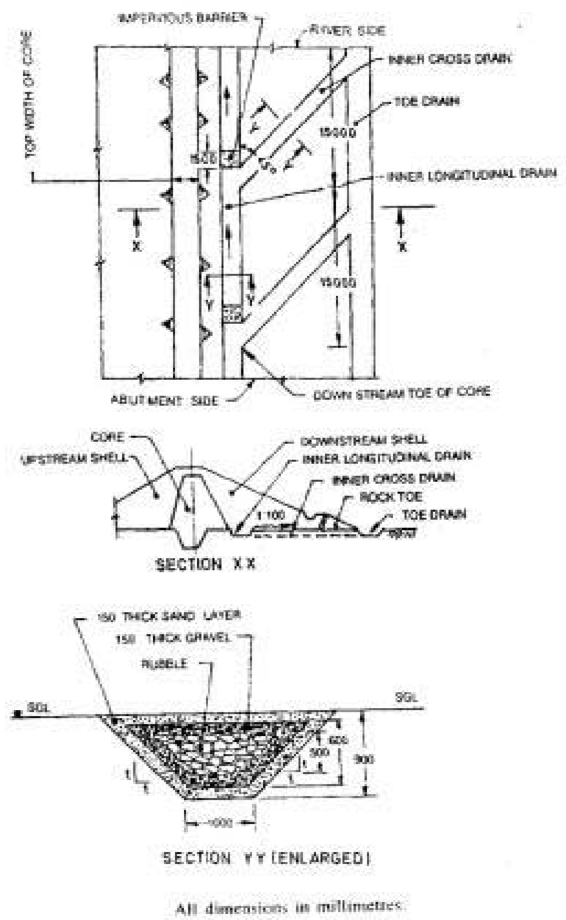


FIGURE 37. Typical arrangement of inner longitudinal and inner cross drains

Transition Zones and Transition Filters

Transition zones/filters in earth and rockfill dams in the upstream and downstream shells are necessary, when the specified gradation criterion is not satisfied between two adjacent zones. When such zones/filters are placed on either side of the impervious core, they help to minimize failure by internal piping, cracking, etc, that may develop in the core or by migration of fines from the core material.

The filter material used for drainage system shall satisfy the following criteria:

a) Filter materials shall be more pervious than the base materials;
b) 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
c) Filter material should help in formation of natural graded layers in the zone of base soil adjacent to the filter by readjustment of particles.

Horizontal Filters at Intermediate Levels

Horizontal filter layers at intermediate levels are sometimes provided in upstream and downstream shells, to reduce pore pressures during construction and sudden drawdown condition and also after prolonged rainfall (see Figure 38).

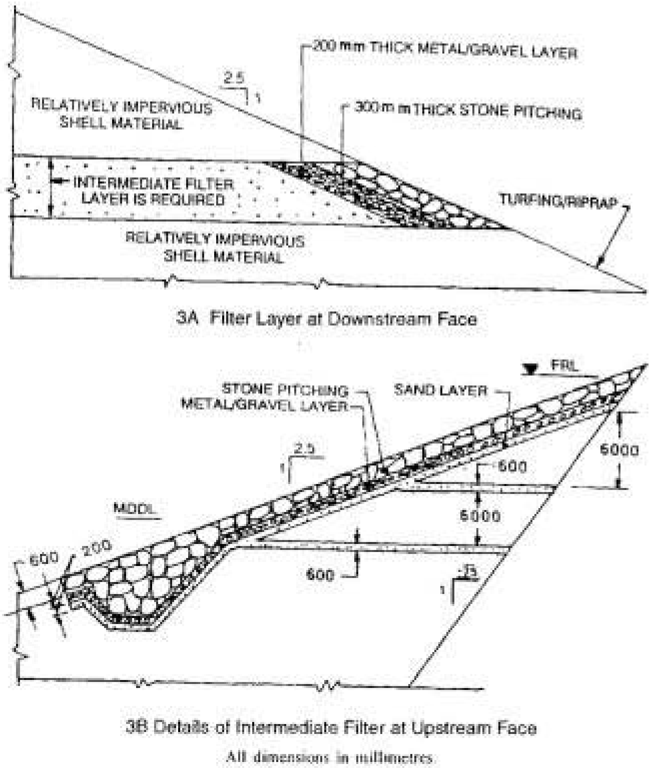


FIGURE 38. Horizontal filter at intermediate levels;
(a) On downstream, and (b) On upstream

The filter layers should be extended upto the outer slopes of the embankment so as to drain out the collected water. These filter layers should not be connected with inclined or vertical filters. A minimum space of 2.0 m or more, should be kept between the face of inclined/vertical filter and downstream intermediate filter. The material of the filter layers should be protected at exposed faces as shown in Figure 38. Details are shown in Figure 39.

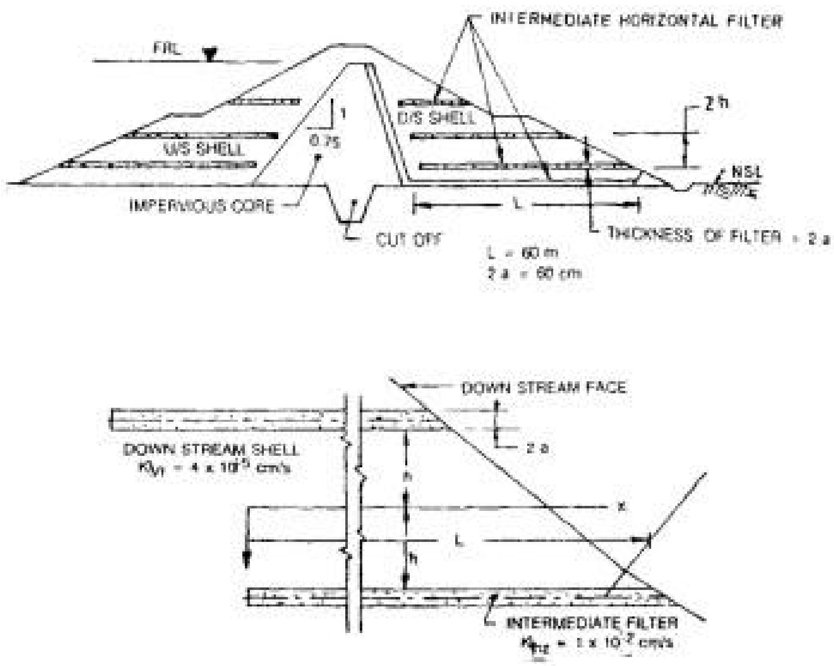


FIGURE 39. Thickness of horizontal and inverted filters

Rock Toe

The principal function of the rock toe is to provide drainage. It also protects the lower part of the downstream slope of an earth dam from tail water erosion. Rock available from compulsory excavation may be used in construction of the rock toe. Where this is not possible and transportation of rock is prohibitively costly, conventional pitching should be used for protecting the downstream toe of the dam. The top level of the rock toe/pitching should be kept above the maximum tail water level (TWL). In the reach where the ground level at the dam toe is above the maximum tail water level, only conventional pitching should be adopted. The top of such pitching should be kept 1.0 m above the top of horizontal filter, or stripped level, whichever is higher. A zone of coarse filter should be introduced between the rockfill/ pitching and the fine filter. A combination of partial rock toe and pitching may also be considered to effect economy.

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

1. Rock toe when TWL is higher than SGL (Figure 40)
2. Pitching when TWL is higher than SGL (Figure 41)
3. Rock toe + pitching when TWL is higher than rock toe (Figure 42)
4. Pitching when SGL is above TWL (Figure 43)
5. Pitching and lined toe drain (Figure 44)

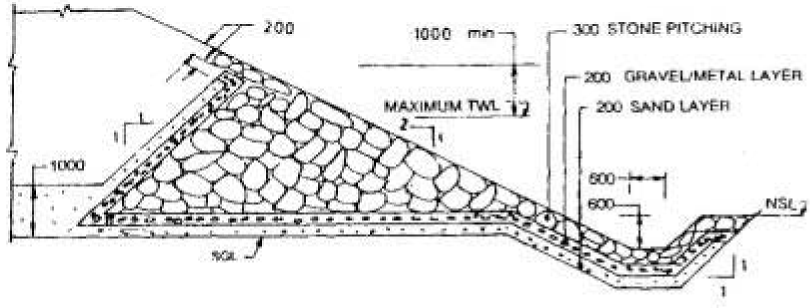
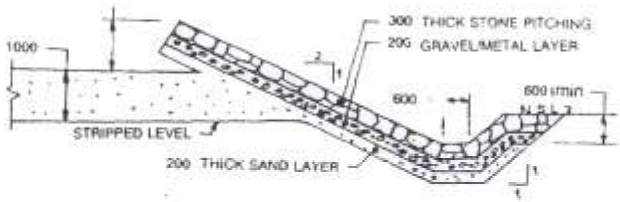
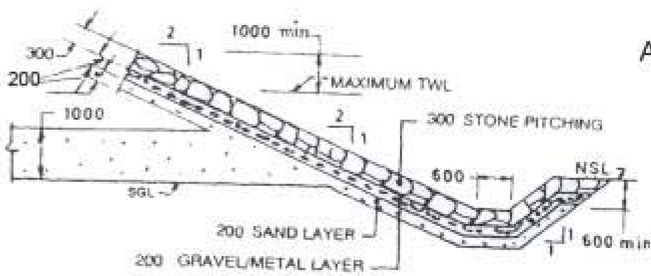


FIGURE 40. Details of rock toe protection with toe drain where TWL is higher than SGL



All dimensions in mm

FIGURE 41. Details of pitching with toe drain where TWL is higher than SGL



All dimensions in mm

FIGURE 42. Details of rock toe and pitching with toe drain where TWL is higher than rock toe

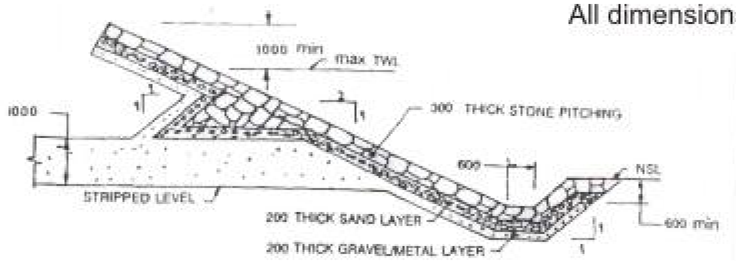


FIGURE 43. Details pitching with toe drain where SGL is above TWL

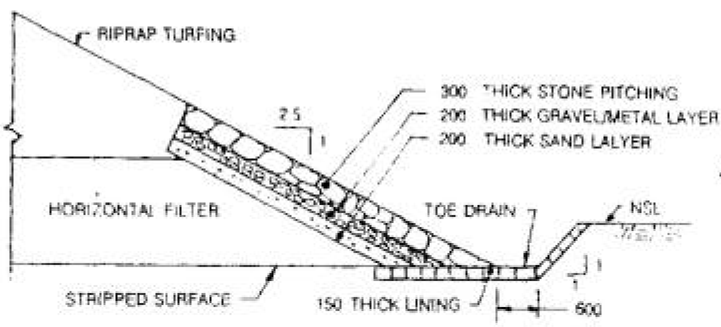


FIGURE 44. Details of pitching and lined toe drain

Toe Drain

Toe drain is provided at the downstream toe of the earth/rockfill dam to collect seepage from the horizontal filter or inner cross drains, through the foundation as well as the rain water falling on the face of the dam, by suitable means according to site conditions. Additional longitudinal drain and cross drains connected with the toe drain are sometimes provided where outfall conditions are poor. It is preferable to provide the toe drain outside the toe of rock toe, to facilitate visual inspection. The section of the toe drain should be adequate for carrying total seepage from the dam, the foundation and the expected rain water.

Details of the above measures required for seepage control within the body of an embankment dam may be had from the Burean of Indian Standards Code IS: 9429-1999 "Drainage system for earth and rockfill dams-code of practice".

For the control of seepage below the dam, through the foundation, the Bureau of Indian Standards Code IS: 8414-1977 "Guidelines for design of under-seepage control measures for earth and rockfill dams mention a number techniques. The provision of seepage control in the foundations, as in the body of the dam, is required to control the loss of water to an amount compatible with the purpose of the project, and the elimination of the possibility of a failure of the structure by piping. Many dams have been in successful service for decades in spite of losses of water. Therefore, the first step in rational design of seepage control measures is to estimate the largest quantity of water that may escape if no attempt is made to intercept percolation through the foundation. In many instances, it would be found that interception of the most conspicuously pervious zones would be sufficient. Sometimes, the reservoir bottom may have to be made impervious to reduce the amount of water seeping into ground. In addition, relief wells may be used at downstream to release the building up of excess pore pressure. These methods are described in the following paragraphs.

Positive Cutoff Trench

The positive cutoff trench (Figure 45) consists of an impervious fill placed in a trench formed by open excavation into an impervious stratum. Grouting of the contact zone of the fill and the underlying strata constitutes an integral part of the positive cutoff. Pockets of such size that compaction equipment cannot be operated and pot holes with overhangs should be filled with concrete.

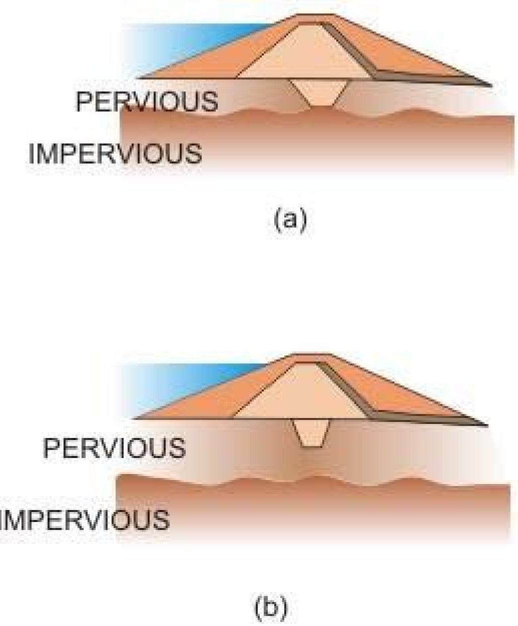
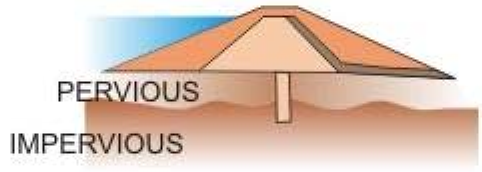


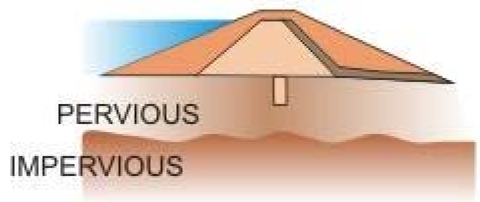
FIGURE 45. Cut off trenches
(a) Positive cut-off
(b) Partial cut-off

Concrete Diaphragm

A single diaphragm or a double diaphragm may also be used for seepage control (Figure 46). Concrete cutoff walls placed in slurry trench are not subject to visual inspection during construction, therefore require special knowledge, equipment and skilled workmen to achieve a satisfactory construction.



(a)



(b)

FIGURE 46. Concrete diaphragm
(a) Complete
(b) Partial

Grout Curtain

Grout Curtain in Pervious Soils: Grouted cutoffs are produced by injection, within the zone assigned to the cutoff, of the voids of the sediments with cement, clay, chemicals, or a combination of these materials. An essential feature of all grouting procedures is successive injection, of progressively finer pockets in the deposit. Inasmuch as grout cannot be made to penetrate the finer materials as long as more pervious pockets are available, the coarser materials are treated first, usually with the less expensive and thicker grouts, whereupon the finer portions are penetrated with less viscous fluids.

Grout Curtain in Rock: Grout curtain in rock admit of routinized treatment if the purpose is only to block the most pervious zones. These can be treated by cement grout with suitable admixtures. Concentrated seepage would generally develop at the base of the positive cutoff. This zone is particularly vulnerable when a narrow base width is used for the cutoff trench in relation to the height of the dam. The depth of the grouted zone would be dependent on the nature of the substrata and their vulnerability to subsurface erosion.
Details about the method of grouting may be had from Bureau of Indian Standards code IS: 11293(Part1)-1985 “Guidelines for the design of grout curtails: Earth and rockfill dams”. An indicative illustration of grout curtain is shown in Figure 47.

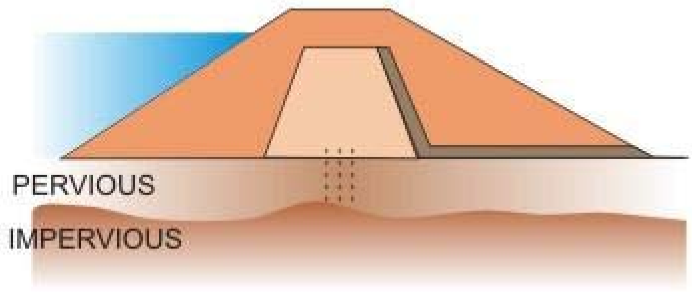


FIGURE 47. Grout curtain

Slurry Trench Cutoff Walls

A backhoe or dragline excavates a trench through the pervious deposits down to suitable impervious materials. A bentonite slurry, retained in the trench above the existing ground-water level, prevents the trench walls from caving. After a sufficient length of trench has been excavated and the bottom suitably prepared, back filling begins.

The physical characteristics of the backfill are specially controlled; in general, the backfill should be well-graded, impermeable in place, and sufficiently coarse to minimize post construction settlements. A selected amount of bentonite slurry may be blended with the backfill to improve its properties. The embankment should be suitably designed to resist cracking by differential settlement due to the slurry trench.

Steel Sheet Piles

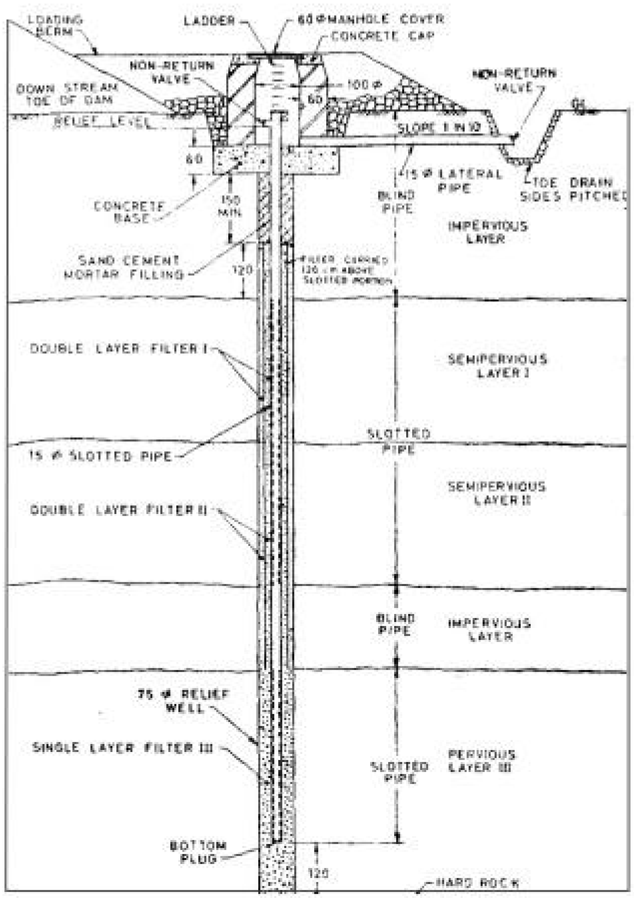
Sheet piles are useful as barrier to arrest internal erosion. But they have proved to be rather ineffective as a positive means of controlling seepage through pervious deposits. Even if sheet pile cutoffs are intact they are not water-tight because of leakage across the interlocks. In addition the locks may break because of defects in the steel or when a pile hits an obstacle. Once the lock is split, the width of the gap increases rapidly with increasing depth and may assume dimensions of a few meters.

Upstream Impervious Blanket

If a positive cutoff is not required, or is too costly, an upstream impervious blanket combined with relief wells in the downstream section may be used. Filter trenches supplement relief wells in heterogeneous deposits and in zones of seepage concentrations. An upstream blanket may result in major project economies, particularly if the only alternative consists of deep grout curtains or concrete cutoff walls. Since alluvial deposits in river valleys are often overlain by a surface layer of relatively impervious soils, it is advantageous if this natural impervious blanket can be incorporated into the overall scheme of seepage control.

Relief Wells

Relief wells are an important adjunct to most of the preceding basic schemes for seepage control. They are used not only in nearly all cases with upstream impervious blankets, but also along with other schemes, to provide additional assurance that excess hydrostatic pressures do not develop in the downstream portion of the dam, which could lead to piping. They also reduce the quantity of uncontrolled seepage flowing downstream of the dam and, hence, they control to some extent the occurrence and/or discharge of springs. Relief wells should be extended deep enough into the foundation so that the effects of minor geological details on performance are minimized. It is necessary to note the importance of continuous observation and maintenance of relief wells, if they are essential to the overall system of seepage control Details of relief wells may be had from Bureau of Indian Standards code IS: 5050-1968 “Code of Practice for design, construction and maintenance of relief wells”. A view of the relief well is shown in Figure 48.



All dimensions in mm

FIGURE 48. A typical relief well

4.7.5 Seepage calculations in embankment dams

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

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

Basic computations to arrive at these two parameters are discussed in the following paragraphs

Location of phreatic line

Phreatic line, also variously also called as saturation line, top flow line, seepage line, etc. is defined as the line within a dam in a vertical plane section below which the soil is saturated and there is positive hydraulic pressure. On the line itself, the hydrostatic pressure is equal to atmospheric pressure, that is, zero gauge pressure. Above the phreatic line, there will be a capillary zone in which the hydrostatic pressure is negative (Figure 49). Since the flow through the capillary zone is insignificant, it is usually neglected and hence the seepage line is taken as the deciding line between the saturated soil below and dry or moist soil above in a dam section.

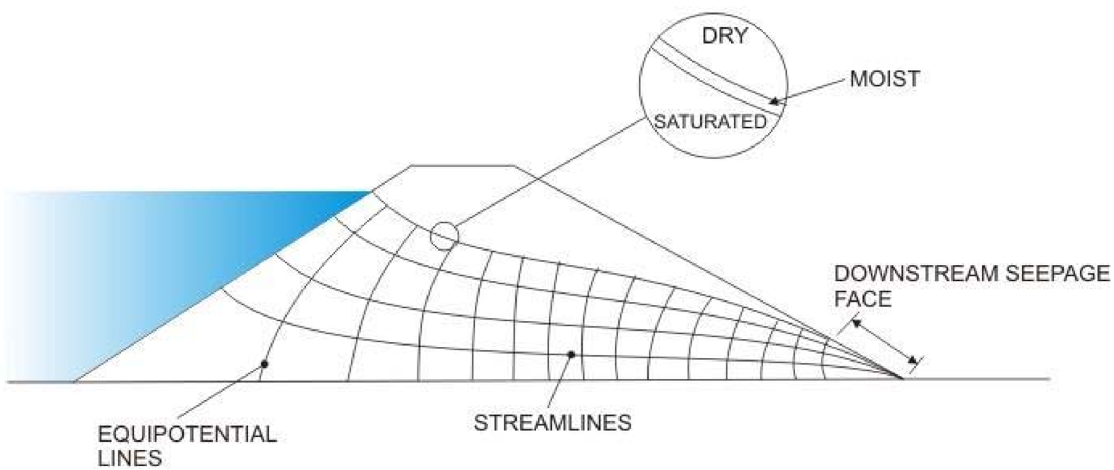


FIGURE 49. Flow net showing equipotential and streamlines for saturated flow through a homogeneous embankment dam

The flow of the seepage water below the phreatic line can be approximated by the Laplace Equation:

$$\frac{\partial^2 \phi}{\partial x^2} + \frac{\partial^2 \phi}{\partial z^2} = 0 \qquad (1)$$

where ϕ is the potential head, and x and z are the coordinates in the horizontal and vertical dimensions, respectively

As may be observed from Figure 48, the streamlines are perpendicular to the equipotential lines, that is, lines joining points with equal potential head ϕ. The upstream face of the embankment dam, conforms to one equipotential.
Surface, since for any point along the face, the sum of hydrostatic head measured below the water surface and the potential head measured below the water surface and the potential head measured above any horizontal datum is the same. The embankment section shown in Figure 48 is homogeneous, which is used rarely in practical dam construction except for small bunds and levees. In the following paragraphs, methods to locate the phreatic line for two practical dam sections would be treated.

Homogeneous dam with horizontal drainage filter

This embankment section is shown in Figure 50. The point Q is located at the upstream face corresponding to the top surface of horizontal drainage of blanket. The point P is the point on the upstream face where it meets the reservoir water. The horizontal distance between P and Q is termed as L, as shown in the figure. Another point P_0 is located on the reservoir surface that is 0.3L from the point P.

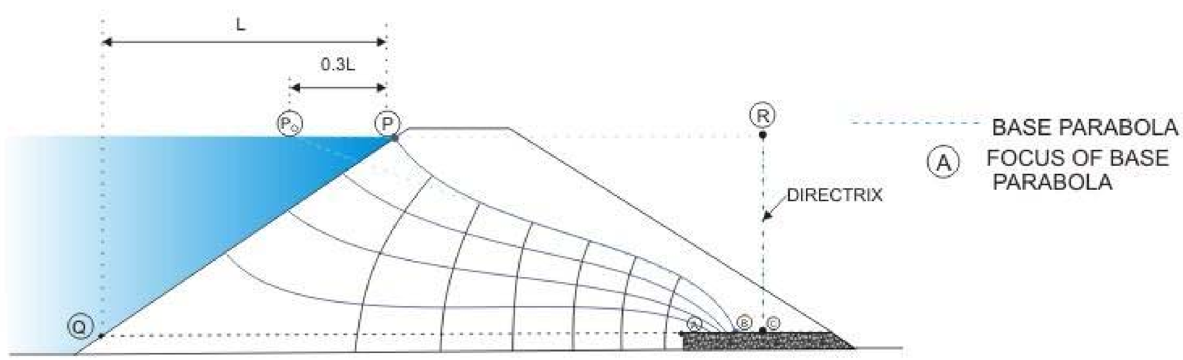


FIGURE 50. Determination of phreatic line for homogenous dam with horizontal drainage blanket

It is assumed that the phreatic line which emanates at P, meets the horizontal drainage blanket at B and is, for most of its downstream part, a parabola (first proposed by A. Casagrande). This curve is termed as the Base Parabola and is assumed to have its focus at A, the upstream edge of the horizontal drainage blanket. The Base Parabola, on its upstream part is assumed to meet the reservoir water surface at a point P_0 that is 0.3L upstream of P, as shown in Figure 49. In order to obtain the Base Parabola, one has to consider P_0 as the centre, and draw an arc A-R, with the radius equal to P_0-A. The point R is on a horizontal line at the same elevation of the reservoir surface. From point R, a perpendicular is dropped on to the top surface of the horizontal drainage blanket to meet it at a point C.

Knowing the focus, the directrix and the point P_0, a parabola can be drawn, which gives the Base Parabola shape. It may be recalled that point B is mid way of points A and C. At its upstream point, however, the parabola has to be modified such that it takes a curve upwards and meets the point P with the gradient of the phreatic line being perpendicular to the dams upstream face.

Homogeneous dam with rock toe

An embankment having a rock toe is shown in Figure 51. The Base Parabola may be drawn in much the same way by considering point A as the focus. The upstream face of the rock toe may be at any angle, say α, to the horizontal. Accordingly, an appropriate value of $\left(\frac{\Delta a}{a+\Delta a}\right)$ has to be read from the curve shown in Figure 52, which is used to determine the point of attachment of the actual seepage line with upstream face of rock toe at point D, as shown in the figure. For α less than 30^0, the above method has to be determined by a slightly modified technique. However, since rock toes with such angles are not practically provided, they are not considered here.

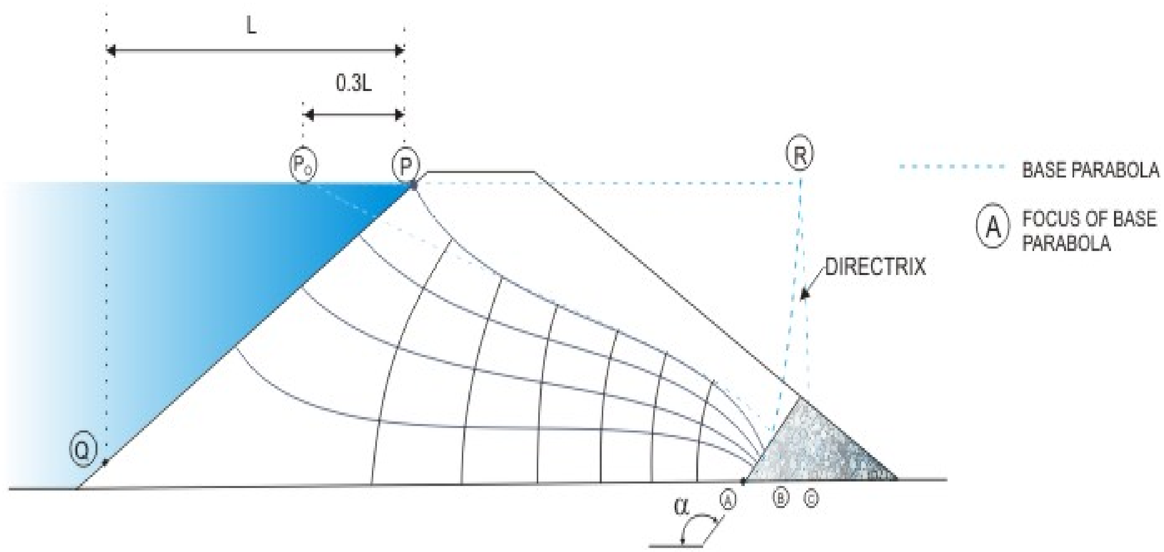


FIGURE 51. Determination of phreatic line for homogenous dam with rock toe

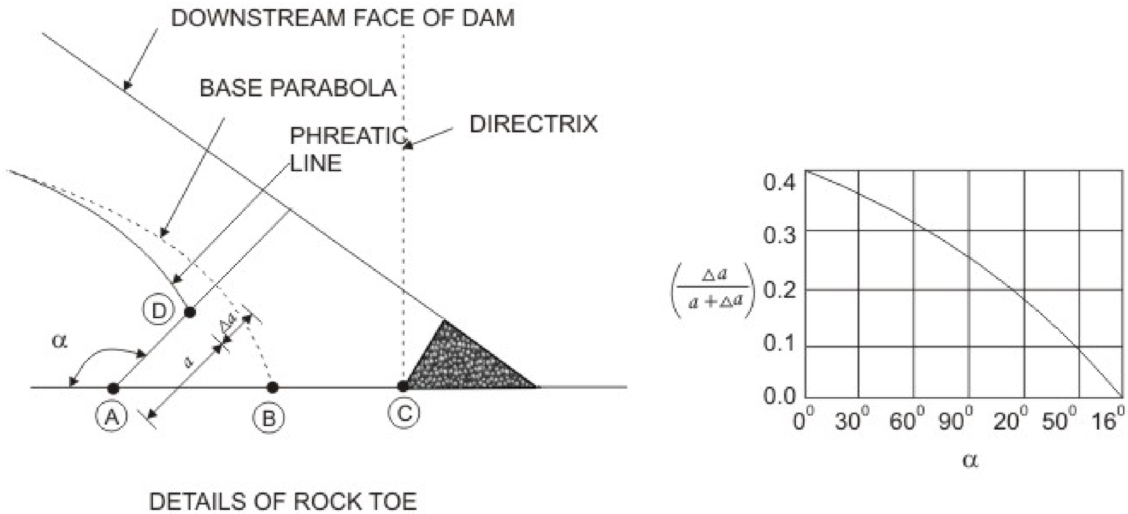


FIGURE 52. Determination of point of attachments (D) of phreatic line and upstream face of rock toe

Quantity of seepage flow

An embankment dam should be so designed that the quantity of water seeping through it is not excessive, otherwise the reservoir water would be drained off within some time. Hence, it is essential for the engineer to calculate the quantity of water seeping out through the dam. One method of calculating this is by drawing flownets, as shown in Figures 48, 49 and 50. Flownets can be drawn by trial and error, with the equipotential lines being approximately at right angles to the stream lines. The saturated zone through the embankment dam is then seen to be divided into a number of flow net boxes, bounded by two streamlines and two equipotential lines. If **h** is the total hydraulic head and $\mathbf{N_d}$ is the number of potential drops, then the drop in potential head (**Δh**) per drop is given as:

$$\Delta h = \frac{h}{\mathrm{N}_d} \tag{2}$$

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 ***Δh/l***

The discharge passing through two streamlines of the field (Δq) is given as

$$\Delta q = \mathrm{K} \cdot \frac{\Delta h}{l} \cdot b \tag{3}$$

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 $\mathbf{N_f}$ is the total number of flow channels, then the seepage per unit width of the embankment (q) is given as

$$q = \sum \Delta q = K \cdot \frac{h}{N_d}\left(\frac{b}{l}\right) \cdot N_f = K \cdot h \cdot \left(\frac{b}{l}\right) \cdot \left(\frac{N_f}{N_d}\right) \qquad (4)$$

The above calculations would give the quantity of water seeping through the body of the dam. Similar calculations have to be done for the quantity of seepage taking place through the foundation.

4.7.6 Stability calculation

For an embankment dam the most important cause of failure is sliding. It may occur slowly or suddenly and with or without any prior warning. Such a failure causes a pportion of the earth or rockfill to slide downwards and outwards with respect to the remaining part generally along a well-defined slide surface (Figure 53). As may be observed, the profile of the slide surface may be nearly approximated by circular arcs (Figure 53 a, b) or by wedges (Figure 53 c, d). The upstream failure surfaces in all cases may be possible, among others, during sudden drawdown of the reservoir level from elevation I to II, shown in the figures. At the time of the failure, the average shearing resistance all along the sliding surface. It is, therefore, necessary that the designers take special care to estimate the possibility of such a failure.

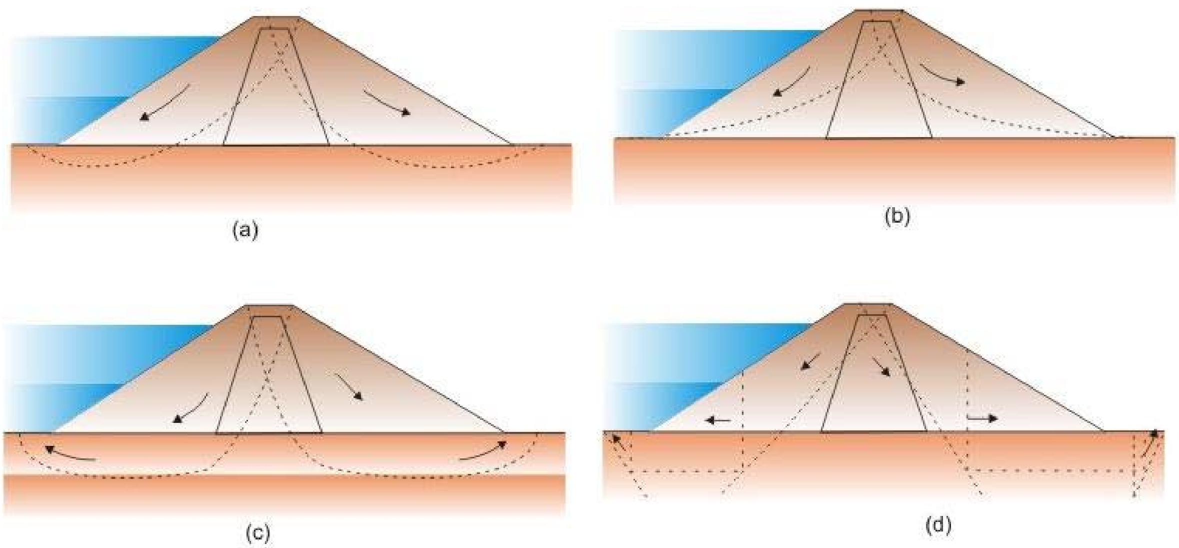


FIGURE 53. Possible modes of failure of embankment dams
(a) Homogeneous foundation
(b) Hard foundation
(c) Layered foundation
(d) Approximation of failure surfaces of(c)

The slope stability methods generally employed to analyse the failure modes are two, depending upon the profile of the assumed failure surface. These are:

a) Circular Arc method, and
b) Sliding Wedge method

In the Circular Arc method of analysis, the surface of rupture is assumed as cylindrical or in the cross section by an arc of a circle. This method, also known as the Swedish Slip Circle method, is generally applicable for analysing slopes of homogeneous earth dams and dams resting upon thick deposits of fine grained materials.

In circular arc method of analysis, the sliding mass is divided in to a convenient number of slices (Figure 54a). Each slice is assumed to act independently of its adjoining slices and the forces acting on the sides of a slice have no influence on the shear resistance which may develop on the bottom of the slice.

The Sliding Wedge method of analysis is generally applicable in the circumstances where it appears that the failure surface may be best approximated by a series of planes rather than a smooth continuous curve. This method is generally applicable under the following two circumstances:

a) Where one or more horizontal layers of weak soil exists in the upper part of the foundation, and

b) Where the foundation consists of hard stratum through which failure is not anticipated and the dam resting on it has a core of fine grained soil with relatively large shells of dense granular material.

In sliding wedge method of analysis, the trial sliding mass is divided into two or three segments (Figure 54b). The top segment is called the active wedge and the bottom segment is called the passive wedge. The middle wedge in case of a three wedge system is called the central block. The resultant of the forces acting on the active wedge and the passive wedge are first determined. These resultants acting on the central block along with other forces on the block shall give a closed polygon of forces for stability.

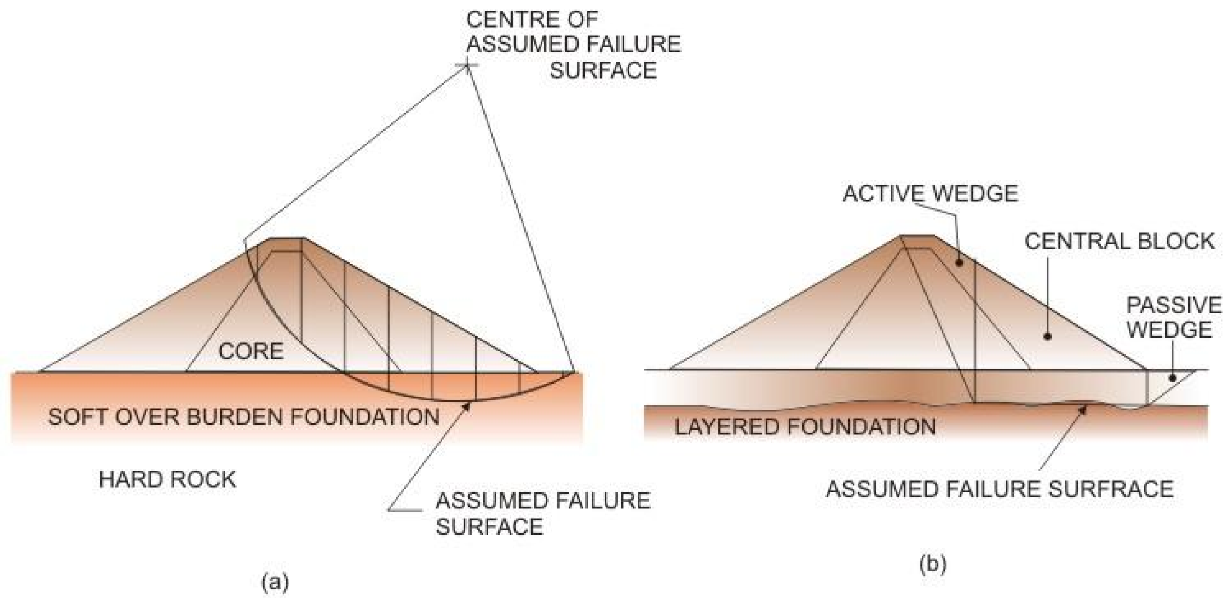


FIGURE 54. Stability analysis techniques for earth dams ; (a) Swedish ship circle method; (b) Sliding wedge method

The details about the two methods of stability analysis for earthen embankment dams is given in the Bureau of Indian Standards Code IS: 894-1975 “Code of practice for stability analysis of earth dams”.

An earth dam has to be safe and stable during all phases of its construction and operation of the reservoir. Hence, the analyses have to be carried out for the most critical combination of external forces which are likely to occur in practice. The following conditions are usually considered critical for the stability of an earthen embankment dam:

- Case I - Construction condition with or without partial pool: Check stability of upstream and downstream slopes
- Case II - Reservoir partial pool: Check stability of upstream slope
- Case III - Sudden drawdown: Check stability of downstream slope
- Case IV - Steady seepage: Check stability of downstream slope
- Case V - Steady seepage with sustained rainfall: Check stability of upstream and downstream slopes

The above critical conditions are explained below. For further details, one may refer to IS: 7894-1975 “Code of practice for stability analysis of earth dams”.

Case I - Construction Condition With or Without Partial Pool (for Upstream and Downstream Slopes)

This represents a situation when the structure is just constructed. In this condition the pore pressures developed as a result of dam material compression due to the overlying fill are not dissipated or are only partly dissipated. If the rate of raising of dam is less than about 15 meters per year, this condition may not become critical as the residual pore pressure in the dam and foundation are expected to be negligible except in highly clayey foundation with high water table, for example, marshy areas.

Construction pore pressures may exceed the pore pressures likely to be developed due to the seepage from the reservoir and consequently may control the design of dam. The magnitude and distribution of these pore pressures depend primarily on the construction water content in embankment and natural water content in the foundation, the properties of the soil, rate of raising, the height of dam and the rate at which dissipation may occur due to internal drainage.

Case II - Reservoir Partial Pool (for Upstream Slope)

This condition corresponds to the initial partial pool filling in which it is assumed that a condition of steady seepage has developed at the intermediate stages. The stability of upstream slope shall be investigated for various reservoir levels on upstream, usually levels corresponding to one-third to two-thirds height of head of water to be stored at full reservoir level and minimum value of factor of safety worked out. The analysis should account for reduction, if any, in the effective normal stresses where the pore pressures developed during construction are not dissipated before a partial pool condition can develop.

All the zones above phreatic line drawn for upstream water level under consideration should be considered as moist for working out resisting and driving forces and zones below it should be taken with their submerged weights for working out both resisting and driving forces.

Partial pool condition may not prove to be critical for all earth dams and hence analysis for this condition needs to be carried out only in cases where it is considered necessary. This condition is likely to be critical in cases of high dams where the range of drawdown is small as compared to the height of dam.

Case III - Sudden Drawdown (for Upstream Slope)

Earth dams may get saturated due to prolonged higher reservoir levels. Sudden drawdown condition corresponds to the subsequent lowering of reservoir level rate faster than pore water can dissipate. This induces unbalanced seepage forces and excess pore water pressures. This condition becomes critical if the materials of the upstream portion of the dam are not freely draining.

Depending upon the value of the coefficient of permeability of the upstream shell material, the pore pressures in the shell material in the drawdown range shall be allowed arbitrarily in the analysis as follows:

a) Full pore pressures shall be considered if the coefficient of permeability is less than 10^{-4} cm/s.

b) No pore pressures shall be considered if the coefficient of permeability is greater than 10^{-2} cm/s.

c) A linear variation from full pore pressures to zero pore pressures shall be considered for the coefficients of permeability lying between 10^{-4} cm/s to 10^{-2} cm/s.

The above values of pore pressures are based on a drawdown rate of 3 m/month.

For the core material which is generally impervious full pore pressures shall be allowed for the core zone lying in the drawdown range. If a zone of random material with the properties intermediate between core and the shell material is provided in between upstream shell and core of the dam, the pore pressures for sudden drawdown condition shall be allowed for in the same way as for the core.

Case IV - Steady Seepage (for Downstream Slope)

The condition of steady seepage is developed when the water level is maintained at a constant level for sufficiently long time and the seepage lines arc established in the earth dam section. This condition is likely to be critical for the downstream slope. In the analysis, existence of tail water and drawdown effects, if any, shall also be taken into account. The stability of downstream slope shall be examined by effective stress method. Steady seepage from level in the reservoir which is sustained for a period of one month should be taken as critical.

The stability analysis of the earth dam shall be done assuming that the dam is fully saturated below phreatic line. Allowance for pore pressure in the analysis shall be made in terms of the buoyancy of the material or by drawing flow nets. The core material lying below the phreatic line (and above the tail water level, if any) shall be considered as saturated for calculating the driving forces and buoyant for resisting forces. All the zones of the dam and foundation lying below the tail water level, if any, shall be considered as buoyant for calculating the driving and resisting forces. A part of upstream pervious shell material below the phreatic line, if any, included in trial sliding, mass shall be considered as saturated for calculating the driving and buoyant for resisting forces. All zones lying above the phreatic line shall be considered as moist for calculating both the driving and resisting forces.

Case V - Steady Seepage with Sustained Rainfall (for Downstream Slope)

Where there is a possibility of sustained rainfall, the stability of the downstream slope shall be analysed on arbitrary assumption that a partial saturation of shell material due to rainfall takes place. Accordingly for this condition of analysis, the shell and other material lying above the phreatic line shall be considered as moist for calculating driving forces and buoyant for resisting forces.

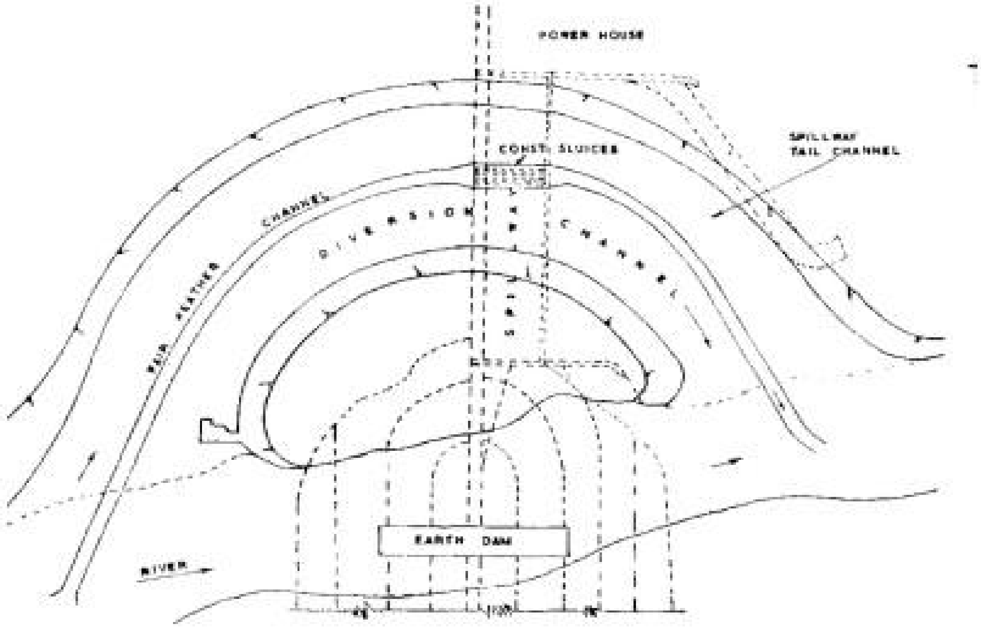
Earthquake Condition

In the regions of seismic activity, stability calculations of the slope of an embankment dam has to include earthquake forces also because they reduce the margin of safety or may even bring about the collapse of the structure. General design approach for earthquake forces is given in the Bureau of Indian Standards Code IS: 1893-1975/2002 "Criteria for earthquake resistant design of structures". Where the analysis is carried out by the circular arc or sliding wedge method, the total weight of the sliding mass considered for working out horizontal seismic forces has to be based on saturated unit weights of the zones below the phreatic line and moist weights above it. If the zone above the phreatic line is freely draining, drained weights shall have to be considered for that zone.

4.7.7 Construction of embankment dams

River diversion

As was discussed for concrete dams in Lesson 4.6, arrangements have to be made to divert the river while constructing an embankment dam. This temporary exclusion of river flow is necessary to provide dry or semi-dry area for the work to continue. However, for concrete dams, after the initial levels in touch with the foundation have been built, the river water during floods may be allowed to overtop the partially completed structure during monsoons. In case of embankment dam, it is rarely allowed as overtopping would generally lead to a washout of the downstream face first and the rest follows the collapse. Hence, for embankment dams, the diversion measures require the detouring of the whole flood water through bye-pass. In only special cases, has an embankment been constructed that has been allowed to overtop only when partly constructed. The downstream face has to be sufficiently protected in that case with concrete blocks or gabions. Details of river diversion for dams may be found in the Bureau of Indian Standards Code IS:10084-1984 (Part 2) "Design of diversion works-Criteria" from where the following has been called. Figure 55 shows a typical example of a diversion channel for earth/rockfill dam project in a narrow river. The layout and principal dimensions, specifically the cross-section of the diversion channel is governed by several considerations such as topography, volume of flood to be handled, water levels during passage of monsoon and non-monsoon floods in consonance with raising of the dam and requirement of excavated material from the diversion channel for use in constructing the earth dam, etc. The coffer dams in such a case form an integral part of the earth and rockfill dam in the finally completed stage, and are also not allowed to be overtopped. Because of the considerable expenditure and time involved in the construction of diversion channel for earth dams, these channels are designed to be useful for other purpose also such as spillway tail channel or power house tail channel. Although, initially such channels may be without protective lining on the sides, they are protected at a subsequent stage when utilized for spillway or power house tail race channel.



In a wide river channel, provided the height of the earth dam is small enough, diversion could be managed by a temporary channel revolving a gap through the earthfill dam while the remainder of the embankment is being constructed (Figure 56). Before the stream is diverted, the foundation required for the dam should be completed in the area where the temporary opening will be left through the embankment. This preparation would include excavation and refilling of a cut-off trench, if one is to be constructed. The stream is then channelised through this area after which the foundation work in the remainder of the streambed is completed.

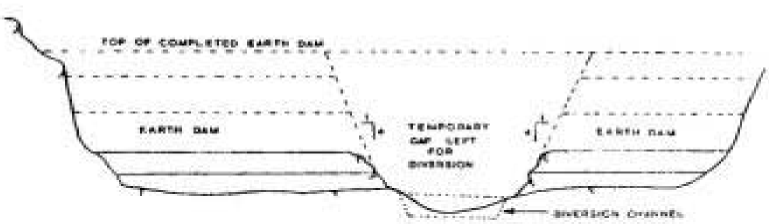


FIGURE 56. Diversion through a gap in a partially constructed earth dam in a wide river

In some rivers, the floods may be so large that provision of diversion channels even for average floods may be highly expensive. The only alternative then is to have the discharge passed through a conduit excavated through one or either abutment. Coffer dams, nevertheless, have to be constructed on the upstream and downstream of the working area to divert the stream flow into the diversion tunnel and to prevent the water on the downstream side of the river from flooding the work space.

Foundation preparation

The principles of foundation treatment for concrete gravity dams have been outlined in Lesson 4.6, Similar methods have to be adopted for embankment dams, too. However, the Bureau of Indian Standards code IS: 11973-1986 “Code of practice for treatment of rock foundations, core and abutment contacts with rock, for embankment dams” deals specifically with the requirement of an embankment dam. Some important points from this standard are explained below.

Basically, the surface under the entire core and under a portion of the upstream filter and downstream transition zone shall be completely excavated to such rock as will offer adequate resistance to erosion of fines in the core.

All loose or semi-detached blocks of rock should be removed. The quality of rock shall be judged characteristic of core material. Rock of ‘Lugeon’ values in percolation test within 10 (Ten) will generally be free of cracks larger than 0.025 mm. Erosion of fines from core materials commonly used would not occur through such cracks. Grouting may be necessary to bring down ‘Lugeon’ values to above allowable limits in the contact zone.

The amount of care required in treating the rock surface is controlled by the character of the core material. If the core material is resistant to piping, especially if it contains considerable coarse material with adequate proportion of sand, surface treatment is less demanding than if the core material is susceptible to piping; for example, a fine silty sand and very lean clays. In the latter case, extreme care should be taken and the core material should be placed only after very careful inspection of the treated surface. For dispersive clays, special precautions, such as protection by filter fabric or plastic concrete may be required.

Small ribs and similar irregularities should be filled with plastic concrete to produce slopes not steeper than about 1:1 where the difference in elevation is a few centimeters to a meter or so. Surface treatment in this fashion should extend upstream to approximately the mid-point of the upstream filter and downstream at least 0.6 to 0.9 m beyond the downstream edge of the fine filter. In particularly adverse situations, such as where there are joints wider than the coarser particles of the filter, surface treatment as described may be necessary under the entire transition zone.

The final rock surface should have smooth contours against which soil can be compacted by heavy equipment. Hand compaction is generally unsatisfactory and it is advisable to place plastic concrete in core contact areas of conduit trenches and other irregularities transverse to the dam axis for a width at least 0.5 H or preferably 1.0 H.

Surface treatment as described may be difficult to accomplish on steeply sloping abutments. In this case, gunite may be used for filling depressions after the cracks and joints have been cleaned and sealed. If there is extensive jointing, especially if the joints slope upward away from the face, adequate sealing of the joints may require constructing a concrete slab, which is dowelled to the rock, and then grouting through the slab.

The depth of excavation necessary in weathered rock is difficult to establish during initial design. The depth of weathering is usually very irregular, being controlled by minor variations in joint spacing and rock type. Abrupt changes in elevation of the surface of 'groutable rock' probably will be found. Overhangs, some of large size, should be anticipated.

Usual practice is to select material, preferably a plastic soil, for the first lift over the rock surface. If plastic soils are limited, the most plastic soil available should be used. Gravel or stone exceeding about 50 mm in size should be removed or excluded from the material placed in this first layer over the rock to improve compaction at the contact. The surface on which the core material is placed should be moist but free of standing water, and the material when placed should be wet of optimum. In dry climates or during dry weather, difficulty may be experienced with this first lift becoming excessively dry where it feathers out on a gentle to moderate slope. In such a case the edge of the fill should be sloped slightly downward toward the contact with the rock. Against steep rock faces or adjacent to concrete structures, sloping the fill slightly upward near the contact is desirable to provide better clearance and better compaction at the contact.

Treatment of rock defects and discontinuities

In evaluating and planning for excavation and seepage control measures, special attention shall be given to discontinuities such as faults and relief (sheet) joints, which may extend for long distance as nearly plane surfaces. Relief joints may exist naturally or may open during excavation. They are most likely to occur in deep, steep-walled valleys, especially in brittle rocks, or where high modulus rock is underlain by low modulus rock. Since they are roughly parallel to the valley wall, they may cause slides during construction. Openings of several centimeters have been observed. Control of seepage through such joints becomes a major problem. Installation of concrete cutoffs across particularly bad joints may be warranted or extensive grouting may be necessary. Drainage from such joints shall be provided.

When seams are filled with silt, clay, etc, or in faults with gauge, it is essential to excavate and backfill the seam and gauge zones in the entire core contact zone. It is advisable to excavate and backfill a further length on the upstream for a distance equal to the reservoir head and backfill it with concrete. On the downstream side the seams should be excavated and backfilled with a well designed and adequate filter again for a distance equal to the reservoir head.

Grouting

There are three main objectives in the grouting programme (see also IS : 6066-1984). These are as follows:

- To reduce the seepage flow through the dam foundation;
- To prevent possible piping or washing of fines from the core into cracks and fissures in the foundation; and

To reduce the hydrostatic pressure in the downstream foundation of the dam. The latter is generally a problem only for dams on fairly weak foundations and critical abutment configurations. This is usually accomplished in conjunction with an abutment drainage system.

To prevent possible piping of the fine core material through the foundation, blanket grouting is accomplished as determined by the rock conditions. If the core foundation of the dam consists of closely fractured and jointed rock, a blanket grout pattern is used with holes spaced at 3 m to 5 m with depths of 6 m to 10 m. If the foundation rock is massive, no blanket grouting is done. Localised area consisting of faults, fissures, or cracks are generally grouted upstream of the cutoff and sometimes downstream.

Quality Control

The performance of an earth or rockfill dam depends upon the control exercised during construction, supervision and inspection. An entirely safe design may be ruined by careless and shoddy execution. Proper quality control during construction is as important as the design. The skill, experience and judgment required of the engineer in charge of construction, is in no way lesser than that of the design engineer. Hence, the Bureau of Indian Standards has published the following publication which provides guidance for construction of embankment dams with regards to quality control IS: 14690-1999 “Quality control during construction of earth and rockfill dams-recommendations”.

4.7.8 Instrumentation

Numerous embankment dams constructed in India and abroad and the height achieved ever increasing, like the Nurek Dam of Russia (Figure 57). From the point of view of safety as well as to garner knowledge about the physical behaviour of these dams, instrumentation has been recommended all medium and large sized dams. The data obtained from these measurements also help the commonly made either explicity or implicity in an embankment dam design. In fact, it is very important to monitor the behavior of the dams under earthquake loadings and those constructed in regions of high seismic activity need to be instrumented carefully.

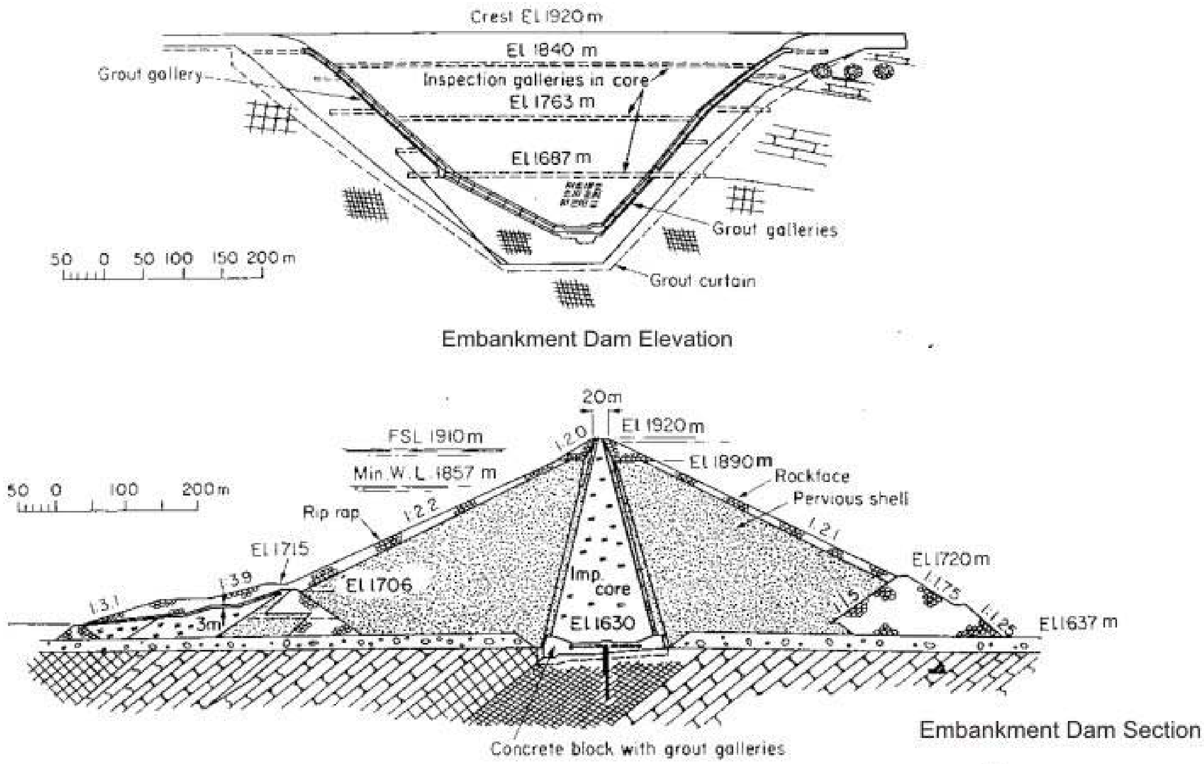


FIGURE 57. Elevation and cross section of Nurek embankment Dam,Russia

Even for the continued maintenance of any embankment dam, vertical and timely observations of the measurements taken will provide means of evaluating the behaviour of the structure and, if need be, allow the engineers to take appropriate remedial measures on the basis of the observed data. Hence, the importance of providing instruments in an embankment dam, or any dam for that matter, cannot be over emphasized. The Bureau of Indian Standards code IS: 7436 (part1)-1993 "Guide for types of measurements for structures in river valley projects and criteria for choice and location of measuring instruments (Earth and Rockfill dams)" provides guidelines for various types of instrumentation to be carried out in embankment dam. The following paragraphs highlights the salient features of the recommended measurements to be taken and the corresponding instruments that have to be installed.

Pore Pressure

The measurement of pore pressure is probably the most important and usual measurement to be made in the embankments. Their measurement enables the seepage pattern set up after impounding of reservoir to be known, the danger of erosion to be estimated, at least partially, and the danger of slides in the dam and abutments to be estimated if the reliable shear strength is known. Valuable information about behaviour during construction and drawdown is obtained.

Movements

Measurement of movements is as important as the measurement of pore pressures. Movements conforming to normal expectations are basic requirements of a stable dam. An accurate measurement of internal and external movements is of value in controlling construction stability. The measurement of the plastic deformation of the upstream and downstream slopes under the cycles of reservoir operation may indicate the likely development of shear failure at weak points.

Seepage

Measurement of seepage through and past a dam, may indicate erosion or blocking of downstream drains and relief wells, by increase or decrease of seepage, respectively at constant reservoir conditions. Seepage and erosion along the lines of poor compaction and through cracks in foundations and fills may specially be indicated by such measurements.

Strains and Stresses

Design analysis of earth and rockfill dams is based on radical simplifications of the stress pattern and the shape of the rupture planes. Stress measurements, therefore, require considerable judgement in interpretation. Accurate measurement of stress is difficult and distribution of stress in earth and rockfill dams is complex. Strains may be calculated from displacements or measured directly.

Dynamic Loads (Earthquakes)

Earthquake causes sudden dynamic loading and measurement of vibrations in dams located in areas subjected to seismicity is important for evolving design criteria for such conditions. The instruments for recording the measurements can be divided into two types: Vertical Movement Gauges and Horizontal Movement Gauges. These are explained below.

Vertical Movement Gauges

Surface Markers: Surface marker points consist of steel bars which are driven vertically into the embankment or the ground and embedded in concrete. A reference base line is established on a firm ground outside the area of movement due to reservoir and embankment load. Position of surface stakes or markers fixed on the embankment are determined by survey with reference to this line. It measures horizontal movements also. Surface markers may be established on lines parallel to the centre line of the dam at 50 to 100m centres. The lines may be at the edge of the top width of the dam, at the edge of berms or at suitable intervals along the slope, at the toe. of the dam and at 50 m and 100 m from toe if foundation soil is not firm. These may be provided both on upstream and downstream slopes except in locations on upstream slope which remain throughout the year below lake water.

Cross-Arm Installation: It consists of telescopic steel casing to which are attached horizontal cross-arms at predetermined vertical intervals. As the soil settles, sections of casing are dragged down and these are thus relocated in their new positions by lowering down the casing a problem fitted with retractable claws which engage the bottom of each section in turn or by using an electrical probe. Cross arms are used in order to eliminate any possibility of the casing sections not settling along with the surrounding soil.

Hydraulic Device: It is made from two 50 mm diameter brass pipe nipples soldered to a common diaphragm. Pipe caps are secured at both ends of the assembly which is then mounted vertically on a steel base plate for anchorage in the embankment. The diaphragm separates the upper (air) chamber from the lower (overflow) chamber and encloses a plastic float valve which prevents water from entering the air chamber during flushing of the lower chamber. Three 8-mm outer diameter plastic tubes are embedded in trenches which are excavated to maintain continuous downward slopes to the instrument terminal. The instrument terminal is equipped with a pump, air compressor and high precision pressure gauges.

Geonor Probe: It consists of a three-pronged tip connected to a double rod which is lowered down a bore hole or driven in soft ground to desired depth. When the outer rod is held and the inner rod driven with hammer, the three prongs are forced out in the surrounding soil. The outer rod is then uncovered from, the tip and withdrawn a few centimeters. The top of the inner rod, which remains in contact with the anchored tip is used as a reference point to measure the settlement of the tip. This device is particularly well suited for measuring settlements of soft foundations under-low embankments.

Foundation Settlement Measuring Device: It is a base plate placed on the foundation line with a vertical column of steel tubings. The position of the base plate is determined by a surrounding device lowered from the top open end of the steel tubings.

Magnetic Probe Extensometer: This system consists of a magnet/lead switch probe of approximately 15mm diameter connected to an indicator with a marker connecting cable. Magnetic ring markers with stainless steel spring parts are installed over a series of PVC access pipes of 33 mm outer diameter and 27 mm inner diameter jointed together. The probes when lowered through the access pipe will give indications in the indicator where the magnet marker rings are located. When settling takes place the marker rings will move with the soil and the fresh positions of the marker rings indicate the amount of settlements with respect to earlier logged position.

Induction Coil Type Extensometers: This induction coil type extensometers consist of an electrical probe made of PVC and having a diameter of 35mm or 43mm which houses a primary electrical exit. The probe is connected to an indicator electrical cable. Indicator has a volt/ammeter to measure the voltage/current increase when the primary coil enters a secondary coil, when there is a steel marker ring or plate, it will indicate a current/voltage which could be read through the indicator. Series of marker rings installed over a corrugated PVC pipe installed over a PVC access tubes or inclinometer tube should help monitoring the settlement.

Horizontal movement gauges

Cross-Arm Installation for Measurement of Horizontal Movement: This installation is similar to that described above but instead of cross-arms fixed at different sections there are two Vertical plates at the same level placed at a certain distance apart. The relative horizontal movements between the two cross-arms are measured by transmitting the same by means of a cable to a pair of counter weights, which move vertically in the tubing. A sounding probe similar to that used in measurement of vertical movement installation determines the position of the counter-weights.

Inclinometers: Plastic or aluminium tubing is placed vertically in the dam with its bottom anchored to firm unyielding stratum. The inclination of the tubing is measured by a sensitive electrical inclinometer, step by step, starting from the bottom of the tubing. Horizontal movements are computed by integrating the movements starting from the bottom, on the basis of changes in the inclination. Vertical movements may also be measured by using telescoping couplings for connecting the sections of the tubings and noting the positions of the ends of each section by a mechanical latching device, or if metal rings are embedded in the end portions of plastic tubing, by an electro magnetic device. Each section of tubing is anchored to the surrounding soil mass by fixing flanges or collars to the tubing. Alternatively, when an electromagnetic sounding device is used, the plastic tubing passes through encircling metal discs which are free to move along with the earth mass and the position of these discs are determined by the device.

Piezometers

Piezometers are installed in embankment dams to monitor the pressure of water within the soil or rock fragment. Typical installation locations of these devices are shown in Figure 58 and the details of some particular types are described below.

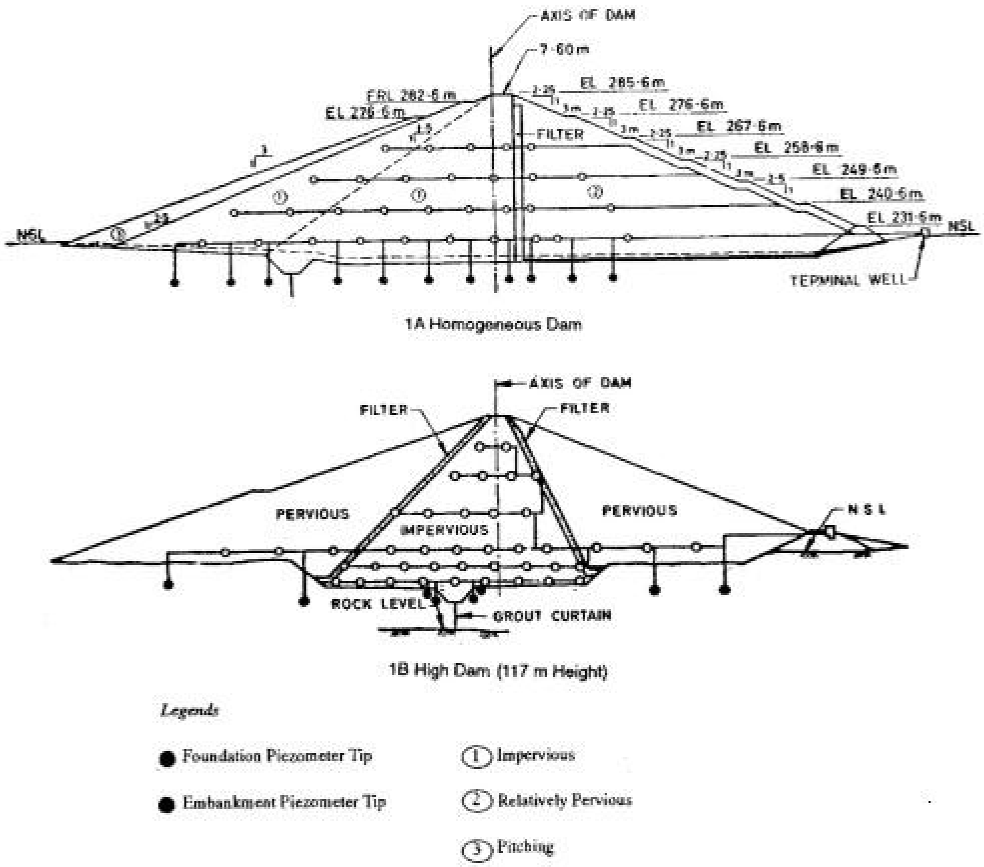


FIGURE 58. Typical installation of piezometers in embankment dams

Porous Tip/ Tube Piezometer: This is a steel or PVC pipe 10 to 40 mm in diameter placed vertically during construction or in a borehole after construction. A porous element is fixed at the bottom of the pipe or alternatively, the lower portion is perforated, and soil prevented from entering the pipe by surrounding the perforated portion by brass wire mesh and a gunny bag filled with filter material. With increase or decrease of pore water pressure in the soil near the perforated portion, water level rises or drops in the pipe and this level is noted by as electrical sounding device or a bell sounder.

Closed System Hydraulic Piezometer: It consists of a porous element which is connected by two plastic tubes to pressure gauges located in a terminal house or terminal well. The terminal house or well contains pumping and vacuum equipment, an air trap and a supply of de-aired water besides pressure gauges. Use of two plastic tubes makes possible the circulation of water through the porous element and to remove air from the system. The pore-water pressure is noted by means of gauges.

Electrical Piezometers: Electrical piezometer consists of a tip having a diaphragm which is deflected by the pore water pressure against one face. The deflection of the diaphragm is measured by a suitable strain gauge which may be suitably calibrated to read pore water pressure. The strain gauge is either electrical resistance (unbonded strain gauge) type or vibrating wire type.

Pneumatic Piezometers: In the pneumatic piezometers, the diaphragm deflection due to pore water pressure is balanced by a known air/gas pressure and recorded at the outside indicator end using pneumatic pressure gauges or pressure transducers.

Earth Pressure Cells: The usual instrument to measure earth pressure is the earth pressure cell. It uses a stiff diaphragm on which the earth pressure acts. The action is transmitted through an equalizing, confined, incompressible fluid (Mercury) on to a second pressure responsive element, the deflection of which is proportional to the earth pressure acting. The deflection is transformed into an electrical signal by a resistance wire (unbonded strain gauge) or vibrating wire strain gauge and transmitted through a cable embedded in the earth work to a receiver unit on the surface. The measure of the electrical signal indirectly indicates the earth pressure by appropriate calibration.

Instruments for measuring effects of dynamic loads due to earthquake include seismographs, accelerographs, and structural response records, details of which may be had from the Bureau of Indian Standards code IS: 4967-1968 “Recommendations for seismic instrumentation for river valley projects”.