Soil and Water Conservation Structures

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Module 1. Perspective on Soil and Water Conservation

Lesson 1. Soil Erosion

1.1 Introduction

Soil erosion is the process by which soil is removed from the Earth's surface by exogenetic processes such as wind or water flow, and then transported and deposited in other locations. In general, soil erosion implies the physical removal of topsoil by various agents, including rain, water flowing over and through the soil profile, wind, glaciers or gravitational pull. Land and water are the most precious natural resources that support and sustain the anthropogenic activities. In India particularly, about 65% of the population depend on agriculture, the only sector which generates half of the employment and maintain ecological balance.

Soil erosion in India is amongst the leading areas of concern as it affects cultivation and farming in the country in adverse and unfavorable ways. Soil erosion leads to deprivation of physical characteristics of soils and damages plant and crops. In India almost 130 million hectares of land, i.e., 45 % of total geographical surface area, is affected by serious soil erosion through gorge and gully, shifting cultivation, cultivated wastelands, sandy areas, deserts and water logging. Soil erosion by rain and transportation of soil particles through rivulets that takes place in hilly areas causes severe landslides and floods. The anthropogenic activities including cutting trees for agricultural implements, firewood and timber; grazing by a large number of livestock over and above the carrying capacity of grass lands, traditional agricultural practices, construction of roads, indiscriminate quarrying and other activities, have all led to the opening of top surfaces to extreme soil erosion. In Indian condition, the control of soil erosion is a challenging task in the sense that the onset of monsoon often coincides with the *kharif* sowing and transplanting. In this stage of kharif crop when canopy cover is minimal, major part of the land is exposed to the rainfall let the land prone to soil erosion. It is prudent to check soil erosion from agricultural lands since it affects majority of people.

1.2 Types of Soil Erosion

Broadly, soil erosion can be divided into three categories depending on the eroding agents namely water erosion, wind erosion and chemical or geological erosion. Soil erosion due to the agents like water and wind is mostly prevalent and tangible. The erosion caused through chemical and geological agents is a slow process and continues to years and often it is non-tangible. Water erosion is further subdivided into classes depending on the effect of water erosion. These include sheet erosion, rill erosion, gully erosion, land slide or slip erosion and stream bank erosion.

1.2.1 Geologic Erosion

Geologic erosion sometimes referred to as natural or normal erosion; represent erosion under the cover of vegetation. It includes soil as well as soil eroding processes that maintain the soil in favorable balance, suitable for the growth of most plants. The rate of erosion is so slow that the loss of soil is compensated by the formation of new soil under natural weathering processes. The various topographical features such as existing of streams, valleys, etc. are the results of geologic erosion.

1.2.2 Wind Erosion

Wind erosion is the detachment, transportation and redeposition of soil particles by wind. A sparse or absent vegetative cover, a loose, dry and smooth soil surface, large fields and strong winds all increase the risk of wind erosion. Air movement must attain a certain velocity (with enough speed to generate visible movement of particles at the soil level) before it can generate deflation and transport of particles. Winds with velocities of less than 12-19 km/hr seldom impart sufficient energy at the soil surface to dislodge and put into motion sand-sized particles. Drifting of highly erosive soil usually starts when the wind attains a forward velocity of 25-30 km/hr. Wind erosion tends to occur mostly in low rainfall areas when soil moisture content is at wilting point or below, but all drought-stricken soils are at risk. Often the only evidence of wind erosion is an atmospheric haze of dust comprising fine mineral and organic soil particles that contain most of the soil nutrients. Actions to minimize wind erosion include improving soil structure so wind cannot lift the heavier soil aggregates; retaining vegetative cover to reduce wind speed at the ground surface; and planting windbreaks to reduce wind speed. Also, be ready for severe wind erosion seasons which tend to be the summers following dry autumns and winters. The most familiar result of wind erosion is the loss of topsoil and nutrients, which reduces the soil's ability to produce crops. Topsoil loss can be seen as rocky or gravelly knolls, thin soils mixed with lighter coloured subsoil, or the presence of calcium carbonate in surface soils.

Soil productivity is affected by wind erosion in various ways. Areas of erosion and deposition within a field increase the variation in soil characteristics, requiring more costly and less efficient soil management practices. Wind removes the smaller clay particles and organic matter from the soil while coarser materials are left behind. The continued loss of fine particles reduces soil quality. In shallow soils and soils with a hardpan layer, wind erosion also results in decreased root zone depth and water-holding capacity. Such changes may occur slowly and go unnoticed for many years especially if mixing by tillage masks the effects of wind erosion.

1.2.2.1 Process of Wind Erosion

The process of wind erosion comprises of three basic stages namely saltation, suspension and surface creep. Fig. 1.1 describes the process.
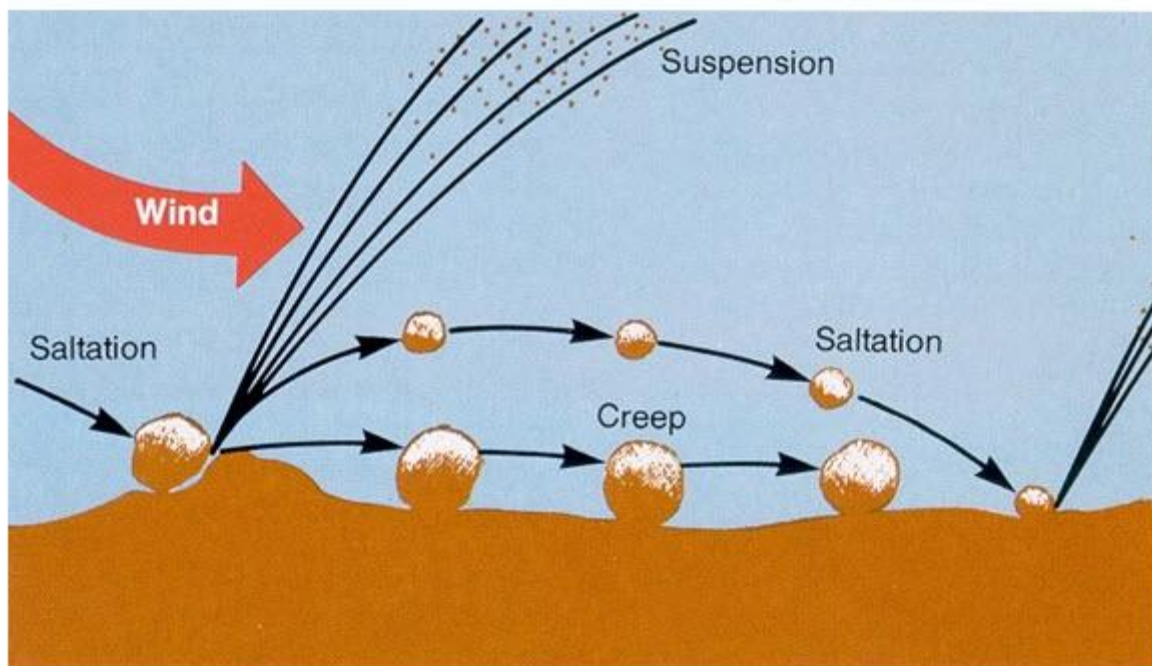
Fig.1.1. Process of wind erosion.

(Source:http://www.weru.ksu.edu/new_weru/images/CreepSaltSusp.jpg: accessed on 2 October 2013.)

(i) Saltation: Saltation occurs when the wind lifts larger particles off the ground for short distances, leading to sand drifts. Fine and medium sand-sized particles are lifted a short distance into the air, dislodging more soil as they fall back to the ground.

(ii) Suspension: Suspension occurs when the wind lifts finer particles into the air leading to dust storms. Very fine soil particles are lifted from the surface by the impact of saltation and carried high into the air, remaining suspended in air for long distances.

(iii) Surface Creep: The movement of large soil particles along the surface of the soil after being loosened by the impact of saltating particles.

1.2.2.2 Extent of Wind Erosion

Several factors, other than the wind velocity itself, contribute to wind erosion. These fall into two main groups of closely interrelated elements: those inherent in the properties of the soil per se and those associated with soil cover. A rough soil structure, especially at the surface, effectively reduces the movement of soil particles. Arid regions, however, are dominated by smooth, pulverized and structureless top soils. Soil texture also influences soil erodibility; soils of fine texture are, for example, particularly susceptible to wind erosion.

Measurements of dust in the air up to three metres above the soil surface at Jodhpur, India, showed that on a stormy day the amount of dust blowing varied between 50 and 420 kg/ha. In the Jaisalmer region of India, where wind speeds generally are higher, average soil loss of 511 kg/ha was recorded.

According to Global Assessment of Human-induced Soil Degradation (GLASOD), 21.6 Mha area of Indian soil is affected by wind erosion, which account for 6% of total geographical area. However, area varied from 12.9 to 38.7 Mha from various sources. The GLASOD assessment of extent of area under wind erosion is presented in Table 1.1

Table 1.1. GLASOD assessment: areas affected by wind erosion (Unit: 1000 ha) – South Asian Countries

Country	Light	Moderate	Strong	Total	Total as percent of land
Afghanistan	1 873	0	209	2 082	5%
Bangladesh	0	0	0	0	0%
Bhutan	0	0	0	0	0%
India	0	1 754	9 042	10 796	6%
Iran	6 559	25 730	3 085	35 374	60%
Nepal	0	0	0	0	0%
Pakistan	3 998	6 742	0	10 740	42%
Sri Lanka	0		0	0	0%
India, dry region	0	1 754	9 042	10 796	-

India humid region	0	0	0	0	-
Dry zone	12 430	34 225	12 337	58 992	39%
Humid zone	0	0	0	0	0%
Region	12 430	34 225	12 337	58 992	18%

1.2.3 Water Erosion

The soil erosion caused by water as an agent is called water erosion. In water erosion, the water acts as an agent to dislodge and transport the eroded soil particle from one location to another (Fig. 1.2).
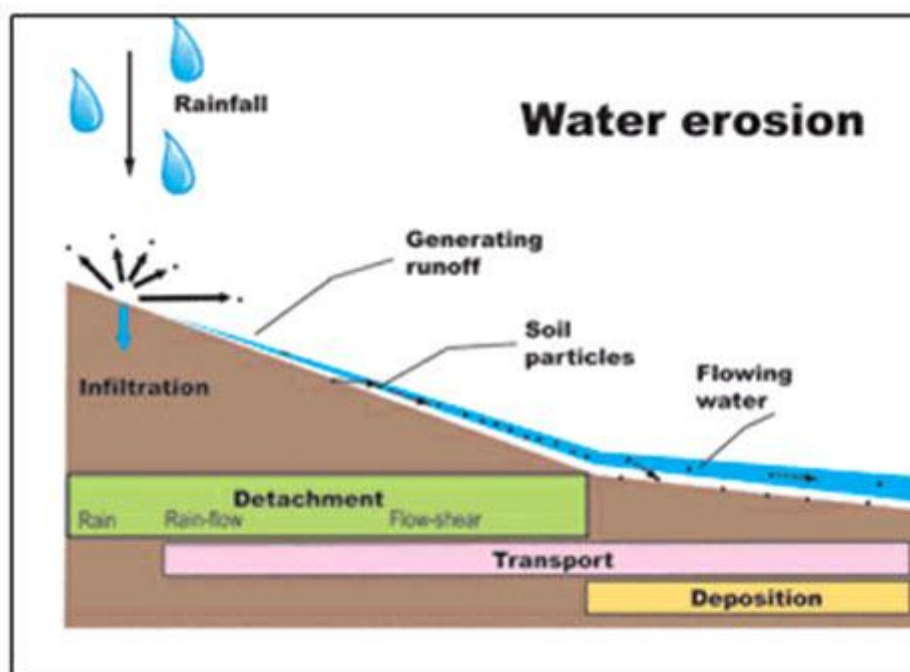
Fig. 1.2. Process of water erosion.

1.2.3.1 Extent of Water Erosion

The extent of water erosion in Indian subcontinent are presented in Table 1.2. In India, 32.8 Mha area in India is affected by water erosion which accounts for 18%

of the land. However, the water erosion extent estimated by different sources varies from 87 to 111 Mha in India.

Table 1.2.GLASOD assessment: areas affected by water erosion (Unit: 1000 ha) – South Asian region

Region	Light	Moderate	Strong	Total	Total as percent of land
Afghanistan	8 560	2 597	0	11 156	29%
Bangladesh	0	1 504	0	1 504	15%
Bhutan	36	0	4	40	10%
India	2 936	17 217	12 620	32 773	18%
Iran	14 504	11 896	0	26 400	45%
Nepal	520	1 072	0	1 592	34%
Pakistan	6 080	1 124	0	7 204	28%
Sri Lanka	72	157	845	1 074	46%
India, dry region	1 177	0	1676	2 853	-
India, humid region	1 759	17 217	10 944	29 920	-
Dry zone	30 320	15 617	1 676	47 613	32%
Humid zone	2 387	19 951	11 791	34 130	20%
Region	32 707	35 568	13 468	81 743	25%

The state wise extent of soil erosion from wind as well water is presented in Table 1.3. The Fig.s are adopted from National Bureau of Soil Survey & Land Use Planning(NBSS&LUP 2005). The NBSS&LUP estimates are higher in case of water erosion and lower in case of wind erosion when compared to GLASOD.

Table 1.3. Extent of erosion in India

Name of the States	Water Erosion (1000 ha)	Wind Erosion (1000 ha)
Andhra Pradesh	11518	0
Arunachal Pradesh	2372	0
Assam	688	0
Bihar+ Jharkhand	3024	0
Goa	60	0
Gujarat	5207	443
Haryana	315	536
Himachal Pradesh	2718	0
Jammu & Kashmir	5460	1360
Karnataka	5810	0
Kerala	76	0
Madhya Pradesh + Chhattisgarh	17883	0
Maharastra	11179	0
Manipur	133	0
Mizoram	137	0
Meghalaya	137	0
Nagaland	390	0
Orissa	5028	0
Punjab	372	282

Rajasthan	3137	6650
Sikkim	158	0
Tamil Nadu	4926	0
Tripura	121	0
Uttar Pradesh + Uttarakhand	11392	212
West Bengal	1197	0
Delhi	55	0
Union Territories	187	0
Grand Total	93680	9483
Grand Total(Million ha)	93.68	9.48

(Source: NBSS&LUP, 2005)

1.2.3.2 Types of Water Erosion

The different types of water erosion are described in the following section.

1.2.3.2.1 Splash Erosion

This type of soil erosion is because of the action of raindrop. The kinetic energy of falling raindrop dislodges the soil particle and the resultant runoff transports soil particles. Splash erosion (Fig. 1.3) is the first stage of soil erosion by water. It occurs when raindrops hit bare soil. The explosive impact breaks up soil aggregates so that individual soil particles are 'splashed' onto the soil surface. The splashed particles can rise as high 0.60 meter above the ground and move up to 1.5 meter from the point of impact. The particles block the spaces between soil aggregates, so that the soil forms a crust that reduces infiltration and increases runoff.
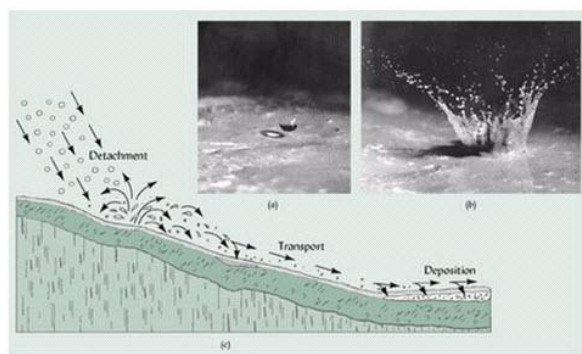

Fig. 1.3. Picture describing splash erosion of the soil.

1.2.3.2.2 Sheet Erosion

Sheet erosion (Fig. 1.4) is the removal of soil in thin layers by raindrop impact and shallow surface flow. This action called skimming and is prevalent in the agricultural land. It results in loss of the finest soil particles that contain most of the available nutrients and organic matter in the soil. Soil loss is so gradual that the erosion usually goes unnoticed, but the cumulative impact accounts for large soil losses. This type of soil erosion is mainly responsible for loss of soil productivities. Soils most vulnerable to sheet erosion are overgrazed and cultivated soils where there is little vegetation to protect and hold the soil. Early signs of sheet erosion include bare areas, water puddling as soon as rain falls, visible grass roots, exposed tree roots, and exposed subsoil or stony soils. Soil deposits on the high side of obstructions such as fences may indicate active sheet erosion.

Fig. 1.4. Picture depicting sheet erosion.

Vegetation cover is vital to prevent sheet erosion because it protects the soil, impedes water flow and encourages water to infiltrate into the soil. The surface water flows that cause sheet erosion rarely flow for more than a few meters before concentrating into rills.

1.2.3.2.3 Rill Erosion

Rills formation is the intermittent process of transforming to gully erosion. The advance form of the rill is initial stage of gully formation. The rills are shallow drainage lines less than 30cm deep and 50 cm wide. They develop when surface water concentrates in depressions or low points through paddocks and erodes the soil. Rill erosion is common in bare agricultural land, particularly overgrazed land, and in freshly tilled soil where the soil structure has been loosened. The rills can usually be removed with farm machinery. Rill erosion is mostly occurs in alluvial soil and is quite frequent in Chambal river valley in India. The typical rill formation is presented in Fig. 1.5.
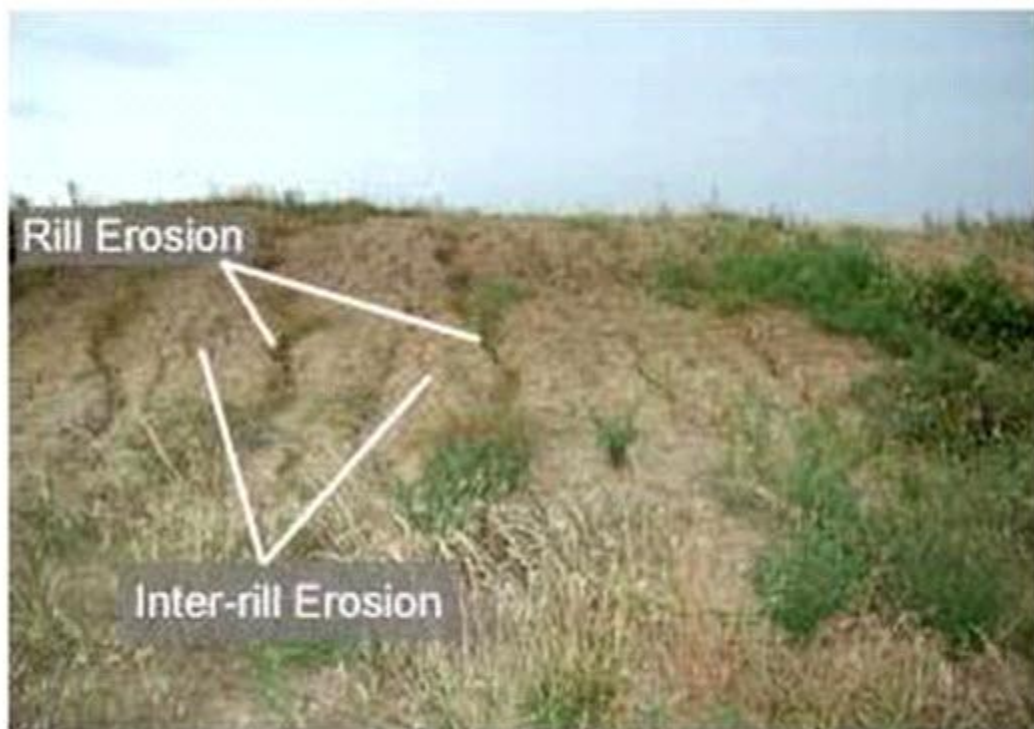
Fig.1.5. Rill formation following the sheet erosion.

1.2.3.2.4 Gully Erosion

The advance stage of rills is transformed into initial stage of gully. Gully formation are initiated when the depth and width of the rill is more than 50 cm. Gullies (Fig. 1.6) are deeper channels that cannot be removed by normal cultivation. Hillsides are more prone to gullying when they cleared of vegetation, through deforestation, over-grazing or other means. The eroded soil is easily carried by the flowing water after being dislodged from the ground, normally when rainfall falls during short, intense storms. Depending upon the depth and width, the gullies further divided into 4 classes namely G1, G2, G3 and G4 (Table 1.4). Gullies reduce the productivity of farmland where they incise into the land, and produce sediment that may clog downstream water bodies. Because of this, much effort are required to invested into the study of gullies within the scope of geomorphology, in the prevention of gully erosion, and in restoration of gullied landscapes. The total soil loss from gully formation and subsequent downstream river sedimentation can be sizable.

Fig.1.6. Picture showing gully erosion.

(Source: CSWCRTI, Dehradun: accessed on 2 October 2013)

Table 1.4. Classification of Gully

Particulars	Description of symbols of Gully			
	G1	**G2**	**G3**	**G4**
Depth in meter	Upto 1.0	1.0-3.0	3.0-9.0	>9.0
Width in meter	<18.0	<18.0	18.0	>18.0
Side slope (%)	<6.0	<6.0	6.0-12.0	>12.0

1.2.3.2.5 Tunnel Erosion

Tunnel erosion (Fig. 1.7) occurs when surface water moves into and through dispersive subsoils. Dispersive soils are poorly structured so they erode easily when wet. The tunnel starts when surface water moves into the soil along cracks or channels or through rabbit burrows and old tree root cavities. Dispersive clays

are the first to be removed by the water flow. As the space enlarges, more water can pour in and further erode the soil. As the tunnel expands, parts of the tunnel roof collapse leading to potholes and gullies. Indications of tunnel erosion include water seepage at the foot of a slope and fine sediment fans downhill of a tunnel outlet. This type of erosion is more frequent in foothills where elevation is between 500-750 meter.

Fig.1.7. Picture showing tunnel erosion.

1.2.3.2.6 Stream Bank Erosion

Stream bank erosion (Fig. 1.8) occurs where streams begin cutting deeper and wider channels as a consequence of increased peak flows or the removal of local protective vegetation. Stream bank erosion is common along rivers, streams and drains where banks have been eroded, sloughed or undercut. This is quite prevalent in alluvial river and streams. Generally, stream bank erosion becomes a problem where development has limited the meandering nature of streams, where streams have been channelized, or where stream bank structures (like bridges, culverts, etc.) are located in places where they can actually cause damage to downstream areas. Stabilizing these areas can help protect watercourses from continued sedimentation, damage to adjacent land uses, control unwanted meander, and improvement of habitat for fish and wildlife.

(a)

(b)

Fig. 1.8. Stream bank erosion in mid-Himalayan region of India.

1.2.3.2.7 Coastal erosion

The waves, geology and geomorphology are the three major factors that affect the coastal erosion. Waves are the cause of coastal erosion. Wave energy is the result the speed of the wind blowing over the surface of the sea, the length of fetch and the wind blowing time. The **geology** of the coastline also affects the rate of erosion. If the coast is made of a more resistant type of rock (say, granite),

the erosion rate will be lower than if the coast is made of a less resistant type of rock (say, boulder clay). The **geomorphology**(or shape) of the coastline further affects the rate of erosion. Headlands cause wave refraction, making waves converge and combining their energy. Wider, shallower bays, meanwhile, allow waves to diverge, losing energy due to friction with the sea bed. A wider beach cause more wave energy to be lost due to friction before the waves can break. A narrower beach will mean that the breaking point of the waves is closer to the coastline, and less energy will have been lost due to friction before they break. Similarly, shingle and pebbles will allow more water to infiltrate and cause more wave energy to be lost due to friction, while sandy beaches allow less infiltration and cause less friction and so allow for a higher rate of erosion. If the beach gradient is steep, this will encourage steeper, higher-energy waves. Paradoxically, though, because shingle and pebble beaches leave less energy for backwash, material tends to be moved upwards, making the beach steeper. The coastal erosion is a major concern for India as about 40% of the Indian coasts are subjected to severe erosion that has the potential to change the coast line. The typical coastal erosion is presented in Fig. 1.9.

Fig. 1.9. A view of eroded Goa beach.

(Source: http://www.concierge.com/ideas/beachisland/tours/)

1.3 Effects of Soil Erosion

The soil erosion adversely affects the livelihood of the people in one way or other. The major losses and problem occurs due to the soil erosion from various agents are listed below.

- Siltation of rivers.

- Siltation of irrigation channels and reservoirs.

- Problems in crop irrigation and consequent need of conserving the water.

- Damage to sea coast and formation of sand dunes.

- Disease and public health hazards.

- Soils eroded by water get deposited on river beds, thus increasing their level and causing floods. These floods sometime have various extreme effects, such as killing human and animals and damaging various buildings.

- Soil erosion decreases the moisture supply by soil to the plants for their growth. It also affects the activity of soil micro-organisms thus deteriorating the crop yield.

- Top layer of soil contains most of the organic matter and nutrients, loss of this soil reducing soil fertility and affecting its structure badly.

- Wind erosion is very selective, carrying the finest particles - particularly organic matter, clay and loam for many kilometres. There the wind erosion causes losses of fertile soils from highly productive farming areas.

- The most spectacular forms are dunes - mounds of more or less sterile sand - which move as the wind takes them, even burying oases and ancient cities.

- Sheets of sand travelling close to the ground (30 to 50 metres) can degrade crops.

- Wind erosion reduces the capacity of the soil to store nutrients and water, thus making the environment drier.

Lesson 2. Soil Conservation

2.1 Introduction

Soil conservation is an on-field activity to address the problem of soil erosion of any type, which has been discussed in previous lecture. In broader sense soil conservation activities control soil erosion through adaptation of different methods. The ultimate objective of soil conservation is to check the further soil erosion and sustain the agricultural productivity. The integrated nutrient management is also associated with soil conservation, and this employs the measures such as correction of soil defects, application of manures and fertilizers, proper crop rotations, irrigation, drainage etc., which aim at maintaining the higher level productivity in soils. However, soil conservation mainly deals with improvement of land use and reclamation of eroded land to utilize the unusable land resources under cultivation as well as protecting the land resources from further degradation. In other words, soil conservation is by itself the proper land husbandry, which would preserve the land and its fertility on a sustained basis and at the same time promote better agriculture, increase yields and achieve maximum benefits from such land. Such land husbandry should be based on proper classification of land utilization and balance allotment of lands for the different purposes, for which various local conditions are suitable.

2.2 Need for Soil Conservation

The basic needs of soil conservation remain to enhance and sustain the agricultural productivity in tandem with the prevailing agro-socio-economic practices in the respective region. However, specific objectives of soil conservation are as follows:

1. To sustain the production from natural resources to meet the basic requirements of food, shelter and clothing of growing population.

2. To preserve topsoil to reduce deterioration in soil fertility and the water holding capacity, thus sustaining productivity.

3. To check the formation of rills and gullies due to soil erosion in the field, which adversely affect the productivity.

4. To increase the groundwater recharge, by sustaining the soil moisture retention capacity of the soil.

5. To maintain the land productivity and prevent shrinkage of arable area.

6. To reduce the dredging work due to sedimentation in creeks, rivers, lakes and reservoirs.

7. To protect water bodies from non-point source pollution.

8. To minimize the flooding risk that affects the sustainability and livelihoods of humans, animals and plants.

9. To control the deterioration of ecosystem due to soil loss, which leads to interruption of nutrient cycle, loss of soil fertility, extinction of flora and fauna and soil erosion etc. ultimately resulting in biological impoverishment and human sufferings.

10. To facilitate environmental system that affects the plant growth and rejuvenation of forests.

2.3 Soil Conservation Programmes

Various watershed development programmes are being implemented by mainly three ministries, namely, Ministry of Agriculture, Ministry of Rural Development & Ministry of Environment & Forests for development of degraded lands.

These programmes are listed below,

1. National Watershed Development Project for Rainfed Area (NWDPRA)

2. River Valley Project & Flood Prone River (RVP & FPR)

3. Watershed Development Project for Shifting Cultivation Area (WDPSCA)

4. Reclamation & Development of Alkali and Acid Soil (RADAS)

5. Watershed Development Fund (WDF)

6. Drought Prone Area Programme (DPAP)

7. Desert Development Programme (DDP)

8. Integrated Wasteland Development Project (IWDP)

9. National Afforestation and Eco-Development Project (NAEP)

2.3.1 National Watershed Development Project for Rainfed Area (NWDPRA)

Rainfed areas constitute about 57% of the total 140.30 million hectares cultivated in the country. Rainfed agriculture is characterized by low levels of productivity and low input usage. Variability in rainfall results in wide variation and instability in yields. The bulk of the rural poor live in the rainfed regions, therefore, Government of India accords highest priority to the holistic and sustainable development of rainfed areas through watershed development approach. The scheme of National Watershed Development Project for Rainfed Areas (NWDPRA) was launched in 1990-91 in 25 States and 2 Union Territories based on twin concepts of integrated watershed management and sustainable farming systems.

During IX Plan, the scheme was extended to three newly formed States of Uttarakhand, Jharkhand and Chhattisgarh. The scheme of NWDPRA has been subsumed under the Scheme for Macro Management of Agriculture (MMA) from

2000-2001. At present, this scheme is being implemented as a programme of Centrally Sponsored Scheme of Macro Management of Agriculture in 28 States and 2 UTs. Funds are released to the States based on Approved Annual Work Plan. The scheme is presently being implemented on the basis of Common Guidelines for watershed development projects developed by National Rainfed Area Authority (NRAA).

2.3.2 River Valley Project and Flood Prone River (RVP & FPR)

Natural resources conservation, development and their scientific utilization are very crucial for sustained agricultural production. Land and water constitute the important natural resource base for meeting the essential requirement of the society such as food, fodder, fiber, fuel and timber. Land degradation poses a severe challenge to useful life of reservoirs and agricultural productivity. The Centrally Sponsored Scheme of Soil Conservation in the catchments of River Valley Project (RVP) was launched during the Third Five Year Plan for mounting a concerted effort at prevention of catchment deterioration.

Floods are annual features in the Indo-Gangetic Plains & in the Brahmaputra basin. Five States, namely, Uttar Pradesh, Bihar, West Bengal, Assam and Orissa account for nearly 80% of the total flood area and about 75% of the damages caused. Several Expert bodies have highlighted the need for proper catchment management for moderating peak floods and improvement of land resources and moisture regime in the catchments and reduction of silt load in the channel flow affecting river beds. In order to achieve these objectives a Centrally Sponsored Scheme of Integrated Watershed Management in the Catchments of Flood Prone River (FPR) in the Gangetic Plains was launched in Sixth Five Year Plan.

2.3.3 Watershed Development Project for Shifting Cultivation Area (WDPSCA)

Shifting cultivation known as *Jhum* cultivation in the North Eastern States is a traditional form of crop production, practiced on hill slopes. Shifting cultivation involves clearance of forest on sloppy land (usually before December), drying and burning the debris (Mid-February to Mid-March before the onset of the monsoon) and cropping. The plot remains fallow and vegetative regeneration takes place till the plot is reused for the same purpose in a cycle. Population pressure compels the shifting cultivators to reduce the earlier fallow period of 20-25 years to 2-5 years in the present days hence reducing the cycle. This form of cultivation is therefore, highly resource depleting and environmentally degrading.

 The Scheme of Watershed Development in Shifting Cultivation Areas (WDPSCA) was launched in 1994-95 by the Ministry of Agriculture and Cooperation, Government of India in the seven North Eastern States. The scheme is aimed at overall development of *jhum* areas on watershed basis, reclaiming the land affected by shifting cultivation and socioeconomic up gradation of *jhumia* families living in these areas so as to encourage them to go in for settled agriculture.

2.3.4 Reclamation & Development of Alkali and Acid Soil (RADAS)

The main objectives of this programme are to:-

- Reclamation and development of the lands affected by alkalinity and acidity

- Improvement of soil fertility by under taking appropriate on-farm measures.

- Development and application of soil amendments for growing suitable field crops and horticulture crops

- Plantation of suitable fuel wood and fodder trees as per local demand and suiting to soil capability

- Improving capacity of extension personnel and beneficiaries in various aspects of alkali and acid soils reclamation technology and

- Generate employment opportunities & thereby reduce rural urban migration.

2.3.5 Watershed Development Fund (WDF)

A Watershed Development Fund (WDF) has been established at NABARD with the objective of integrated watershed development in 100 priority districts of 18 States through participatory approach. Under WDF, two-thirds of amount is given for loan based project and one-third of amount is given for grant based project in the State. A number of externally aided projects are also under implementation on watershed approach, which covers an area of about 1.5 lakh hectares annually.

2.3.6 Drought Prone Area Programme (DPAP)

The basic objective of the programme is to minimize the adverse effects of drought on production of crops and livestock and productivity of land, water and human resources ultimately leading to drought proofing of the affected areas. The programme also aims to promote overall economic development and improving the socio-economic conditions of the resource poor and disadvantaged sections inhabiting the program areas.

2.3.7 Desert Development Programme (DDP)

The basic objective of the program is to minimize the adverse effect of drought and control desertification through rejuvenation of natural resource base of the identified desert areas. The program strives to achieve ecological balance in the long run. The program also aims at promoting overall economic development and improving the socio-economic conditions of the resource poor and disadvantaged sections inhabiting the program areas.

2.3.8 Integrated Wasteland Development Project (IWDP)

Agriculture is the mainstay of India's economy. Land and water therefore, are of critical importance. Vast tracts of the land are, however, degraded but can be brought under plough with some effort. Such lands are known as wastelands. The productivity of these lands is very low and people owning these lands are poor and are therefore forced to earn a living from wage employment. Redressing these lands is regarded as a powerful tool of attacking the issues of poverty and backwardness. Government of India has therefore, launched the Integrated Wastelands Development Program (IWDP) throughout the country so as to improve the productivity of these lands and there by improve the living standards of the rural poor who own these lands. The IWDP is a 100% centrally sponsored scheme. The development of wastelands is taken up on watershed basis. The objective of the program is to arrest rainwater runoff and conserve it in situ where it falls. This would in turn lead to control of soil erosion which is usually caused by rainwater – runoff. Soil and water conservation also leads to improved green cover in the project areas leading to improved productivity of land. Under this program, Wastelands are sought to be developed in an integrated manner based on village micro watershed plans. These plans are prepared after taking into consideration the land capability and site conditions and in consultation with the local people in regard to their needs. The watershed projects are executed by the local people using locally available low cost technologies.

2.3.9 National Afforestation and Eco-Development Project (NAEP)

The National Afforestation and Eco-Development Board (NAEB), set up in August 1992, is responsible for promoting afforestation, tree plantation, ecological restoration and eco-development activities in the country, with special attention to the degraded forest areas and lands adjoining the forest areas, national parks, sanctuaries and other protected areas as well as the ecologically fragile areas like the Western Himalayas, Aravallis, Western Ghats, etc. The detailed role and functions of the NAEB are given below.

- Evolve mechanisms for ecological restoration of degraded forest areas and adjoining lands through systematic planning and implementation, in a cost effective manner.

- Restore the forest cover in the country for ecological security and to meet the fuel wood, fodder and other needs of the rural communities.

- Sponsor research and extension activities to disseminate new and proper technologies for the regeneration and development of degraded forest areas and adjoining lands.

- Create general awareness and help foster people's movement for promoting afforestation and eco-development with the assistance of voluntary agencies, non-government organisations, Panchayati Raj institutions and others

- Coordinate and monitor the action plans for afforestation, tree plantation, ecological restoration and eco-development

- Undertake all other measures necessary for promoting afforestation, tree plantation, ecological restoration and eco-development activities in the country.

Lesson 3. Soil Conservation Approaches

3.1 What are Soil Conservation Approaches?

The top soil layer contains mostly organic matter and nutrients which are very useful for plant growth. In order to get better plant growth, the top soil layer must be protected from wind and water erosion. Measures taken for protecting the top soil layer are called soil conservation measures. These measures protect top soil either through reducing the impact of erosive agents (water and wind) or by improving the soil aggregate stability or surface roughness. The soil conservation measures can be broadly grouped into three categories namely, biological, mechanical, bio-engineering etc.

3.2 Biological Control of Soil Erosion

In this, erosion is controlled through crops or vegetation - that is by nature's way of doing things. It means that cultivation should be done in suitable way by adopting measures, which shall minimize erosion. Improper cultivation leads to severe soil erosion. A permanent vegetative cover is the best protection for soil. Studies have shown that bare ground allows four times more soil erosion compared to permanent plant covered ground. Therefore, vegetation plays an important role in controlling soil erosion and can be used as effective erosion control measure. In this method we try to plant such species, which are capable of holding soil strongly and can survive in very adverse soil condition. The main principle of biological control is to prevent high velocity of water and conserve water within the soil. In biological control following method can be adopted.

3.2.1 Mulching

Mulching is the covering of the soil with crop residues such as straw, maize, stalks etc. These cover protects the soil from the rain drop impact and reduces the velocity of runoff and wind. It is also useful as an alternative to cover crop in dry areas where a cover crop should compete for moisture with the main crop. Fig. 3.1 shows application of mulch in agriculture.

Fig. 3.1. Application of mulch in agriculture.

3.2.2 Agro Forestry

Trees can be incorporated within a farming system by planting them on terraces, contour bounds and as ornamental around the homestead. This reduces soil erosion and provides additional needs to the farmers. Fig. 3.2 shows agroforestry system in which poplars are in cultivated along with, turmeric, mango and litchi.

Fig. 3.2. Agroforestry in practice in India.

3.2.3 Reforestation/Afforestation

Reforestation: Is a process of restocking the existing depleted forests and woodlands either naturally or intentionally. Reforestation can be used to improve air quality, mitigate global warming and rebuilt ecosystem by controlling soil erosion. Forestation/plantation plays the major role in erosion control on gullies areas and landslides.

3.2.4 Crop Rotation

It may be defined as a more or less regular succession of different crops being grown on the same piece of land. Rotation of crop reduces erosion and increases the fertility of soil by different crops being grown on the same patch of land so as to enable intake of plant food from different layers of soil. In addition crop rotation increases crop yield and net profit while it reduces use of chemicals as well as water pollution. For example, leguminous crops like pulses are grown alternately with wheat, barley or mustard. Fig. 3.3 shows typical layout for crop rotation system.
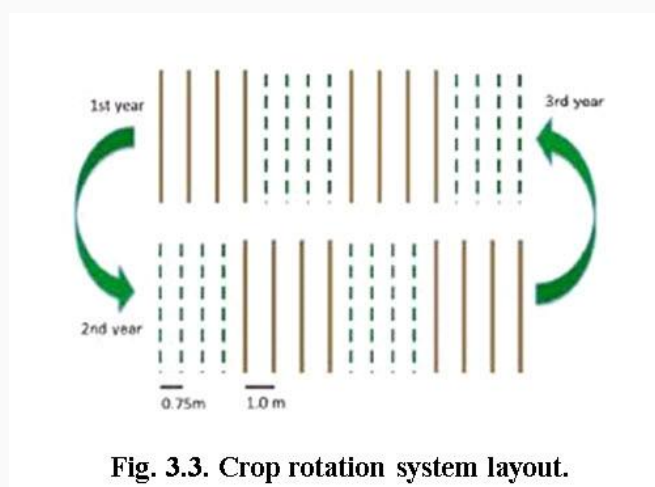
Fig. 3.3. Crop rotation system layout.

3.2.5 Mixed Cropping

In this system two or more than two crops are raised in the same land and in the same time. Mixed cropping is also known as inter cropping or co-cultivation. Mixed cropping system offers several benefits which include better utilization of soil nutrients, low weeds and insect pests, resistance to climate extremes, increase in overall productivity and use of limited resources to the fullest extent.

Fig. 3.4. Pigeonpea and sorghum intercropping.

3.2.6 Cover Cropping

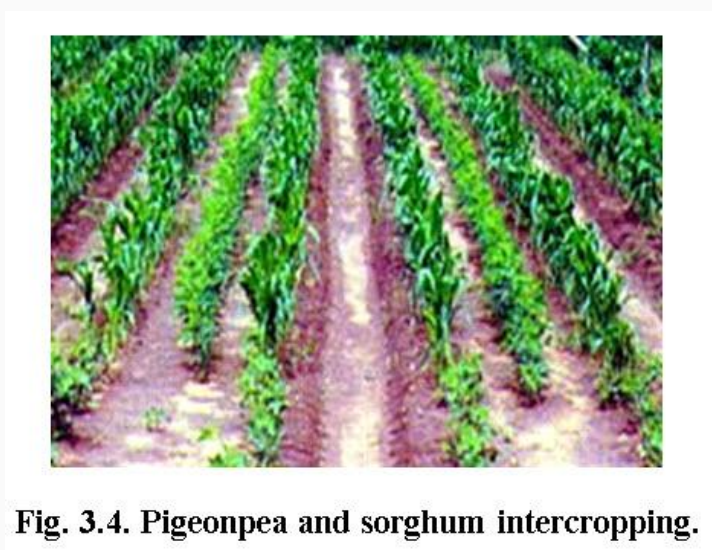Cover crops are grown as a conservation measure either during off-season or for ground protection under trees. These also add organic matter to soil. All these provide good cover from erosion control point of view and at the same time furnish hay or fodder and serve as soil building crops. These cover crops are also grown under trees to protect the soil from the impact of water drops falling from the canopy particularly important for tall trees like rubber where height of fall is more.

Fig. 3.5. Rubber plantation with cover crop in Kerala.

3.2.7 Contouring

Contouring refers to applying all tillage practices, such as, ploughing, planting, cultivation & harvesting on the contour i.e. across the slope rather than up and downhill. In regions of low rainfall, this helps in conservation of moisture and in humid areas reduces erosion by reducing surface runoff. The furrows between the ridges developed by contour tillage operations catch and hold the water, thereby, checking the high water velocity, which erodes soil and causes sheet, rill or gully erosion. On steep slopes or under conditions of high rainfall intensity and soil erodibility, contour farming alone will increase gullying because row breaks may release stored water. Under such conditions they are supplemented by strip cropping.

Fig. 3.6. Contour farming.

3.2.8 Strip Cropping

It is the practice of growing alternate strips of row crops and inter-tilled crops in the same field. In this, the crops are grown in strips at right angles to the slope of land. Erosion is largely limited to row - crop strips and soil removed from these is trapped in the next strip, which is generally planted with a leguminous or grass crop. Strip cropping reduces soil erosion due to both water and wind erosions, and water borne contamination. Strip cropping is generally of three types, namely, contour strip cropping, field strip cropping and buffer strip cropping.

(a) Buffer strip cropping   (b) Contour strip cropping

(c) Field strip cropping

3.2.9 Conservation Tillage

Conservation tillage is any method of soil cultivation that leaves previous crops residue on field before and after planting the next crop. It decreases soil erosion, runoff, water pollution, CO_2 emission and also fossil fuel. Conservation tillage methods include **no-till**, **strip-till**, **ridge-till** and **mulch-till**. **No-till** involves planting crops directly into residue that has been not at all, whereas in strip-till narrow strips are tilled and the rest of the field left untilled (strip-till). In Ridge-till row crops are planted directly on permanent ridges of 10-15 cm by clearing previous year's crop residues form ridge tops only. **Mulch-till** is any other reduced tillage system that leaves at least one third of the soil surface covered with crop residue.

Fig. 3.8. Conservation tillage.

3.3 Mechanical Methods

Mechanical practices are engineering measures used to control soil erosion from sloping land surface. The purpose of constructing the mechanical structures is to (1) to increase the time of stay of runoff water to increase the infiltration time for the water, (2) to break the land slope ,thus reducing the velocity of the runoff water. Bunds and terraces are mechanical structures used to control the soil erosion.

A terrace is an earth embankment, constructed across the slope to control runoff and thus reduces the soil erosion. Terraces are act as a slope divider. The terraces can be classified in to two groups. Bench terrace reduces land slope whereas broad base terrace removes or retains water on sloping land. The original bench terrace system consist of a series of flat shelf-like areas that convert a steep slope of 20 to 30 percent to a series of level, or nearly level benches. **Broad base terrace** is broad surface channel or embankment constructed across the slope of rolling land. On the basis of primary function the broad base terrace is further classified as Graded or Channel Type and Level or Ridge Type. The primary

function of graded terrace is to remove excess water in such a way as to minimize erosion. Erosion is controlled by reducing the slope length and conducting the intercepted runoff to a safe outlet at a non-erosive velocity. The primary function of level terrace is moisture conservation. In low to moderate rainfall regions they trap and hold rainfall for infiltration into the soil profile. They can be used even in high rainfall areas if the soil is permeable.

Fig. 3.9. Terrace system.

Based on the functional requirements, they can be divided in to two types: Contour bunds - storage of water and Graded bunds - safe disposal of excess water.

Bunds are similar to the terraces which have narrow base. Generally two types of bunds are practiced namely graded bunds and contour bunds. Graded bunds are used where rainfall is very high and contour bunds are used where rainfall is low. The choice of the type of the bunds depends on the slope, rainfall, soil type and the purpose of making the bund in the area. Contour bunds are constructed following the contour as closely as possible. A series of such bunds divide the area into strips and acts as barriers to the flow of water, thus reducing the amount and velocity of the runoff. **Graded bunds** are used for safe disposal of excess runoff in high rainfall areas and regions where the soil is relatively impervious. They may have uniform grade or variable grade.

3.4 Bio-Engineering Method

Biotechnical engineering techniques are combined with biological knowledge to build geotechnical and hydraulic structures and to secure unstable slopes and banks. Whole plants or their parts are used as construction materials to secure unstable sites, in combination with other (dead) construction material. Biotechnical methods using willows and other woody plants are especially appropriate for constructing several soil conservation structures. This structure stabilizes the soil, reduce the movement speed of running water, and thus reduce the surface erosion. Maintenance of these structures is a very important aspect as

compared with other methods of soil conservation. The maintenance cost of bioengineering structures is somewhat high in initial period and later on it becomes very less. Bio engineering methods of soil conservation have the following advantages.

- Immediately effective after installation

- Material easily available as structures also serves as a nursery for new plant material

- Flexibility in preparation and protection

Bio engineering methods also have some disadvantage

- High demand on material and labour

- Occasionally thinning of thicket necessary

- Labour intensive

Table 3.1 presents brief description and applications of some of the bio-engineering measures.

Table 3.1. Bio-Engineering techniques and their description

Technique	Application	Description
Wattle fence	Over-steepened slopes where vegetation cannot naturally establish	Long cuttings (e.g. willow) laid horizontally and supported with larger vertical plant stakes or rebar. Soil is filled in behind the fence; creates a series of terraces.
Live bank protection	Along stream banks, to protect against further erosion.	Wattle fences are contoured around bends in the stream, in areas that are susceptible to erosion; soil is then backfilled behind the fences.
Live palisades	Adjacent to a stream or river where the natural vegetation has been removed.	Large posts of a tree such as cottonwood are sunk in trenches some distance back from the edge of the stream.
Brush	Stabilizing shallow earth	Benches are excavated in the slope, willow branches are laid, on a slight angle, on the benches; branches are covered with soil, with

layering	slumps and loose soil slopes and gullies	just the tips sticking out.
Coconut fiberfascines	Stabilizing the base of a stream bank or shoreline.	Fascines (fibre "log rolls") are placed along the base of a bank; they collect sediment and help to stabilize the bank; riparian vegetation may then be planted in and alongside the roll.
Prevegetated mats	Lakeshores and wetlands	Plants are grown on mats made of a slowly biodegradable material such as coconut fibre; the mats are then simply placed on the slope.
Live gravel bar staking	Unnaturally large gravel bars amid braided streams (caused by upstream resource activities)	Using an excavator, live stakes are inserted in the gravel bar, in order to start recolonization of natural vegetation, and a return to a single stream channel.

Fig. 3.10. Fascine consisting of live branches.

Lesson 4. Physiographic Characteristics of Watershed

4.1 The Watershed

A watershed represents land area usually contains a well-connected stream network and well defined outlet or discharge point, where the representative area drains when rainfall occurs. In other words, area of the land over which runoff generates and then drains into stream at a given location is called watershed or catchment area. Watershed is also known as drainage basin, catchment area etc.
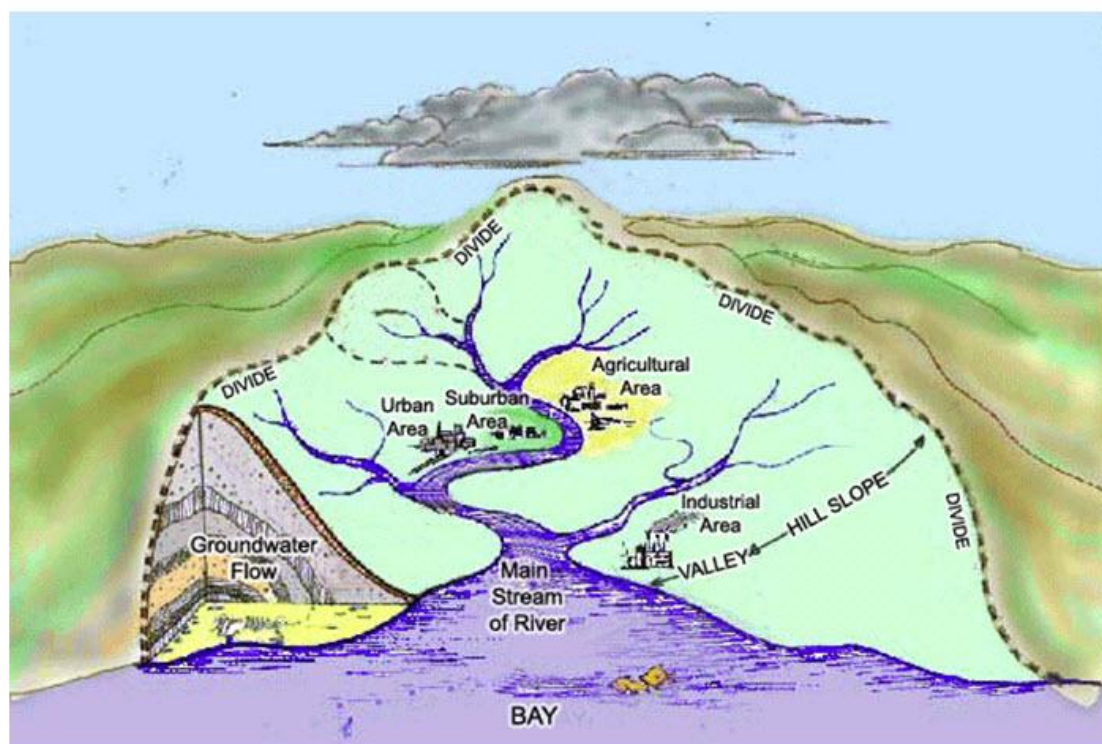
Fig. 4.1. The watershed.

4.2 Physioaphic Characteristics of Watershed

Physiographic characteristics of any watershed are the description of watershed in terms of area, slope, shape, drainage density, aspect, relief, land use and soil characteristics etc. Physical properties of watersheds significantly affect the characteristics of runoff and as such are of great interest in hydrologic analyses. These watershed characteristics are described below in details.

4.2.1 Area of the Watershed

The area of watershed is also known as the drainage area and it is the most important watershed characteristic for hydrologic analysis. The runoff from watershed is generated after the interaction of precipitation with the watershed area. Watershed area is important parameters in hydrological model to estimate the volume of runoff. The area of watershed is delineated either manually using toposheets or through digital elevation model derived using geographic information system (GIS). Once the watershed has been delineated, its area can be determined using planimeter or can be approximated using GIS.
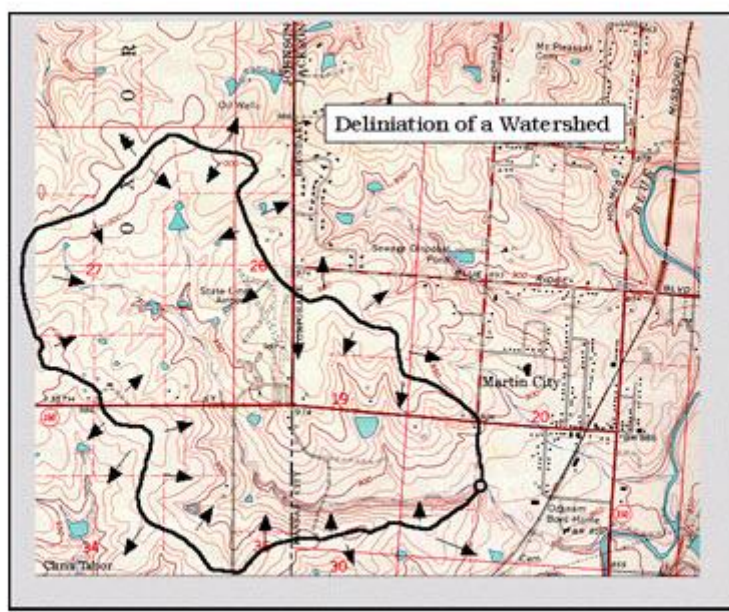

Fig. 4.2. Delineation of watershed.

The size of a watershed may vary from a few hectares to thousands of square kilometer. Table 4.1 provides a system of classifying watersheds at different levels of aggregation.

Table 4.1: System of Classification of Watersheds in India

| Category	Number	Size Ranges ('oooha.)

Regions	6	25,000-100,000
Basins	35	3,000-25,000
Catchments	112	1,000-3,000
Sub-Catchments	500	200-1,000
Watersheds	3,237	50-200
Sub-watersheds	12,000	10-50
Milli-watersheds	72,000	1-10
Micro-watersheds	400,000	0.5-1

4.2.2 Length of Watershed

Length of watershed is defined as the longest distance between outlet and any point on the perimeter. This length is usually measured to compute the time dependent parameters of watershed such as time of concentration (time taken to reach the runoff generated from remotest point of watershed to outlet).Time dependent parameters of watershed are useful in determining time for peak flow required to establish the hydrograph of the watershed. The watershed length is therefore measured along the principal flow path from the watershed perimeter to the outlet. Since the channel does not start from the watershed boundary, it is determined by extending the main channel to the boundary and then measuring the length of the channel.

Example 4.1: Main channel passes through points A, B, D, E and J. The length of AB is 1.8 km, BD is 1.3 km, DE is 1.7 km and EJ is 1.8 km. The remotest point of the watershed is K which is 0.8 km far from the start of main channel, i.e., point J. What will be the watershed length?

Solution:
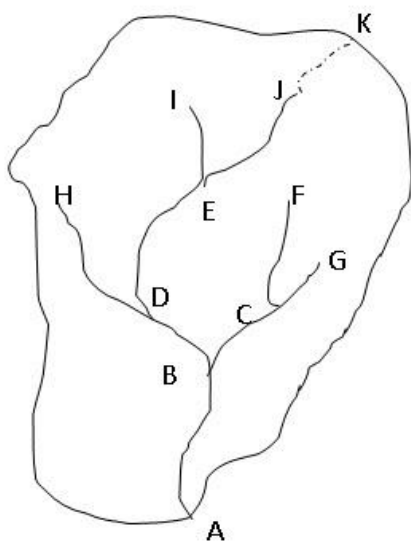
The length of main channel is

AB+BD+DE+EJ = 1.8+1.3+1.7+1.8 = 6.6 km

The distance between the start of main channel and remotest point (shown by dotted line in the picture) is JK

Length of JK is 0.8

Hence total length of watershed is 6.6+0.8 = 7.4 km

4.2.3 Slope of Watershed

Watershed slope reflects the rate of change of elevation with respect to distance along the principal flow path. It is usually calculated as the elevation difference between the highest and lowest elevation of the point of the watershed divided by watershed length. Watershed slope affects time of concentration, as well as time to peak.

Example 4.2: In continuation of example 1: K, A and J is having elevations of578m, 316 m, 532 m respectively. Calculate the watershed slope, channel slope and overland slope?

Watershed slope = elevation difference between point K and A divided by watershed length i.e. (578-316)/7.4 = 262/7.4 = 35.4 m per km = 3.54%

Channel slope = elevation difference between point J and A divided by channel length i.e. (532-316)/6.6 = 216/6.6 = 32.73 m per km = 3.27%

Overland slope = elevation difference between point K and J divided by length of overland flow i.e. (578-532)/0.8 = 57.5 m per km = 5.75%

In this case overland slope is higher than channel slope, hence on field soil conservation activities such as trenching and bunding etc. should be prioritize over drop structures etc.

4.2.4 Shape of Watershed

Watershed shape basically determines the shape of the resulting hydrograph. Watersheds have an infinite variety of shapes, and the shape supposedly reflects the way that runoff will accumulated at the outlet. A circular watershed would result in runoff from various parts of the watershed reaching the outlet at the same time. An elliptical watershed having the outlet at one end of the major axis

and having the same area as the circular watershed would cause the runoff to be spread out over time, thus producing a smaller flood peak than that of the circular watershed. Fig. 4.3 showing the basin effects on hydrograph shape.
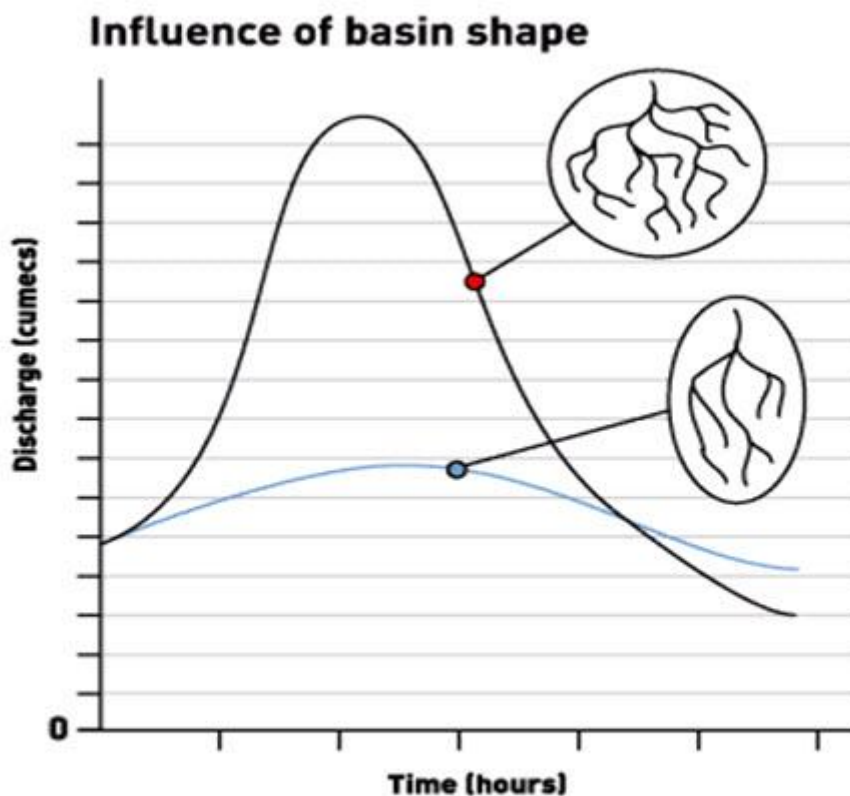
Fig. 4.3. Effect of basin on hydrograph shape.

4.2.5 Stream Order

The order of stream is hierarchical arrangement of different streams in the watershed. The first order streams are the originating streams and mostly in the forms of G1 or G2 type gullies. On confluence of two first order streams, downstream is called second order stream. If second order stream is confluences with first order stream, the stream still is second order. However, if two second order streams confluences, then 3 order stream comes in existence. The general rule is that when two same order stream confluences, next order stream comes into existence. Most of the watersheds are having 3rd order streams. However, the big catchment, the stream order may be of 5th and higher.
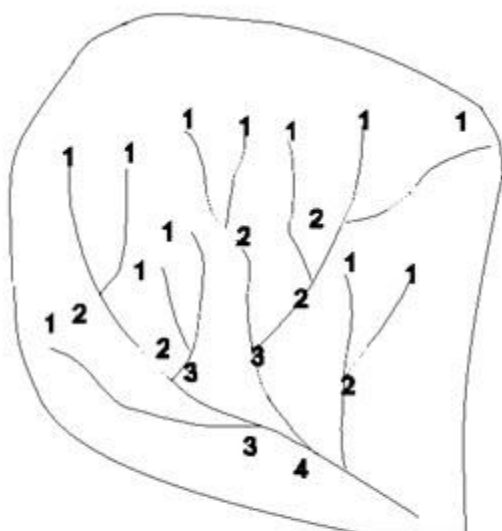
Fig. 4.4. Stream Order of watershed.

A number of watershed parameters have been developed to reflect basin shape. Following are a few typical parameters:

Table 4.1. Formula to determine different shape factor of watersheds and its effect on hydrograph

SL. No	Characteristics	Formula	Effect on hydrograph shape (refer figure below)
1	Stream order	Hierarchical	Higher stream order increases time of concentration, time to peak and base time
2	Stream length	Length of the stream	Higher the length, higher the time of concentration, time to peak and base time
3	Bifurcation ratio	R_b = Bifurcation ratio, n_u = number of stream segment of order u n_{u+1} = number of	Higher the length, higher the time of concentration, time to peak and base time

		stream segment of order u+1	
4	Relief ratio	Rh = relief ratio; H = total relief of watershed in kilometer and Lb = watershed length, km	Higher relief reduces time to peak and increases peak discharge
5	Drainage density	D = drainage density; Lu = total stream length of all orders, km and A = watershed area, km^2	Higher drainage density increase time to peak and peak discharge
6	Form factor	Rf = Form factor, A = watershed area, km^2 L_b= watershed length, km	Higher form factor increases peak discharge and reduces time of concentration and base time
7	Circularity ratio	Rc = circularity ration, A = watershed area,km^2 and P watershed perimeter, km	As circulatory ration approaches to unity, time of concentration and peak discharge increases.
8	Elongation ratio	Re = Elongation ratio, A = area of watershed, km^2	Higher elongation ration increases concentration time and time to peak.

		L_b = watershed length, km	
9	Length of overland flow	(Horton, 1945) L_g = length of overland flow, km D = Drainage density, km^{-1}	More length of overland flow increases time of concentration, time to peak and reduces peak discharge.

(Source: Chow, 1964)
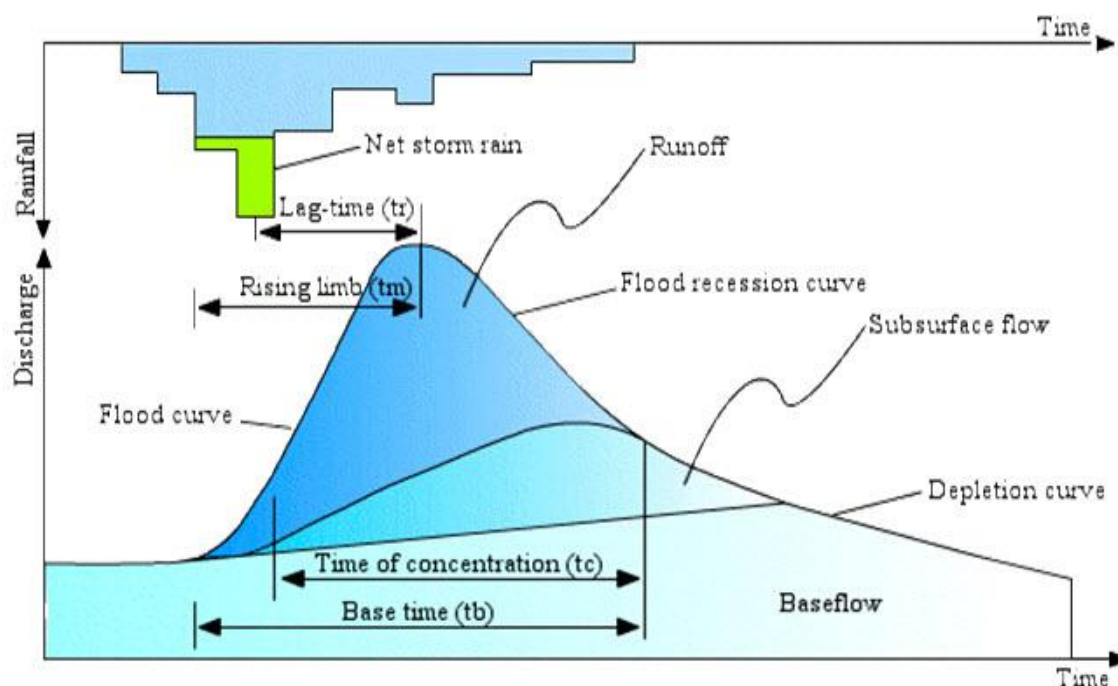
Fig. 4.5. Components of hydrograph.

4.2.6 Watershed Maps

There are several watershed maps describing different characteristics of watershed. These are also known as thematic maps. The map include elevation map (contour map), land use/land cover map, slope map, soil map, flow direction map, watershed delineation map etc. These thematic maps are overlaid in order to determine the specific characteristics of watershed. The GIS is often used in the overlaying process and map calculations.

4.2.6.1 Delineating Watershed Boundary from Toposheet

Topographical maps provide information about the lay of the land. The special feature of topographical maps (or toposheets) is that along with direction, scale and legend, they also provide information about the relief of the land using contour lines(contour lines are imaginary lines joining points on the same elevation). It also contains the information of drainage network, water harvesting structures, land use, soil type villages and urban settlements, roads. A typical toposheet is shown in Fig. 4.6.Toposheets are made on the basis of latitudes and longitudes. Every part of India has been mapped by the Survey of India, using latitudes and longitudes to classify the country into a grid. Toposheets are available mainly on 3 scales: 1:1,000,000, 1:2, 50,000 and 1:50,000. For some special areas toposheets on 1:25,000 are also available.
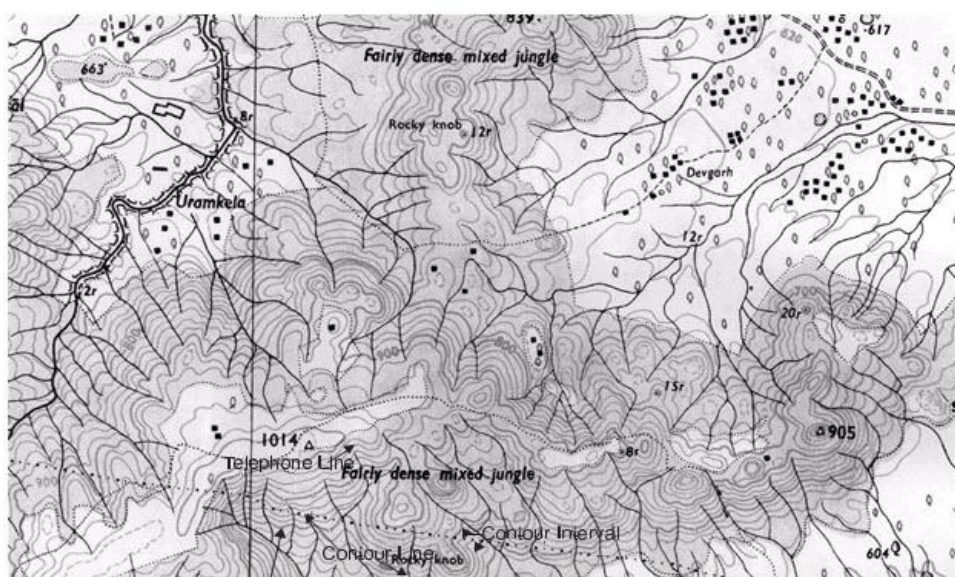
Fig. 4.6. A typical Toposheet.

4.2.6.1.1 Steps for Watershed Delineation

Step 1. Identify the point with respect to which the watershed is to be marked(the exit point or outlet). In Fig. 4.7, this is the point marked "A".
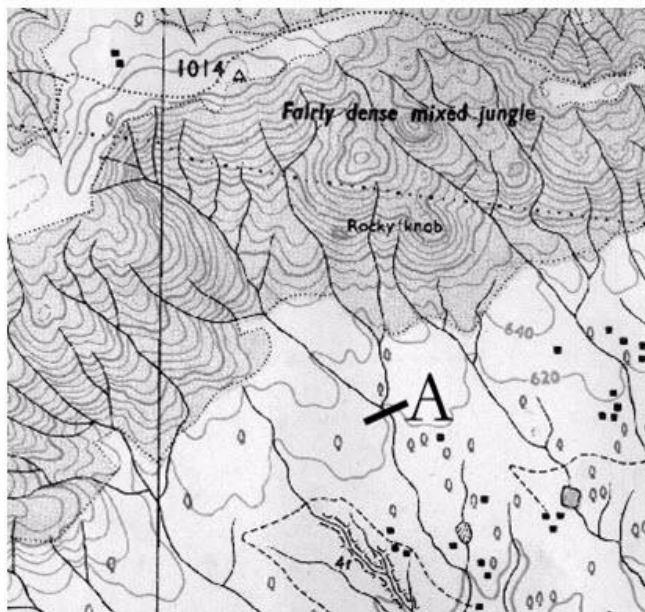
Fig. 4.7. Watershed delineation: First choose an Outlet point.

(Source: BACPE, 2007)

Step 2. Mark out drainage lines of various orders, which drain into this common point (Fig. 4.8).The way to do this is to begin from the exit point (A) and move along the drainage line to its origin. Mark out nearby drainage lines which do not drain to this common point. Different colours can be used to distinguish drainage lines belonging to our watershed from drainage lines outside our area.

Step 3. From the exit point, draw a line around the drainage system, enclosing all drainage lines which drain to point A and leaving out other drainage lines which do not drain to point A (see Fig. 4.9). This boundary line will terminate at the exit point. This line demarcating the watershed boundary is called a ridge line. A ridge line is an imaginary line joining all points of higher elevation in a selected watershed and separating the watershed from other watersheds
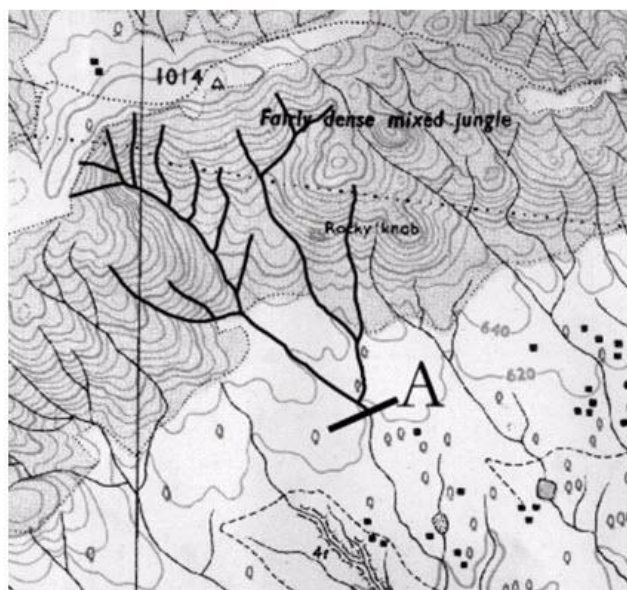
Fig. 4.8. Then mark the drainage lines. (Source: BACPE, 2007)
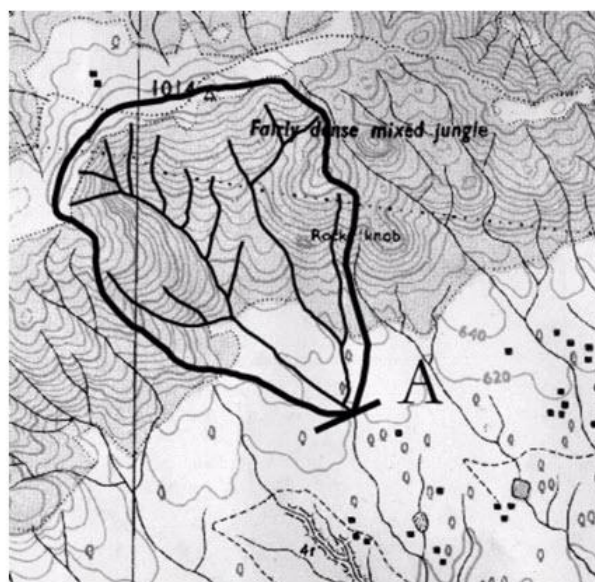
Fig. 4.9. Mark out the watershed boundary. (Source: BACPE, 2007)

Precautions

- Remember to be very careful when outlining drainage lines, particularly when including drainage lines which fall in the watershed and leaving out those that do not.
- Remember that a toposheet shows several other lines (such as roads, telephone lines), which on a photocopy will look similar to a contour line. So exercise care.
- Ensure that the watershed boundary (the ridge line) never crosses any drainage line inside the watershed. If this has happened, be sure that it is a mistake and correct it.

4.2.6.2 Automatic Delineation of Watershed

The watersheds can also be delineated automatically using modern computing tools such as GIS. GIS needs digital elevation map (DEM) for automatic delineation of watershed. In this case the contour features from toposheets are scanned and accordingly digitized using either screen digitizer or table digitizer. Respective elevations are assigned to the digitized contour. After elevations are assigned, the outlet is defined using latitude and longitude. The GIS calculates the flow direction map, slope map, flow accumulation map and computes drainage network and using these maps along with drainage network, GIS finally delineate watershed. However, watershed delineated using GIS should be cross checked with the toposheet. Nowadays DEM are available at various websites as a free resource (eg: http://www.bhuvan.nrsc.gov.in/bhuvan_links.php, etc.), such as SRTM (Shuttle Radar Topographic Mapping) or ASTER (Advanced Space born Thermal Emission and Reflection) and can be used to delineate the watershed. The DEM from SRTM or ASTER source usually has 1 meter elevation interval and 30 and 90 m resolution in grid form. The 30 m resolution suggest singular elevation value for 0.09 ha (30mX30m) and similarly 90 meter resolution suggest singular elevation value for 0.81 ha (90mX90m). Though the automatic delineation is free from human interventions and errors and produces same result every time. However, these are limitations as well. Accuracy is largely dependent on resolution (Higher the resolution, greater the accuracy). Higher resolution data requires higher computing and data transfer infrastructure along with highly skilled personnel. This may significantly increase the cost of delineation.

Lesson 5. Peak Rate of Runoff

5.1 Introduction

One of the key parameters in the design and analysis of soil and water conservation structures is the resulting peak runoff or the variations of runoff with time (hydrograph) at the watershed outlet. The runoff generates from rainfall excess drains though the channels of different order and finally reaches the watershed outlet. The flow at outlet starts with minimum flow called base flow (sometimes its value is zero) and attains the maximum flow after some times and then recedes to base flow again. This time variation of flow is called hydrograph. The maximum flow at outlet thus attained is called peak flow of runoff. This peak flow also includes base flow which should be separated to obtain net peak flow due to rainfall excess. In many instances, however measurement of runoff is not possible and therefore, it is estimated through different hydrologic models/methods. Rational method is very commonly used.

5.2 Rational Methods

The estimation of peak flow of runoff is particularly important to design various soil and water conservation structures. Rational method is a simple method to estimate peak flow of runoff. In SI units, the equation of rational method that relates the area of watershed (A) in ha, and rainfall intensity (i) in mm/hr(for a duration equal to time of concentration, T_c) with some dimensionless coefficients to peak flow rate (q) in m³/s is given below

$$q = \frac{C \times I \times A}{360} \qquad (5.1)$$

The runoff coefficient represents the integrated effects of infiltration, evaporation, retention, flow routing and interception, all of which affect peak rate of runoff.

The runoff coefficient,(C) varies with rainfall rate, land use and cultivation practices, and hydrologic soil groups. The hydrologic soil group B values under different land use are given in Table 5.1. For other hydrologic soil group i.e. A, C and D, there are correction factor which should be multiply to the C values of group B to get the final runoff coefficient. The correction factors are given in Table 5.2.

Table 5.1: Typical values of runoff coefficient, C for different land use and rainfall intensity for hydrologic soil group B

Crop and Hydrologic conditions	Coefficient C for rainfall rates of		
	25 mm/h	100 mm/h	200 mm/h
Row crop, poor practice	0.63	0.65	0.66
Row crop, good practice	0.47	0.56	0.62
Small grain, poor practice	0.38	0.38	0.38
Small grain, good practice	0.18	0.21	0.22
Grassland with rotating species good practice	0.29	0.36	0.39
Permanent pasture under good practices	0.02	0.17	0.23
Forest with matured woodland	0.02	0.1	0.15

(Source: Schwab et al., 1993)

Table 5.2: Conversion factor to convert runoff coefficient of soil group B to other soil group

Crop and Hydrologic conditions	Conversion factor to convert value of C from soil group B to		
	Soil group A	Soil group C	Soil group D
Row crop, poor practice	0.89	1.09	1.12
Row crop, good practice	0.86	1.09	1.14
Small grain, poor practice	0.86	1.11	1.16
Small grain, good practice	0.84	1.11	1.16

Grassland with rotating species good practice	0.81	1.13	1.18
Permanent pasture under good practices	0.64	1.21	1.31
Forest with matured woodland	0.45	1.27	1.40

(Source: Schwab et al. 1993)

Table 5.3: Runoff coefficient as affected by slope

Hydrologic Soil Group	Slope (%)	Land use/cover			
		Forest	Meadows	Pasture	Farmland
A	<2%	0.08	0.14	0.15	0.14
	2-6%	0.11	0.22	0.25	0.18
	>6%	0.14	0.30	0.37	0.22
B	<2%	0.10	0.20	0.23	0.16
	2-6%	0.14	0.28	0.34	0.21
	>6%	0.18	0.37	0.45	0.28
C	<2%	0.12	0.26	0.30	0.20
	2-6%	0.16	0.35	0.42	0.25
	>6%	0.20	0.44	0.52	0.34

	<2%	0.15	0.30	0.37	0.24
D	2-6%	0.20	0.40	0.50	0.29
	>6%	0.25	0.50	0.62	0.41

Rational Formula for Multiple Land Use

The runoff coefficient for composite land use are modified as weighted average over area and is given as

$$C = \frac{\sum C_i A_i}{A_i} \qquad (5.2)$$

Where, C_i is the runoff coefficient for i type land use and A_i is the area for the i type land.

5.3 Time of Concentration

The Rational method uses the rainfall intensity for the duration equal to the time of concentration. The time of concentration is the time lag to reach the generated runoff at remotest point of the watershed to the outlet. The time of concentration is related to the geomorphological characteristics of the watershed such as watershed slope and length of main channel. The following empirical relationship between time of concentration and channel length and slope is given by Kirpich (1940):

$$T_c = 0.0195 \times L^{0.77} \left(\frac{L}{H}\right)^{0.385} \qquad (5.3)$$

Where, Tc is time of concentration (min), L is length of main channel (m) and H is elevation different between outlet and remotest point in the watershed (m). The T_c thus obtained is used to establish rainfall intensity using intensity-duration-frequency curve.

5.4 Rainfall Intensity Duration srnd Frequency (IDF relationship)

The intensity-duration-frequency (IDF) curve is a set of characteristics curve that describe the rainfall characteristics specific to the region. In many design

problems related to watershed management, it is necessary to know the rainfall intensities of different durations and different return periods. It takes into the account of probabilistic of the rainfall to exceed certain intensity and its frequency in the selected return period. The inter-relationship among the intensity, i, (mm/h), duration, D (h) and return period, T (years) is described below:

$$i = \frac{KT^x}{(D+a)^n} \qquad\qquad (5.4)$$

K, T, a and n are the constants. These constant are specific to the given catchment.
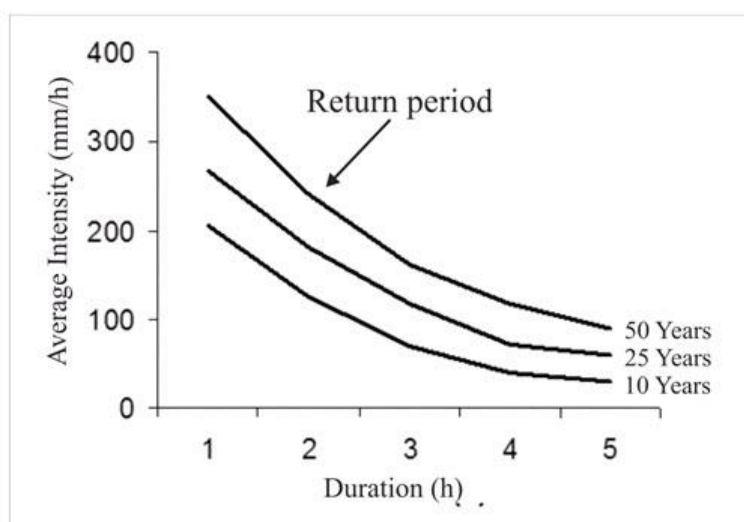
Fig. 5.1. Intensity-duration-frequency curves.

)

Fig. 5.1 shows a typical variation of intensity with duration and return period. Typical values of the constants, K, T, a and n for few Indian location are given in Table 5.4.

Table 5.4. Values of constant for IDF relationship

Station	K	x	a	n
Bhopal	6.93	0.189	0.50	0.878
Nagpur	11.45	0.156	1.25	1.032

Chandigarh	5.82	0.160	0.40	0.75
Bellary	6.16	0.694	0.50	0.972
Raipur	4.68	0.139	0.15	0.928

(Based on the studies of CSWCRTI, Dehradun)

Example 5.1: Consider a watershed of certain area. Let's assume there is a rain gauge station and the watershed is meteorologically homogeneous. The rainfall data for last 30 years are analyzed and presented in Table 5.5.

Table 5.5: Rainfall data corresponding to different RI and duration at a sample station

Return Period (years)	Excess rainfall (mm/hr)					
	5-min	10-min	15-min	30-min	60-min	90-min
2	12.8	16.3	20.9	28.4	36.1	42.3
5	17.6	24.2	29.8	38.4	47.3	58.2
10	24.3	30.1	38.5	44.9	58.0	67.8
20	39.1	35.1	43.8	51.2	59.4	71.4
25	45.2	50.3	54.2	57.3	61.3	75.6
50	55.9	59.3	66.3	71.2	76.3	81.7
100	71.6	78.5	87.3	91.4	96.7	102.5

The IDF curves are established using double-log paper with abscissa as intensity and ordinates as duration (minutes). The points are plotted on the log-log paper for return period such that one IDF respective to return period.

Plot the IDF curve for rainfall data given above

Solution:

Take a log-log paper. Plot the points for 12.8, 16.3, 20.9, 28.4, 36.1 and 42.3 mm/hr intensity respective to 5, 10, 15, 30, 60 and 90 minute duration corresponding to 2 year return period on this log paper as shown in Fig. 2. Follow the same procedure for the other return periods.
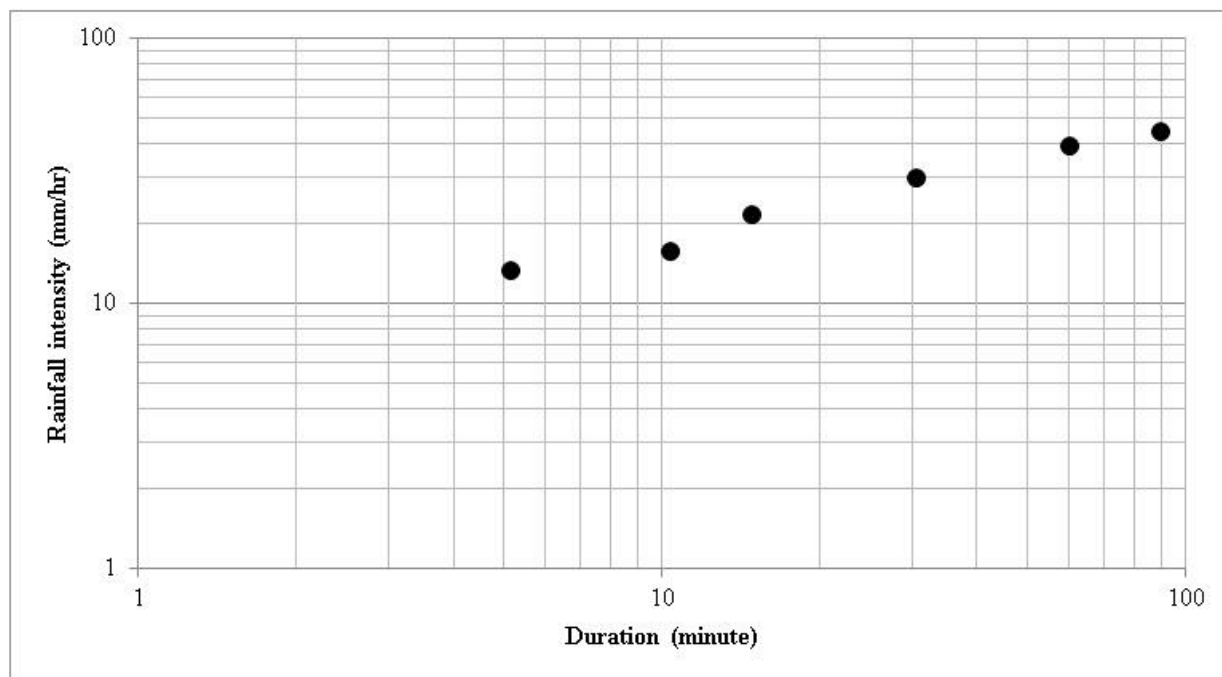
Fig. 5.2. IDF curve for 2 year return period.

5.5 Limitations of Rational Method

When used correctly, the Rational method can be a very effective tool at estimating runoff. However, several limitations should be considered before using this method.

- The Rational method assumes that the drainage basin characteristics are fairly homogeneous. If the watershed being considered includes a variety of surfaces, such as paved areas, wooded areas, and agricultural fields, then another method should be selected.

- The Rational method is less accurate for larger areas and is not recommended for drainage areas larger than 80 ha.

- The only output from Rational method is a peak discharge (the method provides only an estimate of a single point on the runoff hydrograph).

- The average rainfall intensities used in the formula have no time sequence relation to the actual rainfall pattern during the storm.

5.5.1 Assumptions

- For this method, it is assumed that a rainfall duration equal to the time of concentration results in the greatest peak discharge.

- The time of concentration is the time required for runoff to travel from the most distant point of the watershed to the outlet. Intuitively, once a rainfall event begins the amount of water flowing out of the watershed will begin to increase until the entire watershed is contributing water, at the time of concentration.

- Rainfall intensity is uniform over duration of time equal to or greater than the time of concentration. Under steady rainfall intensity, the maximum discharge will occur at the watershed outlet at the time when the entire area above the outlet is contributing runoff

- Rational coefficients are independent of the intensity of the rainfall.

Example 5.2: Determine the design peak runoff rate for a 50-year return period storm from a 50-ha watershed, with the following characteristics:

Area(ha)	Topography, per cent slope	Soil group	Land use, treatment, and hydrologic condition
30	5-10	C	Row crop, contoured, good
20	10-30	B	Woodland, good

The maximum length of flow is 600 m and the difference in elevation along the path is 3 m.

Solution:

The watershed gradient can be calculated as $(3/600) \times 100 = 0.5\ \%$

The time of concentration (T_c) is calculated by using the Eq. 5.3 as follows:

$$T_c = 0.0195 \times (600)^{0.77} \left(\frac{600}{3}\right)^{0.385}$$

$$T_c = 20 \text{ min.}$$

For a 50-year return period of the study area, the rainfall intensity for duration equal to the time of concentration of the watershed can be calculated from the Eq. 5.4 is 97 mm/h

The runoff coefficients C from Table 5.1 for row crop, good practice, and woodland are 0.56 and 0.10, respectively, and the factor correcting hydrologic soil group B to group C for the 30-ha subarea from Table 5.2 is 1.09

The runoff coefficient is calculated as follows

$$C = \frac{30}{50} \times 0.56 \times 1.09 + \frac{20}{50} \times 0.10 = 0.41$$

.... (from Eq. 5.2)

Thus, the design peak runoff rate is calculated by using Eq. 5.1 as

$$q = \frac{0.41 \times 97 \times 50}{360} = 5.524 \, m^3/s$$

Example 5.3: The land use pattern of a watershed is given in the following table. The hydrologic soil group of the watershed is B. Calculate the peak rate of runoff for rainfall intensity of 100 mm/h for the duration of Tc of watershed.

Land Use classification	Crop and management practice	Area(ha)
Land use A	Forest with matured woodland	25
Land use B	Row crop, good practice	30
Land use C	Small grain, good practice	20
Land use D	Grassland with rotating species, good practice	10

Solution:

From the Table 5.1, the composite runoff coefficient for the given watershed is calculated as

$$C = \frac{25 \times 0.1 + 30 \times 0.56 + 20 \times 0.21 + 10 \times 0.36}{85} = 0.32 \text{ (from Eq. 5.2)}$$

So, the peak runoff rate is calculated by using the Eq. 5.1 as

$$q = \frac{0.32 \times 100 \times 85}{360} = 7.56 \; m^3/s$$

Example 3: A catchment has an area of 500 ha. The average slope of the land surface is 0.5% and the maximum travel depth of rainfall in the catchment is approximately 2 km. The maximum depth of rainfall in the area with a return period of 25 years is as tabulated below:

Time duration (min)	5	10	15	20	25	30	40	60
Rainfall depth (mm)	15	25	32	45	50	53	60	65

Consider that 200 ha of the catchment area has cultivated sandy loam soil (C = 0.2) and 300 ha has light clay cultivated soil (C = 0.7). Determine the peak flow rate of runoff by using the Rational method.

Solution:

The time of concentration (T_c) is calculated by using the Eq. 5.3 as follows:

$$T_C = 0.0195 \times (2000)^{0.77} \times (0.005)^{-0.385}$$

T_c = 52.21 min.

The maximum rainfall depth for 52.21 min duration would fall between the period 40-60 min and is located at 12.21 min after the 40 min period at which the maximum rainfall depth is 60 mm, as from the Table provided/ available data.

The rainfall depth during the 12.21 min period $\frac{65-60}{20} \times 12.21 = 3 \; mm$

Therefore, at 52.21 min duration, the rainfall depth = 60 + 3 = 63 mm.

The average rainfall intensity = maximum rainfall depth/T_c (during the period of T_c)

$$= \frac{63}{52.21} \times 60 = 72.4 mm/h$$

The runoff coefficient is calculated as follows

$$C = \frac{200 \times 0.2 + 300 \times 0.7}{500} = 0.50 \; \text{(from Eq. 5.2)}$$

The Rational method of predicting peak runoff rate is calculated by using Eq. 5.1 as

$$q = \frac{0.50 \times 72.4 \times 500}{360} = 50.28 \text{ m}^3\text{/s}$$

Lesson 6. Runoff Estimation

In the previous lecture, peak runoff estimation with the Rational method was discussed. However, in many situations information on runoff volume is needed for designing storage structures in conjunction with disposal of excess runoff such as farm ponds, drop structures etc. There are several techniques to compute runoff volume or runoff depth, among them curve number (SCS-CN) technique is most widely used.

6.1 SCS-CN Method

The SCS curve-number (SCS-CN) method was developed by the Soil Conservation Service for estimating runoff volume (SCS, 1969). It is widely used to estimate runoff from small-to medium-sized watersheds. It relies on only one parameter, i.e., curve number CN.

6.1.1 Basic Concepts

Runoff volume V_q is the total volume of runoff water occurring over a period of time expressed as

$$V_q = \int_0^t Q(t)\, dt \qquad (6.1)$$

Where Q_t is the discharge at time t.

This runoff volume resulted due to the precipitation occurred on a drainage basin. The Curve Number Method is based on two phenomena. The fundamental hypotheses of this method are:

1. Runoff starts after an initial abstraction I_a(mainly consists of interception, surface/depression storage, and infiltration) has been satisfied,
2. The ratio of actual retention of rainfall to the potential maximum retention S is equal to the ratio of direct runoff to rainfall minus initial abstraction.

To describe the phenomena, mathematically the relationship can be expressed as:

$$\frac{F}{S} = \frac{V_q}{P-I_a} \qquad (6.2)$$

Where F is the actual retention, S is the potential maximum retention, P is the accumulated rainfall depth, I_a is the initial abstraction.

Fig. 1(a) and (b) shows the above relationship for certain values of the initial abstraction and potential maximum retention. After runoff has started, the actual retention equals to rainfall minus initial abstraction and runoff.

Thus,

$$F = P - I_a - V_q \qquad (6.3)$$

Putting Eq. (6.3) in (6.2) gives

$$\frac{P-I_a-V_q}{S} = \frac{V_q}{P-I_a} \qquad (6.4)$$

Thus,

$$V_q = \frac{(P-I_a)^2}{P-I_a+S} \qquad (6.5)$$

To eliminate the need to estimate the two variables I_a and S in Eq. (6.5), a regression analysis was made on the basis of recorded rainfall and runoff data from small drainage basins (SCS 1972). The following average relationship was found

$$I_a = 0.2S \qquad (6.6)$$

Physically it means that for a given storm, 20% is the initial abstraction before the start of runoff. For Indian conditions,

$$I_a = 0.3S \qquad (6.7)$$

The value of I_a is subjected to corrections based on different AMC conditions and soil type and can vary from 0.1S to 0.4S. For red soil (Alfisol) and black soil (Vertisol), I_a value is taken as 0.15S and 0.3S respectively (Dhruvanarayana, 1993).

Combining Eqns. (5) and (6) gives

$$V_q = \frac{(P-0.2\,S)^2}{P+0.8\,S} \text{ for } P>0.2\ S \qquad (6.8)$$

The Eq. (6.8) is the rainfall-runoff relationship used in the CN Method. It allows the runoff depth to be estimated from rainfall depth, given the value of the potential maximum retention S.

6.1.2 Estimation of S

Estimation of the potential maximum retention, S in a watershed is very difficult as it depends on the characteristics of soil-vegetation-land use (SVL) complex and antecedent soil-moisture conditions. The Soil Conservation Service (SCS) expressed S as a function of curve number as:

$$CN = \frac{1000}{S+10} \qquad (6.9)$$

or

$$S = \frac{1000}{CN} - 10 \qquad (6.10)$$

Where CN is a dimensionless number ranging from 0-100 as shown in Fig. (2), S is in inches. For SI unitof S (mm) the Eq. (6.9) is modified to

$$CN = \frac{25400}{S+254} \qquad (6.11)$$

The usual practice to compute runoff, V_q, first compute S for given CN values (using Eq.6.10) and then substitute S in the (Eq. 6.8).

For example: For paved areas, when CN equals 100, S becomes zero (Eq.6.10) and all rainfall will become runoff (Eq.6.8). In contrasts, for highly permeable, flat lying soils, when CN equals zero, S will go to infinity, (Eq. 6.10), hence, all rainfall will infiltrate and there will be no runoff. In drainage basins, the reality will be in between these two conditions.
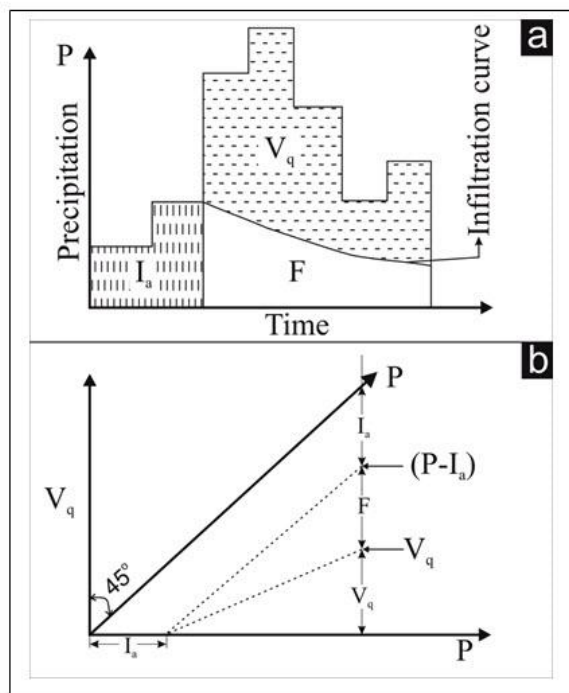
Fig. 6.1. (a). SCS relation between precipitation, runoff, and retention. (b)

A mass curve representation of the SCS relation between precipitation,

runoff, and retention. (Source: Singh, 1992)

To estimate the volume of direct runoff Eq. (6.8) and (6.10) can be used for the known amount of precipitation and curve number. The SCS (1969) developed a graphical solution as shown in Fig. 6.2 of these equations. Either of these approaches can be made to estimate the volume of surface runoff.
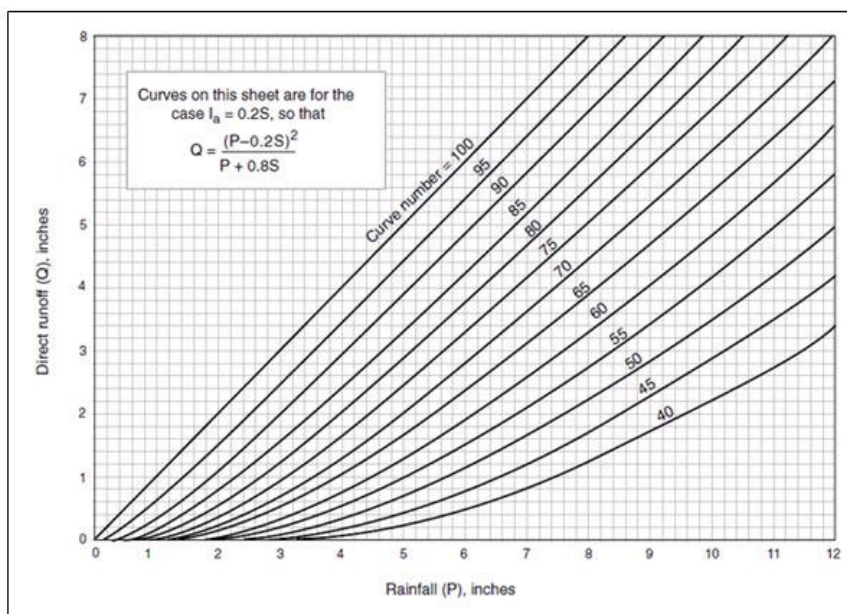

Fig. 6.2. Volume of direct runoff as a function of rainfall and curve

number. (Source: Soil Conservation Service, 1969)

6.1.3 Limitations of SCS CN Method

The followings are the limitations of SCS curve number method:

(i) The soil group of the basin should have uniform hydrologic characteristics.

(ii) Rainfall should be uniform and distributed uniformly over the basin area.

(iii) All other hydrologic characteristics should be uniform.

As most of the drainage basins do not satisfy the above assumptions, this curve number method over-predicts by a large magnitude.

6.1.4 Peak Flow Rate Determination using SCS-CN

The SCS-CN estimates the peak runoff rate by using the following equation developed by Ogrosky and Mockus (1957) by using the 6-hour rainfall as the design frequency of small watersheds.

$$Q_p = \frac{0.0208 \times Q \times A}{t_p} \tag{6.12}$$

Where Q_p is peak rate of runoff in m³/s, Q is the runoff depth in cm, A is area of watershed in ha, t_p is the time to peak in hour. Time to peak, t_p, is estimated from time of concentration, t_c, in hour, using the following equation:

$$t_p = 0.6 \times t_c + \sqrt{t_c} \tag{6.13}$$

The time of concentration, t_c can be determined by the CN Method using the following equation (Schwab et al., 1993):

$$t_c = \frac{L^{0.8}\left[\left(\frac{1000}{CN}\right)-9\right]^{0.7}}{4407 \, (S_g)^{0.5}} \tag{6.14}$$

Where L is the longest flow length in metre, CN is the curve number, S_g is the average slope of the watershed in percent.

6.2 Hydrologic Soil Group

The CN values are highly dependent on the soil surface. The soil surfaces are grouped into 4 classes which are known as hydrologic soil groups. These are classified into 4 classes on the basis of runoff potential of the surface and are described below:

1.   **Group-A:** (Lowest Runoff Potential): Soils in this group have the lowest runoff potential(high infiltration rates) even when thoroughly wetted and consist chiefly of deep, well to excessively drained sands or gravels. These soils have a high rate of water transmission.

2.   **Group-B:** (Moderately Low Runoff Potential): Soils having moderate infiltration rates when thoroughly wetted and consisting chiefly of moderately deep to deep, well drained to moderately well-drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission.

3.   **Group-C:** (Moderately high Runoff Potential): Soils having slow infiltration rates when thoroughly wetted and consisting chiefly of soils with a layer that impedes downward movement of water, or soils with moderately fine to fine texture. These soils have a slow rate of water transmission.

4.   **Group-D:** (Highest Runoff Potential): Soils having very slow infiltration rates when thoroughly wetted and consisting chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a clay pan or clay layer at or near the surface and shallow soils over nearly impervious material. These soils have a very slow rate of water transmission.

The characteristics and ranges of infiltration rates of the soil groups are described in Table 6.1.

Table 6.1. Soil group classification (Source: Singh, 1992)

Group	Soil characteristics	Minimum infiltration rate(in./h)
A	Deep sand, deep loss, and aggregated silts	0.3-0.45
B	Shallow losses and sandy loam	0.15-0.30
C	Clay loams, shallow sandy loam, soils in organic content, and soils usually high in clay	0.05-0.15
D	Soils that swell upon wetting, heavy plastic clays, and certain saline soils	0-0.05

6.3 Antecedent Moisture Condition (AMC)/Antecedent Runoff Conditions

Antecedent Moisture Condition is the preceding relative moisture of the pervious surfaces prior to the rainfall event. AMC is an important factor in runoff process because it reflects the relative saturation of the soil, which influences the infiltration process. AMC is also known as Antecedent Runoff Condition (ARC). Antecedent moisture considered as low, when there has been little preceding rainfall and high, when there has been considerable preceding rainfall prior to the rainfall event under consideration. For purpose of practical application, SCS suggests three levels of AMC as follows:

AMC-I: Soils are dry but not to wilting point. Satisfactory cultivation has taken place.

AMC-II: Average conditions.

AMC-III: Sufficient rainfall has occurred within the immediate past 5 days. Saturated soil conditions prevail.

The limits of these three AMC classes, based on total rainfall magnitude in the previous 5 days, are given in Table 6.2. It is to be noted that the limits also depend upon the seasons like growing season and dormant season are considered.

Table 6.2. AMC for determining the value of CN

AMC Type	Total Rain in Previous 5 days	
	Dormant season	Growing Season
I	Less than 13 mm	Less than 36 mm
II	13 to 28 mm	36 to 53 mm
III	More than 28 mm	More than 53 mm

6.3.1 Runoff Curve Number Determination

The determination of the CN value for a watershed is a function of soil characteristics, hydrologic condition and cover or land use. CN values for Hydrological soil cover (Under AMC-II conditions) for Indian conditions are given in Table 6.3. For watersheds with multiple soil types or land uses, an area-weighted CN should be calculated. Table 6.4 shows the CN values for fully developed and developing urban areas. For AMC condition I and III, the multiplying factors given in Table 6.5 are used to convert the curve number for respective AMC conditions at interval of 10. For other values of CN, multiplication factorcan be obtained after interpolation.

Table 6.3. Runoff curve numbers (AMC-II) for the Indian conditions

Sl. No	Landuse	Treatment/Practice	Hydrologic condition	Hydrologic soil group			
				A	B	C	D
1	Cultivated	Straight row	-------	76	86	90	93
		Contour	Poor	70	79	84	88
			Good	65	75	82	86
		Contour and terraced	Poor	66	74	80	82
			Good	67	75	81	83
		Bunded	Poor	59	69	76	79
			Good	95	95	5	95
		Paddy(rice)	-----				
2	Orchards	With under stony cover	-----	39	53	67	71
		Without under Stony cover	-----	41	55	69	73
3	Forest	Dense	-----	26	40	58	61
		open	-----	28	44	60	64
		shrubs		33	47	64	67
4		------	Poor	68	79	86	89

	Pasture		Fair	49	69	79	84
			Good	39	61	74	80
5	Wasted Land	------	-----	71	80	85	88
6	Hard surface	------	------	77	86	91	93

**Table 6.4. Runoff curve number (values for fully developed and developing urban areas)**

Cover description		Curve numbers for hydrologic soil group			
Cover type and hydrologic condition	Average % impervious area	A	B	C	D
Open space (lawns, parks, golf courses, cemeteries, etc.): Good condition (grass cover > 75%) Fair condition (grass cover 50% to 75%) Poor condition (grass cover less than 50%)		39 49 68	61 69 79	74 79 86	80 84 89
Impervious areas: Paved parking lots, roofs, driveways, compacted gravel, etc.		98	98	98	98

(excluding right-of-way)					
Small open spaces within developments or ROW:		72	82	87	89
Streets and roads: Paved: curbs and storm sewers (including right-of-way) Paved: open ditches (including right-of-way) Gravel (including right-of-way) Dirt (including right-of-way)		90 83 76 72	93 89 85 82	95 92 89 87	97 93 91 89
Urban districts: Commercial and business Industrial	85 72	89 81	92 88	94 91	95 93
Residential districts by average lot size: 1/8 acre or less (townhouses) 1/4 acre 1/3 acre 1/2 acre 1 acre 2 acres	65 38 30 25 20 12	77 61 57 54 51 46	85 75 72 70 68 65	90 83 81 80 79 77	92 87 86 85 84 82
Developing urban areas: Newly graded areas (pervious areas only, no vegetation)		77	86	91	94

Table 6.5. Multiplication factor for converting AMC II to I and III conditions

Curve number/ weighted curve number for AMC II	Factors to convert from AMC II to	
	AMC I	AMC II
10	0.40	2.22
20	0.45	1.85
30	0.50	1.67
40	0.55	1.50
50	0.62	1.40
60	0.67	1.30
70	0.73	1.21
80	0.79	1.14
90	0.87	1.07
100	1.00	100

Example 6.1: In a watershed shown in the figure below, a water harvesting structure is planned to construct at point A. The catchment area to this point is 137 ha, out of which 78 ha area is under groundnut cultivated in straight row, 29 ha area is under fodder cultivation and remaining area is covered with tree plantation. The prevailing soil type of the catchment is vertisol and rainfall analysis suggested 86.4 mm 6-hours duration rainfall can be expected for 25 years return period and experience frequent rainfall in the season. Determine the potential runoff volume that can be generated from this catchment.
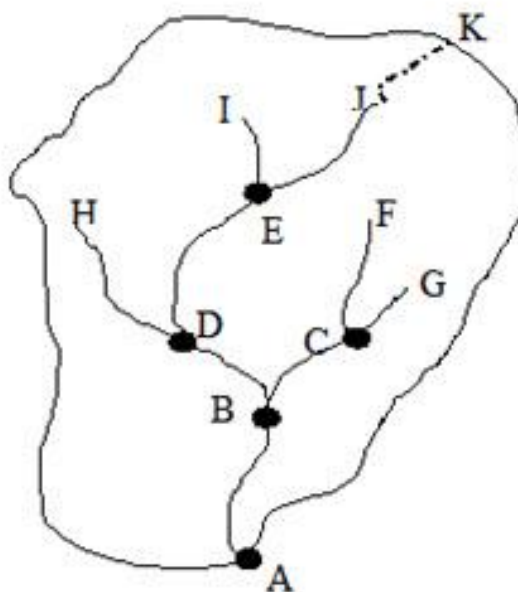
Solution:

Since the soil type is vertisol (black soil), the hydrologic soil group of this catchment is D. Using Table 3 the CN values for ground nut cultivation, fodder cultivation and plantation would be 93, 80 and 73 respectively.

Hence the weighted curve number would be

$$= (93×78+80×29+73×30)/137 = 85.86 \text{ or } 86(\text{say})$$

Since the area experience frequent rainfall, the AMC conditions would be III.

By linear interpolation, the multiplying factor for CN value of 86 would be 1.1. Hence the converted CN value would be 86×1.1 = 95.6

Now, compute S, we know that $$CN = 25400/(S + 254)$$

$$S + 254 = 25400/95.6$$

Therefore,   S = 11.7

Since the soil type is black soil, I_a will be equal to 0.25S

Now

$$Q = \frac{(P-0.25S)^2}{(P+0.75S)} = \frac{(86.4-0.25 \times 11.7)^2}{(86.4+0.75 \times 11.7)}$$

= 73.2 mm

Total volume of storage structure when all the runoff to be stored will be 10ha-m approximate

Lesson 7. Runoff Measurement using Weirs

7.1 Introduction

Runoff information is needed for design of various soil and water conservation structures. In the previous lectures (5 and 6), peak runoff rate as well as runoff volume estimations are discussed. Runoff is estimated because it is not always possible to obtain field measured data, which is costly and time consuming. However, there are many methods of runoff measurements. In field, it is generally carried out using current meters and calibrated or rated channel cross sections. The flumes or standardized weirs, together with water level readings or automatic recorders are also used. In this lesson only weirs will be discussed.

7.2 Weirs

Weirs consisting of a barrier are dam structures of known hydraulic specifications, placed across the stream to constrict the flow, and allow the water to flow over a crest. It is a calibrated instrument used to measure the flow in an open channel, or the discharge of a well or a canal outlet at the source.Weir openings may be rectangular, trapezoidal, or triangular in cross section. The basic formula for calculating discharge through a weir is

$$Q = CLH^m \qquad (7.1)$$

Where Qis the discharge over weir, C isthe coefficient dependent on the nature of crest and approach conditions, L is the length of crest, H is the head of crest, m is an exponent depending upon the weir opening.

7.3 Classification of Weirs

Weirs are classified based on the shape of their opening or notch. The edge of the opening can be either sharp or broad-crested.

7.3.1 Broad-Crested Weirs

A weir that has a horizontal or nearly horizontal crest sufficiently long in the direction of the flow so that the nappe will be supported and hydrostatic pressures will be fully developed for at least a short distance. Broad crested weir is shown in Fig. 7.1.
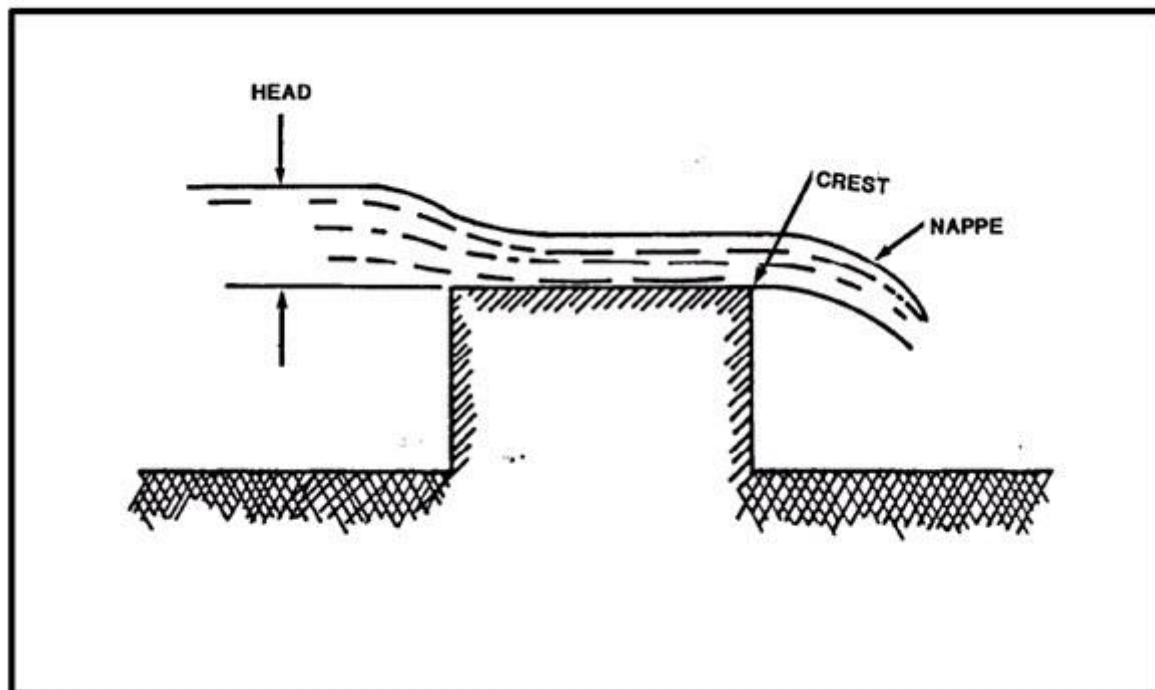
Fig. 7.1 Broad crested weir.

Broad crested weirs are recommended to overcome the difficulty of maintaining the sharp edge of weirs under field conditions for longer periods. Its discharge depends upon the shape of the weir and the crest, which define the nappe characteristics. Eqns. (2) and (3) represent the general formula for determination of maximum discharge considering the velocity approach and without velocity approach respectively

$$Q_{max} = 1.71 C_d L (H_1^{\frac{3}{2}} - H_V^{\frac{3}{2}}) \qquad (7.2)$$

$$Q_{max} = 1.71 C_d \ L \times H^{3/2} \qquad (7.3)$$

Where Q_{max} is the maximum discharge over the weir, C_d is the coefficient of discharge, L is the length of the weir, H is the head of water above the crest, H_v is the velocity head, H_1 is the total head ($H_1 = H + H_v$) and V is the velocity of flow over the weir.

Example 7.1: Determine the maximum discharge over a broad-crested weir 60 meters long having 0.6 m height of water above its crest. Take coefficient of discharge as 0.595. Also determine the new discharge over the weir, considering

the velocity of approach. The channel at the upstream side of the weir has a cross-sectional area of 45 sq meters.

Solution:

Given, L = 60 m; H = 0.6 m; C_d = 0.595; A = 45 m²

Maximum discharge over the weir without considering the velocity of approach

We know that the maximum discharge over the weir,

$$Q_{max} = 1.71 C_d \; L \times H^{\frac{3}{2}}$$

$$H_v = \frac{V^2}{2g}$$

$$Q_{max} = 1.71 \times 0.595 \times 60 \times 0.6^{3/2}$$

$$Q_{max} = 28.4 \; m^3/s$$

Maximum discharge over the weir considering the velocity of approach

We know that velocity of approach,

$$V = \frac{Q}{A} = \frac{28.4}{45} = 0.63 m/s$$

and the head due to velocity of approach,

$$H_v = \frac{V^2}{2g} = \frac{(0.63)^2}{2 \times 9.81} = 0.02 \; m$$

Therefore, Total head, H1 = H + Hv = 0.6 + 0.02 = 0.62 m

We also know that the maximum discharge over the weir,

$$Q = 1.71 C_d L \left(H_1^{\frac{3}{2}} - H_V^{\frac{3}{2}} \right)$$

$$Q = 1.71 \times 0.595 \times 60 \times \left(0.62^{\frac{3}{2}} - 0.02^{\frac{3}{2}} \right)$$

$$Q = 29.63 \ \text{m}^3/\text{s}$$

Maximum discharge over the weir = 29.63 m³ /s.

7.3.2 Sharp-Crested Weir

These are generally used for water measurement on the farm or small streams. These can be of three types based on the shape of notch. These are

- Rectangular Weir

- Cipoletti Weir or Trapezoidal Weir

- V Notch Weirs or Triangular Weir

The commonly used terms with sharp created weirs are shown in Fig. 7.2.
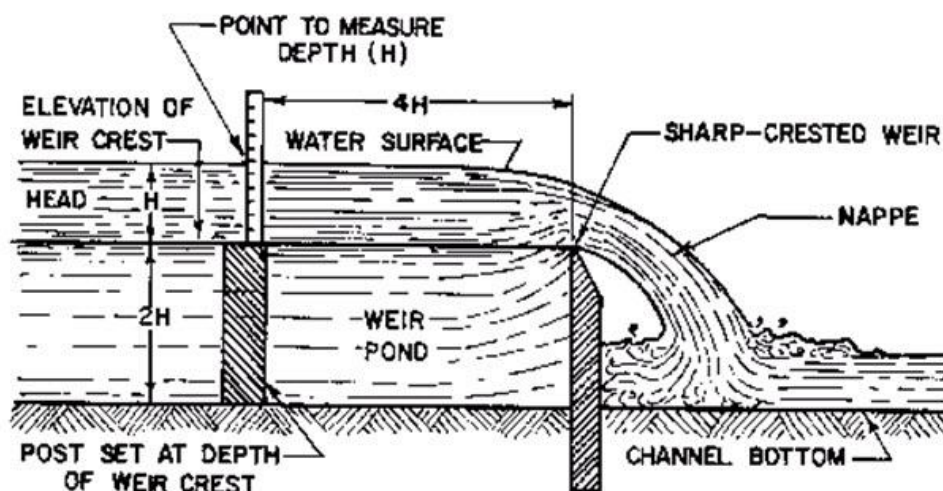

Fig. 7.2. Profile of a Sharp-crested weir.

7.3.2.1 Rectangular Weirs

Rectangular weir takes its name from the shape of its notch. The discharge through a weir or notch is directly related to the water depth (H), (Fig.7.3) and H is known as the head. This head is affected by the condition of the crest, the contraction, the velocity of approaching stream and the elevation of the water surface downstream from the weir. Rectangular weirs can be suppressed, partially contracted, or fully contracted.
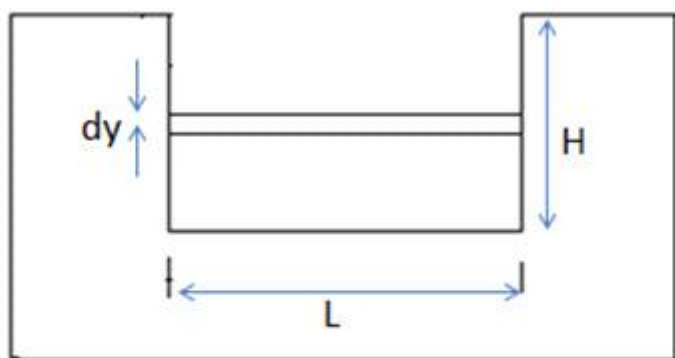

Fig. 7.3. Definition sketch for rectangular weir.

Considering theFig.7.4, discharge through the rectangular weir is calculated as

$Q = 0.0184LH^{3/2}$ (7.4)

For one side contraction $Q = 0.0184 (L0.1H)$ (7.5)

For both side contraction $Q= 0.0184(L0.2H)$ (7.6)

Example 7.2: Water flows through a contracted rectangular weir 120 cm long to a depth of 30 cm, it then flows along a rectangular channel 150 cm wide and over a second weir which has length equal to the width of the channel. Determine the depth of water over the second weir.

 Solution:

The first weir is contracted, i.e. both end contracted.

Given, Length of the weir (L) = 120 cm

Depth of water over the weir (H) = 30 cm

$$\text{So, discharge of flow} = 0.0184(L - 0.2H)H^{\frac{3}{2}}$$

$$= 0.0184 \times (120 - 0.2 \times 30) \times 30^{\frac{3}{2}}$$

= 344.67 liters/sec

In second weir, length of the weir (L) =150 cm

Discharge through first weir and second weir is same.

Let assume, depth of water over second weir is = H cm

Now,

$$Q = 0.0184LH^{\frac{3}{2}}$$

344.67 = 0.0184×150×H³/²

H= 24.97 cm. **Ans.**

So, the depth of water over the second weir is 24.97 cm.

7.3.2.1.1 Suppressed Rectangular Weir

A rectangular weir whose notch or opening sides are coincident with the sides of the approached rectangular channel and extend unchanged downstream from the weir. The weir goes across the entire channel width for a suppressed rectangular, sharp-crested weir. Fig. 7.4 (a) and (b) shows suppressed and contracted rectangular weirs.

Fig. 7.4. (a) Suppressed and (b) Contracted rectangular weir.
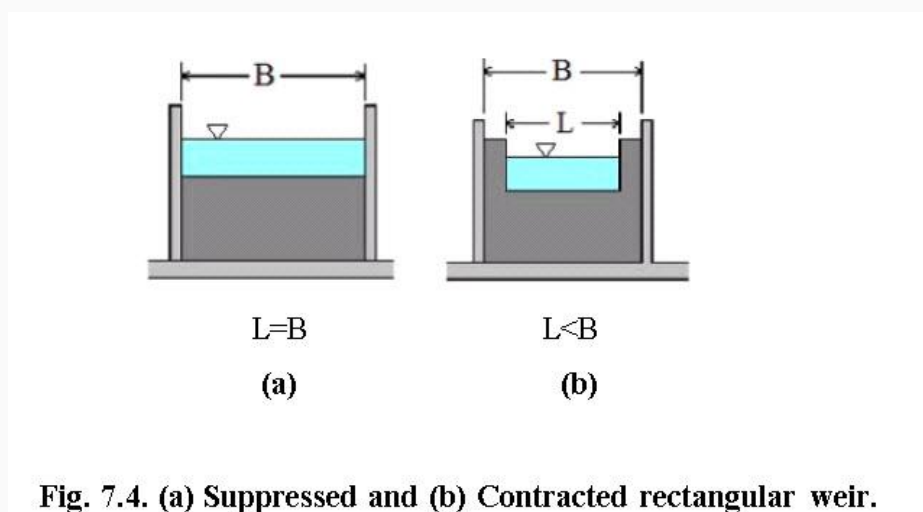
7.3.2.1.2 Contracted Rectangular Weir

A contracted rectangular sharp-crested weir, as shown in Fig. 7.4(b) has weir length less than the width of the channel. This type of rectangular weir is sometimes called an unsuppressed rectangular weir. The sides and crest of a weir are far away from the sides and bottom of the approach channel. The nappe will fully contract laterally at the ends and vertically at the crest of the weir.

7.3.2.2 Cipoletti Weirs

The Cipolletti weir (Fig. 7.5) is trapezoidal in shape. The slope of the sides, inclined outwardly from the crest, should be one horizontal to four vertical. The selected length of notch (L) should be at least 3H and preferably 4H or longer. Cipoletti weirs are considered fully contracted.
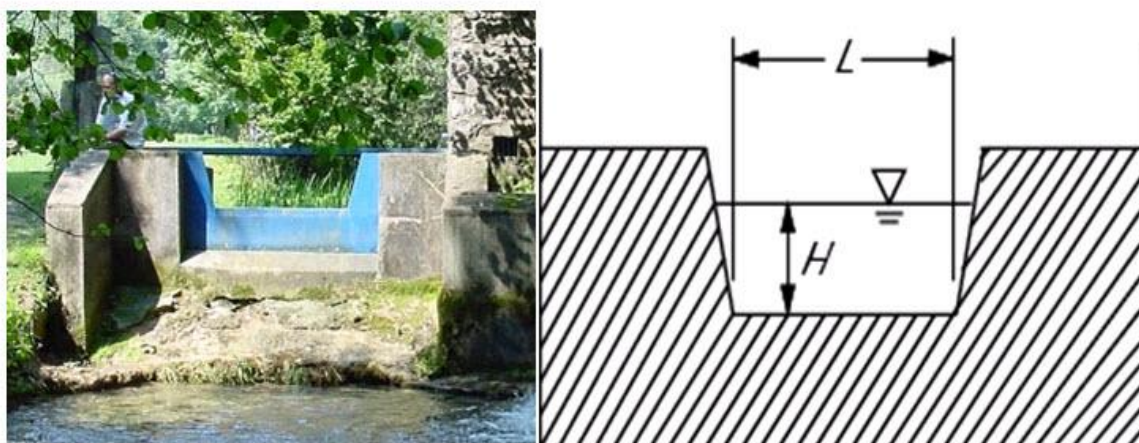

Fig. 7.5. Cipoletti weirs.

Discharge through Cipoletti weir is computed by the following formula:

$$Q = 0.0186 L H^{\frac{3}{2}} \qquad (7.7)$$

Example7.3: A Cipoletti weir has a breadth of 60 cm at its crest. The head of water flowing over the crest is 30 cm. Determine its discharge.

Solution:

Given, Crest width (L) = 60 cm

Head of flow over the crest (H) = 30 cm

So, discharge through the weir (Q) $=0.0186 L H^{\frac{3}{2}}$

$$Q = 0.0186 \times 60 \times 30^{\frac{3}{2}}$$

$$Q = 183.37 \text{ liters/sec}$$

Discharge of Cipoletti weir is 183.37 litres/sec. **Ans.**

7.3.2.3 V- Notch Weir

A V-notch weir (sometimes called a triangular weir) is shown in Fig. 7.6. This weir is especially good for measuring low flow rates. The flow area decreases as H increases, so a reasonable head is developed even ata very small flow rate. Depth of water above the bottom of the V is called head (H). Its design causes small changes in discharge hence causing a large change in depth and thus allowing more accurate measurement than with a rectangular weir. Head (H) should be measured at a distance of at least 4H upstream of the weir.

The most common weirs are 90° weirs, for larger flow measurements, and 45° weirs for medium-range flow measurements. Sometimes, 60° weirs are used for intermediate to larger flows, and 30° or 22.5° weirs are used for very small flows.
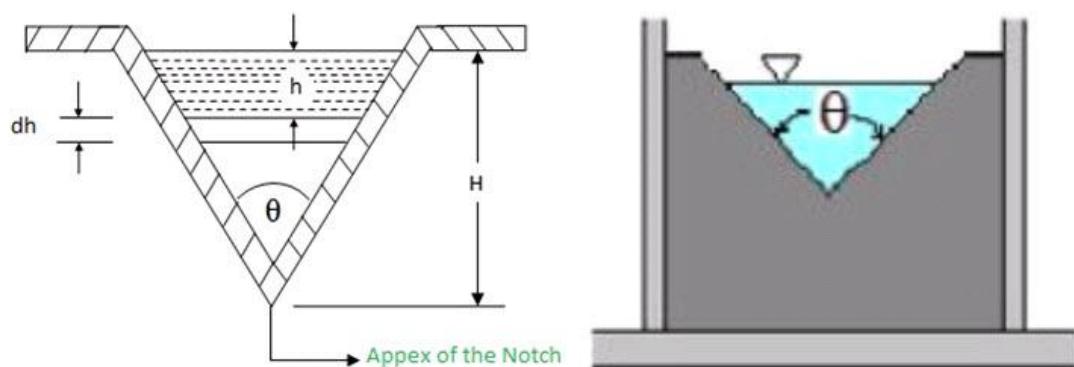

Fig. 7.6. V-notch weir.

Considering the Fig. 7.6, the discharge over a V-Notch sharp-crested weir is calculated as

$$Q = C.H^{\frac{5}{2}} \qquad (7.8)$$

Where,

$$C = \frac{8}{15}\sqrt{2g}\tan\left(\frac{\theta}{2}\right) \qquad (7.9)$$

If $\theta = 90^0$

$$C = \frac{8}{15}\sqrt{2g} \qquad (7.10)$$

For $\theta = 90^0$, H in cm and Q in litres/sec

$$Q = 0.0138 H^{\frac{5}{2}} \qquad (7.11)$$

In SI units:

$$Q = \frac{8}{15}C_d\sqrt{2g}\tan(\theta/2)\ H^{5/2} \qquad (7.12)$$

Where Q is the discharge over weir (m^3/sec), C_d is the Coefficient of discharge, θ is the angle of notch (degrees) and H is the head above bottom of notch (m).

Example 7.4: Determine discharge of 90° V-notch having 30 cm head of flow.

Solution:

Discharge through V-notch $\qquad$ Q=0.0138

$\qquad\qquad\qquad\qquad$ Q = 0.0138

$\qquad\qquad\qquad\qquad$ Q = 68.02 litres/sec

So, discharge through 90° V-notch having 30 cm head of flow is 68.02 litres/sec.

Sharp crested weirs may also be classified as suppressed and contracted.

7.4 Operation of Weirs

Properly constructed and installed weirs provide most accurate flow measurement. However, improper setting and operation may result in large errors in the discharge measurements. To ensure reliable results in measurement, the following precautions are necessary in the use of weirs.

1. The weirs should be set at lower end of a long pool sufficiently wide and deep having smooth flow at velocities less than 15cm/sec.

2. Baffles may be put in weir pond to reduce velocity.

3. The weir wall must be vertical.

4. The center line of the weir should be parallel to the direction of flow.

5. The crest of weir should be level so that water passing over it will be of the samedepth at all points along the crest.

6. Notch should be of regular shape and its edge must be rigid and straight.

7. The weir crest should be above the bottom of the approach channel.

8. The crest of weir should be placed high enough so that water will fall freely below weir.

9. The depth of water flow over the rectangular weir should not less than about 5 cm and not more than about 2/3 crest width.

10. The scale or gauge used for measuring the head should be located at a distance of about four times the approximate head. Zero of scale should be exactly at the same level as the crest level of the weir.

7.5 Limitations of Weirs

1. Weirs are not always suitable for measuring flow. Sufficient head is required for operating any type of weir.

2. They are not accurate unless proper conditions are maintained.

3. They require a considerable loss of head which is mostly not available in channels on flat gradients.

4. Weirs are not suitable for water carrying silt.

Weirs are not easily combined with turnout structures.

Lesson 8. Runoff Measurement using Flumes

8.1 Flumes

A flume is a stabilized channel section for measuring the flow. They are less inclined than weirs which make them well suited for runoff measurement. They require a very low head loss for operation. Examples of flumes are Parshall flume, H-flume, cut-throat flume, long-throated flume and venturing flume. They can be a "flat-bottom" type. In the case of a flat-bottom flume, the shape of the side walls creates a contraction of the flow of liquid (ex. Cut-throat flume). They can also combine vertical and side contractions (ex. Parshall flume).

Flumes work according to the venturi principle by reducing the flow area, velocity increases and water depth changes. A flume usually has three sections: a converging section, throat section and diverging section. Sizes vary according to the type and shape of flume. For practical purposes, to determine the absolute flow of a flume, the calibration curves supplied by the manufacturer should be used.

Advantages

- Minimal drop in pressure.

- Enables measurement in a large range of flow.

- The flow-rate in flumes is usually high enough to prevent sedimentation; they are therefore self-cleaning.

- Provides a reliable measurement in free flow and submerged flow conditions.

Disadvantages

- Installation is usually expensive.

- Installation requires extremely careful work.

- Requires a secure watertight base.

- Flow at the entrance must be evenly distributed, with little turbulence, to produce accurate measurements.

8.2 Parshall Flume

Parshall flumes are devices for the measurement of flow of water in open channels when depth of flow is less i.e., head drop is very small, the volume of flow is less and channel bed slope is less. Fig. 8.1 shows Parshall flume. The flume consists a converging section with a level floor and walls converges towards the throat section, a throat section with a downward sloping floor and parallel walls, and a diverging section with an upward sloping floor and diverging walls towards the

outlet. The size of flume is determined by the width of its throat. The size ranges from 7.5 cm to several m in throat width.
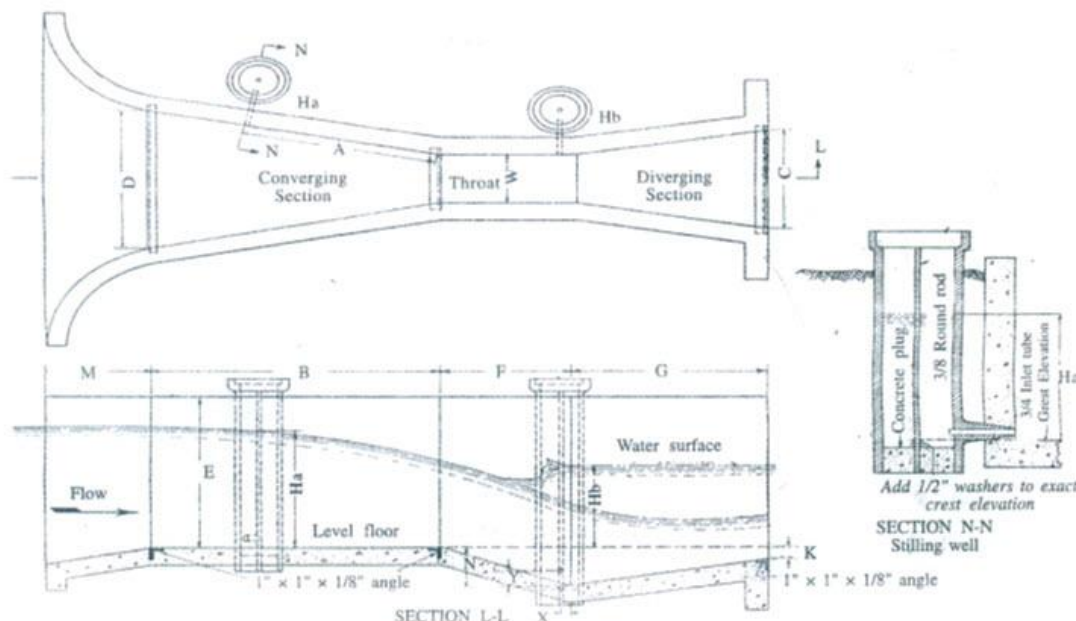

Fig. 8.1. Plan and elevation of a concrete Parshall flume showing component parts. (Source: Michael, 2009)

Parshall flumes are available in various sizes. Care must be taken while constructing the flumes exactly in accordance with structural dimensions.

On the basis of the throat width, Parshall flumes have been classified into three main groups.

(i) Very small - 25.4 mm to 76.2 mm.

(ii) Small 152.40 mm to 2438.4 mm.

(iii) Large 3048 mm to 15240 mm.

Standard dimensions of Parshall flumes with discharge values are presented in Table 8.1 and 8.2, respectively. Discharge through the flume can occur under either free or submerged flow condition. Flow is submerged when the down-stream water elevation retards the rate of discharge. To determine discharge through the flume under free flow condition, head is measured at upstream section (H_a). However, downstream head (H_b) is also measured for submerged flow condition. Free flow condition prevails if the submergence ratio (H_b/H_a) remains within 0.5, 0.6 and 0.7 for width of throat varying from 2.5 to 7.5 cm, 1.5 to 22.5 cm and 3.0 to 24.0 cm, respectively.

Table 8.1. Dimensions and capacities of Parshall flume of various sizes (Letter, refer Fig. 8.1) (Source: Michael, 2009)

Throat width	A	B	C	D	E	F	G	K	N	X	Y	Free-flow capacity	
cm	cm	cm	cm	cm	cm	cm	cm	cm	cm	cm	cm	Minimum, litres / second	Maximum, litres / second
7.5	31	46	18	26	46	15	30.5	2.5	5.7	2.5	3.8	0.85	28.4
15	41.4	61	39	39.7	61	30.5	61	7.6	12	5.1	7.6	1.4	110.8
23	58.8	86	38	57.5	76	30.5	45.5	7.6	12	5.1	7.6	2.5	253
30	91.5	134	61	84.5	92	61	91.5	7.6	23	5.1	7.6	3.13	456.6

Table 8.2. Free flow discharge values for Parshall Flume.

	Discharge, litres-per second			
Head	Throat width			
cm	7.5 cm	15 cm	23 cm	30 cm
3	0.8	1.4	2.6	3.1
4	1.2	2.3	4	4.5

5	1.7	3.3	5.5	7
6	2.3	4.4	7.2	9.6
7	2.7	5.4	8.5	11.4
8	3.4	7.2	11.1	14.4
9	4.3	8.5	13.5	17.7
10	5	10.2	15.9	21.1
11	5.8	11.6	18.1	23.8
12	6.7	13.5	21.1	27.5
13	7.5	15	23.3	31
14	8.5	17.3	26.7	35
15	9.4	19.2	29.5	38.7
16	10.4	21.2	32.5	42.7
17	11.4	23.2	35.6	46.6
18	12.4	25.3	39	51.2
19	13.6	27.8	42.5	55
20	14.3	30	45.8	59.7
21	15.8	32.7	49.3	64.7

22	17.1	35.2	53.3	69.8
23	18.2	37.7	56.8	74
24	19.4	40.1	60.5	79
25	20.7	42.7	64.5	84.1
26	22	45.7	69.3	89
27	23.3	48.1	72.4	94.3
28	24.8	51.5	76.7	100
29	26	54	80.7	105.1
30	27.5	57.3	85.2	111

8.2.1 Flow Measurement

Generally, Parshall flumes are calibrated empirically in laboratory conditions before installing in the field. However in case of free flow conditions, the discharge is measured by the following formula:

$$Q = CH^n \qquad (8.1)$$

Where, Q is the discharge, C, n is the flume coefficients which vary with the size of the flume, and H is the measuring head.

Table 8.3 gives a set of standard values for the C, n for different dimensions (these co-efficient are in fps units so the calculated discharge would be in ft^3/s and head has to be in ft,)

Table 8.3. Value of C and n for different throat widths

Throat width	Coefficient (*C*)	Exponent (*n*)
1 in	0.338	1.55
2 in	0.676	1.55
3 in	0.992	1.55
6 in	2.06	1.58
9 in	3.07	1.53
1 ft	3.95	1.55
2 ft	8.00	1.55
3 ft	12.00	1.57
4 ft	16.00	1.58
5 ft	20.00	1.59
6 ft	24.00	1.59
7 ft	28.00	1.60
8 ft	32.00	1.61
10 ft	39.38	1.60
12 ft	46.75	1.60
15 ft	57.81	1.60

20 ft	76.25	1.60
25 ft	94.69	1.60
30 ft	113.13	1.60
40 ft	150.00	1.60
50 ft	186.88	1.60

In the above figure, H_a is upstream head and H_b is downstream head.

Advantages:

i) This instrument is effective when the total head drop is small.

ii) Its operation is independent of approaching velocity.

iii) Being a self-cleaning device, it is not affected by sand or silt deposition.

8.2.2 Installation of Parshall flume

The Parshall flumes are installed considering the upstream condition, flume crest and downstream channel

1. **Upstream conditions:** Upstream conditions should promote laminar flow conditions at the flume inlet. Channel turns, tees, elevation drops or other obstructions should be avoided. The upstream channel slope should not allow excessive velocity at   the flume. A slope of almost flat, to 3% maximum, for very small flumes, and 2% maximum for larger flumes is the ideal slope value. A 1:4 sloping ramp upstream should be provided for flumes that must be installed above the channel floor.

2. **Crest of the flume:** The crest of the flume (the floor of the converging section where depth measurements are made) must be level both longitudinally & transversely.

3. **Downstream channel:** The downstream channel should not permit submerged flow conditions to occur. Long, narrow, flat or undersized channels can result in a backwater effect at the flume and should be avoided. A large fall or steep slope immediately downstream of the metering station can eliminate the possibility of submerged flow conditions.

8.3 H Flumes

H flumes were designed in the mid-1930s by the USDA Agricultural Research Service. H-flumes (Fig.8.2) are devices to guide the natural flow to converge

through V-type cross section. In H flume, the high flows pass at wide opening whereas at low flow the opening is reduced to maintain the sensitivity of flow measurement. The H-flumes measure highly varying flow (as high as 100 times to the low flow) accurately.
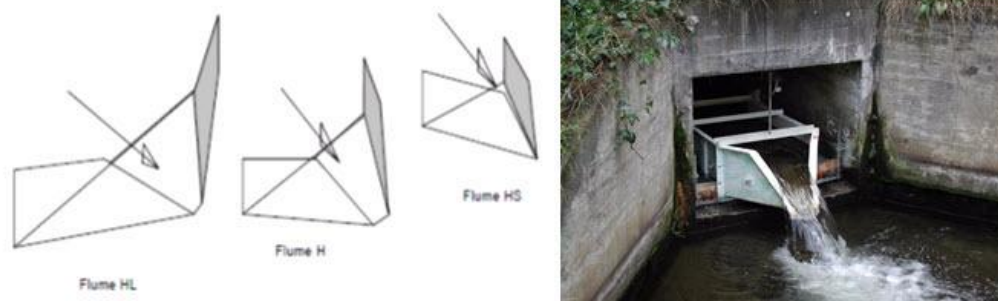

Fig. 8.2. Types of H-flumes.

Based on the capacity of flumes, there are three categories of H flumes: Hs, H and HL, all of which have the same shape, but differ in size and angles. Design parameters and dimensions for different flumes are given in Tables 8.4 and 8.5, respectively.

1. HS-Flumes: Of this 'small' category H-flume, the flow depth is restricted to 30cm and is capable to measure flow ranging between 0.005 to 23 l/s.

2. H-Flumes: Of this 'normal' category H-flume. In this case the maximum flow depth are 1.4m and the is able to measure flow ranging from 0.01 l/s to 2400 l/s.

3. HL-flumes: This is largest H-flume among the three categories. Though in this case also, the maximum flow depth is 1.4m, the HL-flume is wider than the normal H-flume and hence the accuracy is lesser. The use of this type of flume is only recommended if the anticipated discharge exceeds the capacity of the normal H-flume. The HL-flume is able to handle flow rate as high as 3300 l/s.

Table 8.4. Design parameters of different type of H-flume

Type	W/H	L/H	T/H
HS	1.90	1.35	0.1
H	1.05	1.50	0.05
HL	3.20	1.50	0.2

Table 8.5. Dimension of different type of H-flume

Type	Flume Dimensions (feet)[1]			
	Height	Width	Length	Spout
	H	W	L	T
H	0.5	0.95	0.68	0.05
	0.75	1.43	1.01	0.08
	1.00	1.90	1.35	0.10
	1.5	2.85	2.03	0.15
	2.00	3.80	2.70	0.20
	2.50	4.75	3.38	0.25
	3.00	5.70	4.05	0.30
	4.50	8.55	6.08	0.45
HS	0.40	0.42	0.60	0.02
	0.60	0.63	0.90	0.03
HL	4.00	12.80	6.00	0.80

8.3.1 Description

An H flume is the result of a combination of the physical and mechanical characteristics of a weir and flume. Because of its shape, it resembles a triangular weir more than a flume. It consists of two sections: approach section shaped by converging sides, and the control section consisting an opening that is the result of shape of the converging sides.

8.3.2 Operating Principle

An H flume operates according to the Venturi principle. Due to lateral restrictions, the flume restricts the flow area, causing the water level upstream from the throat to rise. The flow can be obtained by simply measuring the water depth, because this depth varies proportionally with flow.

This type of flumes can be used under both free and submerged flow conditions. Their use in free flow conditions is strongly recommended. In a free flow condition, a flow measurement can be obtained using only one measuring point, but in submerged flow conditions, the depth downstream from the throat section must also be measured.

8.3.3 Applications

- An H flume was developed to measure the flow of irrigation water from small catchment areas and surface water. It is also used to measure the flow of irrigation water, slow-flowing watercourses and water in sewer systems.

- The geometry and operating principle of an H flume make it a very useful tool for measuring the flow of water that contains solids.

- It is easy to manufacture, relatively inexpensive and is suitable for temporary measurement systems.

8.3.4 Installation of H-flumes

When a flume is installed, it is important to ensure that the flume's physical characteristics correspond to recommended dimensions. When flumes are installed, the approach boxes should, whenever possible, be placed below the natural ground surface. Where the watershed or plot slope is small and the flow is dispersed, gutters may be provided to collect the runoff at the bottom of the slope and channel it into the approach box. Metal flumes should be fixed to the concrete approach. The concrete cut-off wall should extend below the concrete approach at the upstream face of the flume to provide substantial support and to prevent seepage below the flume. The flume floor must be level. If silting is a problem, a 1 in 8 sloping false floor can be set to concentrate low flows and thereby reduce silting.

Lesson 9. Open Channel Flow

9.1 Open-Channel Flow Definition

Open channel flow also called free gravity flow, is the flow of water induced by the effect of gravity. The surface of the flowing liquid is at atmospheric pressure and free to air. The example of open channel flow includes flow in canal, river etc. However, open channel flow example also includes half-closed pipe flow under the influence of gravity and flow at atmospheric pressure. The difference between open channel flow and pressurized flow can be summarized as:

1. In open channel flow, the liquid has free surface.

2. Open channel flow is subjected to gravity and atmospheric pressure.

3. Open channel flow is always under the action of gravity.

On the basis of cross-section, the open channel can be classified as natural channel in which the cross-section varies with length (e.g. river and streams etc) and manmade channel in which the cross-section does not change significantly over length and intended to carry water for various purposes. The manmade channel further classified as rectangular channel, triangular channel, parabolic channel, circular channel and trapezoidal channel.

9.2 Types of Flows in Open Channel

Open channel are classified in to following classes depending on the relative change in the flow regime with respect to time and space.

(a) Steady Flow and Unsteady Flow

If the flow characteristics do not change over time at any point, the flow is called steady flow whereas if the flow characteristics are changed with time, the flow is called unsteady flow. Mathematically, for steady flow the conditions $\partial v/\partial t$ and $\partial v/\partial x$ should be equal to zero and vice-versa for unsteady flow.

(b) Uniform Flow and Non-Uniform Flow

The flows are classified under these classes when space is considered as the criteria. When the depth of flow is unchanged over different section of the channel, the flow called as uniform flow. Whereas the depth of flow changes with section, the flow classified as varied flow. Fig 9.1 shows the uniform flow and the varied flow. The varied flow is further classified as rapidly varied flow (RVF) and gradually varied flow (GVF). In case of RVF, the flow changes abruptly over a short distance such as hydraulic jump. In case GVF, the flow changes gradually over the length in a long reach of the channel, such as irrigation canal network.
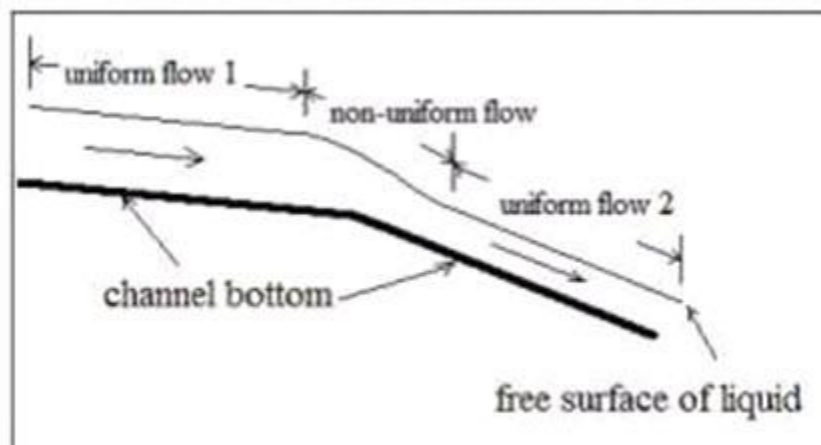
Fig. 9.1. Uniform flow and non-uniform (Varied) flow.

The following figure describes the classification of flow in different section of channel. The flow characteristics are changed due to different flow conditions are illustrated in the Fig.9.2.
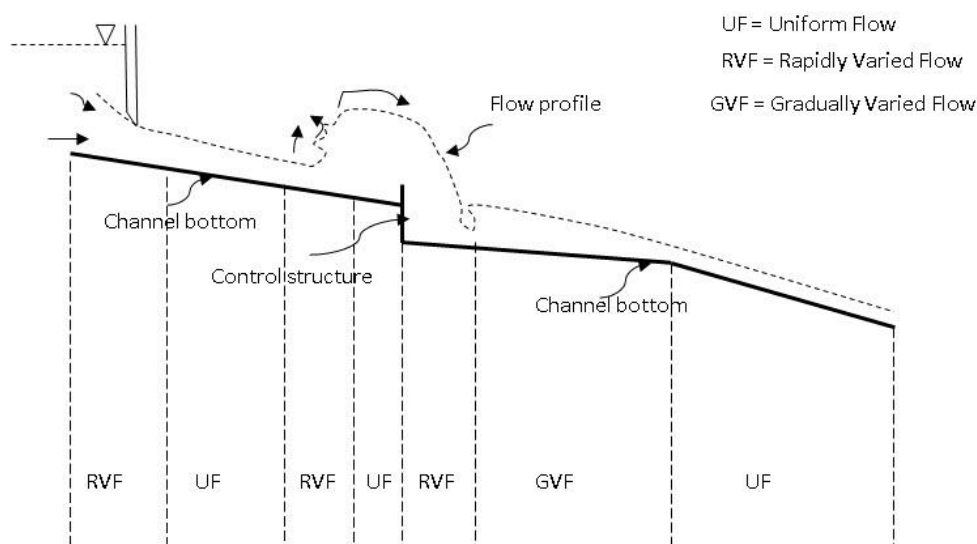

Fig. 9.2. Variation in flow characteristics due to change in flow conditions.

(c) Laminar and Turbulent Flow

The Reynolds number (R_e) is used to characterize the flow as laminar, turbulent or in transition state. The Reynolds number is defined as the ratio of viscous force to inertia force. If R_e is less than 500, the flow is laminar, for R_e in between 500 to

2000, the flow is in transitional state and the flow is turbulent if R_e is greater than 2000 (Modi and Seth, 1991).   The reynold number is given as

$$R_e = \frac{\rho v R}{\mu} \qquad \qquad (9.1)$$

Where, R_e is the reynold number, ρ is mass density of water, v is mean velocity of water R is hydraulic radius and μ is absolute viscocity. The hydraulic radius, R, also called as hydraulic mean depth, is the ratio of wetted area to its wetted perimeter.
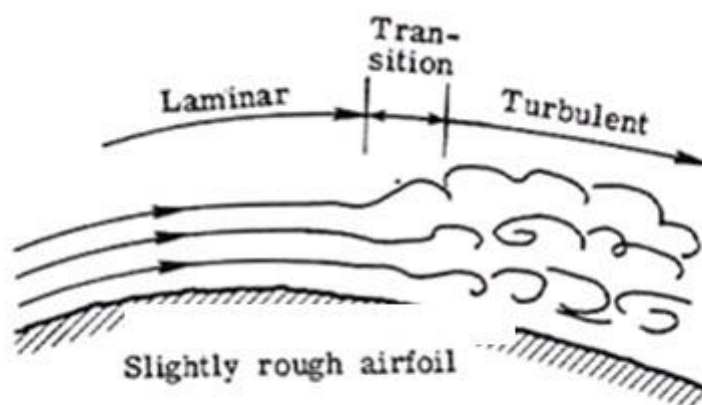

Fig. 9.3. Laminar, transitional and turbulent flow.

In laminar flow the fluid particles will appear to move in definite smooth paths or streamlines. Laminar flow is known to occur in shallow overland or sheet flow conditions. Turbulent flow is the most common type occurring in open channel drainage facilities, it is the type considered for most hydraulic procedures.

(d)   Subcritical, Critical and Super Critical Flow

The regimes of flow which may be present in an open channel are subcritical flow, critical flow and supercritical flow. This flow regime which exists under a given set of conditions may be determined by calculation of the Froude Number. The Froud

number defines the relative effect of gravity and inertia forces causing water to flow and is expressed as:

$$F_v = \frac{V}{\sqrt{gD}} \qquad (9.2)$$

Where is Froud number, is mean velocity of flow and is hydraulic depth. Hydraulic depth is defined as the ration of wetted area to the top width.

Subcritical flow :exists when the Froude Number is less than 1.0. Subcritical flow is characterized by low velocities and large depths. When a channel is operating in the subcritical flow regime, the formation of a hydraulic jump is not possible. If the channel cross-section and the discharge are constant, then the regime of flow becomes a function of the slope of the channel. In this case, the slope of the channel defines the flow regime and the slope may be referred to as either subcritical slope, critical slope, or supercritical slope. In order for subcritical flow to occur, the channel slope must also be subcritical. Critical flow will occur when the Froude Number is equal to 1.0. For a given channel cross section and discharge, critical flow will occur when the channel slope is equal to the critical slope. = velocity distribution coefficient

Supercritical Flow: Supercritical flow will occur when the Froude Number is greater than 1.0. Supercritical flow is characterized by high velocities and shallow depths. When a channel is operating in the supercritical regime, the formation of a hydraulic jump is possible. A hydraulic jump will occur when the flow regime changes from supercritical to subcritical in a short distance. For example, a hydraulic jump will occur in a channel if the channel slope abruptly changes from a supercritical value to a subcritical value.

Lesson 10 Energy and Momentum Equation

This chapter deals with three equations commonly used in fluid mechanics and soil and water conservation: the conservation of mass, conservation of energy, and conservation of momentum. These equations have great importance in hydraulic structure design, soil and water conservation structures design, canal simulations, irrigation systems design etc. We now give a brief description of the conservation of mass, energy and momentum equations.

- **Conservation of Mass** – This principle says that the mass neither be created nor destroyed. This forms the basis for continuity equation

- **Conservation of Energy** – this principle says that the energy neither be created nor destroyed, only can change the form. This principle forms the basis for energy equation

- **Conservation of Linear Momentum** – The interaction of force and time acting on the object is equal to the change in momentum of the object.

10.1 Mass in Open Channel Flow

The conservation of mass or continuous steady flow is expressed mathematically in the basic continuity equation as:

$$Q = A \times V \tag{10.1}$$

Where, Q is the discharge, in m^3/sec ;A is the cross-sectional area, in and V is the average channel velocity, in $m/sec.$

10.2 Energy in Open Channel Flow

The total energy head at a point in an open channel is the sum of the potential and kinetic energy of the flowing water. The energy per unit weight is the sum of pressure head or static head, velocity head or kinetic head and potential head. For steady irrotational flow these sum are constant. These explained by the following equation.

$$\frac{p}{w} + \frac{v^2}{2g} + z = C \tag{10.2}$$

Equation 2is commonly known as Bernoulli's equation. The term $\frac{p}{w}$ is pressure head, ratio of pressure (p) and specific weight (w); $\frac{v^2}{2g}$ is kinetic head

proportional to square of velocity (v), and z is the potential head. All term has the units in m.

As per Bernoulli's principle, the total energy in steady, irrotational flow is constant at any point, thus

$$\frac{p1}{w} + \frac{v1^2}{2g} + Z_1 \qquad = \frac{p2}{w} + \frac{v2^2}{2g} + Z_2 \qquad (10.3)$$

If a tube with a 90 degree elbow is inserted into the flow with the open end pointing into the flow, then the water level will rise to a level higher than the water surface elevation in the channel, and this distance is a measure of ability of the water velocity to do work. Newton's laws of motion can be used to determine that this distance is $\frac{v^2}{2g}$, where g is the acceleration due to gravity. Therefore the total energy head at a point in an open channel is. $h + Z + V^2/2g$.

A longitudinal profile of total energy head elevations is called the energy grade line. The longitudinal profile of water surface elevations is called the hydraulic grade line. The energy and hydraulic grade lines for uniform open channel flow are illustrated in figure10.1. For flow to occur in an open channel, the energy grade line must have a negative slope in the direction of flow. A gradual decrease in the energy grade line for a given length of channel represents the loss of energy caused by friction. When considered together, the hydraulic and energy grade lines reflect not only the loss of energy by friction, but also the conversion between potential and kinetic forms of energy.

As water flows down a channel, the flow loses energy because of friction and turbulence. The total energy head between two points in a channel reach can be set equal to one another if the losses between the sections are added to the downstream total energy head. This equality is commonly known as the Energy Equation, which is calculated as:
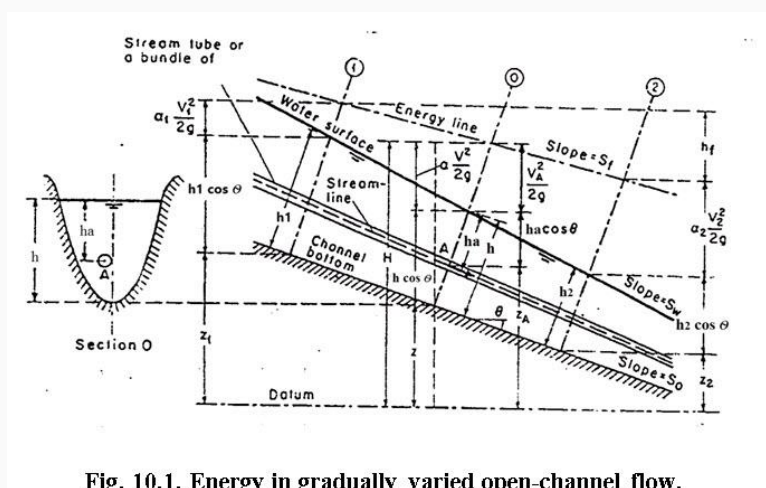

Fig. 10.1. Energy in gradually varied open-channel flow.

Total energy at section -1

$$H_1 = z_1 + h_1 \cos\theta + \alpha_1 \frac{v_1^2}{2g} \qquad (10.4)$$

Applying energy principle to section 1 and 2 we have.

$$z_1 + h_1 \cos\theta + \alpha_1 \frac{v_1^2}{2g} = z_2 + h_2 \cos\theta + \alpha_2 \frac{v_2^2}{2g} + h_f \qquad (10.5)$$

Where,

h_f = friction loss between section 1 and 2.

If the bed slope of the channel is small; i.e., $\Theta = 0$

$$z_1 + h_1 + \alpha_1 \frac{v_1^2}{2g} = z_2 + h_2 + \alpha_2 \frac{v_2^2}{2g} + h_f \qquad (10.6)$$

If $\alpha_1 = \alpha_1 = 1$ (uniform velocity distribution)

$$z_1 + h_1 + \frac{v_1^2}{2g} = z_2 + h_2 + \frac{v_2^2}{2g} + h_f \qquad (10.7)$$

Where,

d_1 and d_2 are depth of open channel flow at channel sections 1 and 2 respectively, in m; v_1 and v_2 = are average channel velocities at channel sections 1 and 2 respectively, in m/sec; z_1 and z_2 = are channel elevations above an arbitrary datum at channel sections 1 and 2, respectively, in m; hf = is the head or energy loss between channel sections 1 and 2 in m; Θ is the bed slope of the channel; g is the acceleration due to gravity, 9.8. h_f

The above equation is same as the well-known Bernoulli's equation.

10.3 Momentum Equation in Open Channel Flow

According to Newton's Second Law of Motion, the change of momentum per unit of time is equal to the result ant of all external forces applied to the moving body. Application of this principle to open channel flow produces a relationship which is virtually the same as the Energy Equation expressed in equation 10.7.

The momentum of the flow passing the channel section per unit time is expressed as

Momentum

$$= \beta \gamma Q V / g, \qquad (10.8)$$

Where B is the momentum coefficient, Y is the unit weight of water, Q is the discharge, and is V the mean velocity.

Applying the momentum principle for the momentum change per unit time in the body of water enclosed between section 1 and 2 may be written:

$$\gamma \frac{Q}{g}(\beta_2 V_2 - \beta_1 V_1) = P_1 - P_2 + W \sin \theta - F_f \qquad (10.9)$$

Where, p1 and p2 are the resultants of pressure acting on two sections; W is the weight of water enclosed between section 1 and 2; F_f is the total external force of friction and resistant action along the surface of contact between the water and the channel.

The slope of the channel is assumed relatively small. Thus, in the short reach of a rectangular channel of small slope and width
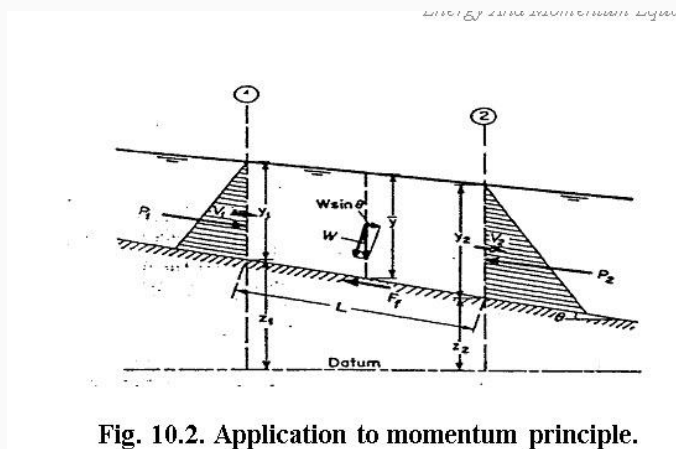

Fig. 10.2. Application to momentum principle.

Fig. 10.2. Application to momentum principle.

$$P_1 = \frac{1}{2}\gamma b y_1^2$$

$$P_2 = \frac{1}{2}\gamma b y_2^2$$

$$F_f = \gamma h_f b \bar{y}$$

Where,

$\bar{y}$ is the average depth $= (y_1 + y_2)/2$

$$Q = \frac{1}{2}(V_1 + V_2)b\bar{y}$$

Also, the weight of the body of water is

$$W = \gamma b \bar{y} L$$

$$\sin\theta = \frac{Z_1 - Z_2}{L}$$

Substituting all the above expression for the corresponding items in Equation (10.9) and simplifying,

$$z_1 + y_1 + \beta_1 v_1^2/2g = z_2 + y_2 + \beta_2 v_2^2/2g + h_f' \qquad (10.10)$$

Where, is the Losses due to external forces exerted on water by the walls of the channel. This equation appears to be practically same as the energy equation (10.7).

Lesson 11. Energy and Momentum

The total energy head at a point in an open channel is the sum of the potential and kinetic energy of the flowing water. The potential energy is represented by the elevation of the water surface i.e., the sum of depth of flow (h) and the elevation of channel bottom (z) from an arbitrary datum. The water surface elevation is a measure of the potential work that the flow can do during its movement from a higher elevation to a lower elevation. The kinetic energy is the energy of motion as measured by the velocity (v).

As water flows down a channel, the flow loses energy because of friction and turbulence. The total energy head between two points in a channel reach can be set equal to one another if the losses between the sections are added to the downstream total energy head. This equality is commonly known as the energy equation, as described in lesson 10 (equation 10.7) which is expressed as:

$$z_1 + h_1 + \frac{v_1^2}{2g} = z_2 + h_2 + \frac{v_2^2}{2g} + H_f \tag{11.1}$$

Where, andare channel elevations above an arbitrary datum at channel sections 1 and 2, respectively;andare depth of open channel flow at channel sections 1 and 2, respectively; and are average channel velocities at channel sections 1 and 2, respectively; is the head or energy loss between channel sections 1 and 2 and is the acceleration due to gravity.

 Now, as described in lesson 10, according to Newton's Second Law of Motion, the change of momentum per unit of time is equal to the resultant of all external forces applied to the moving body.

11.1 Specific Energy

The total energy of a flowing liquid per unit weight is given by,

$$E = z + h + \frac{v^2}{2g} \tag{11.2}$$

 Where, z is the height of the bottom of the channel from datum; $h$is the depth of liquid; v is the mean velocity of flow
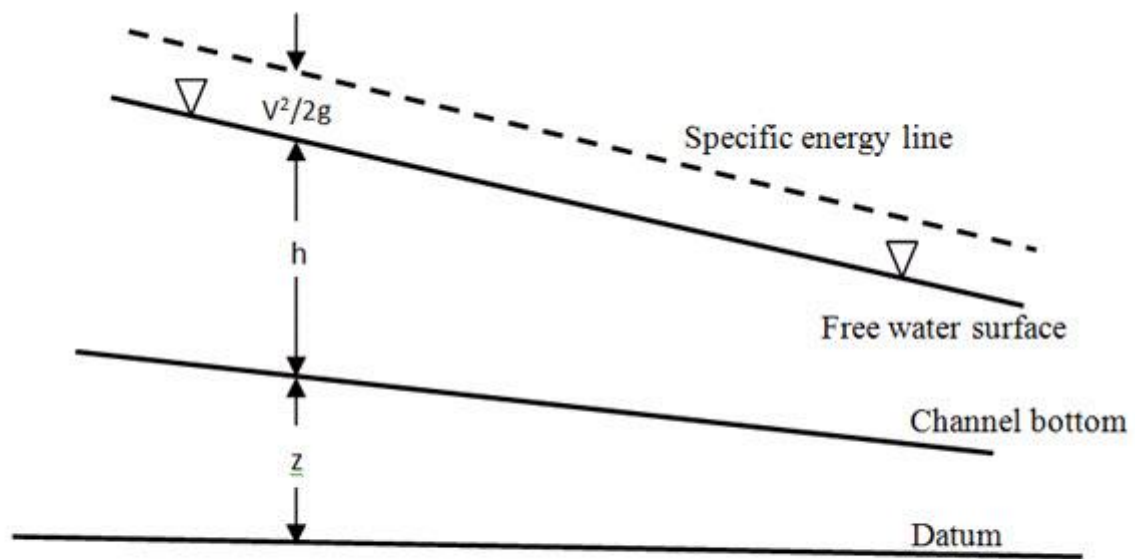
Fig. 11.1. Specific energy.

Fig. 11.1. Specific energy.

Consider the channel bottom as the datum, and then the total energy per unit weight of liquid will be,

$$E = h + \frac{v^2}{2g} \qquad (11.3)$$

Eq. (11.3) gives the specific energy of a flowing liquid. Therefore, the specific energy of a flowing liquid is the energy per unit weight of the liquid with respect to the bottom of the channel

11.2 Specific Energy Curve

It is the curve which shows the variation of specific energy with depth of flow. Specific energy curve is obtained as,

From eq. (11.3) Specific energy of a flowing liquid,

$$E = h + \frac{v^2}{2g} = E_p + E_k \qquad\qquad (11.4)$$

Where, E_P is the potential energy of flow (h) and E_k is the kinetic energy of flow ($v^2/2g$).

Now, consider a rectangular channel with a steady but non-uniform flow. Let Q is the discharge through the channel, b is the channel width, h is the depth of flow and q is the discharge per unit width i.e. Q/b

$$\text{Velocity of flow } (v) = \frac{\text{Disharge } (Q)}{\text{Cross sectional area } (b \times h)} = \frac{q}{h} \qquad (11.5)$$

Substituting the value of v in eq. (11.3), we get;

$$E = h + \frac{q^2}{2gh^2} \qquad\qquad (11.6)$$

Eq. (11.6) gives the variation of specific energy (E) with the depth of flow (h). Hence, for a given discharge Q, for different values of depth of flow, the corresponding values of E may be obtained. Then a graph between specific energy, E (along X-X axis) and depth of flow, h (along Y-Y axis) may be plotted.
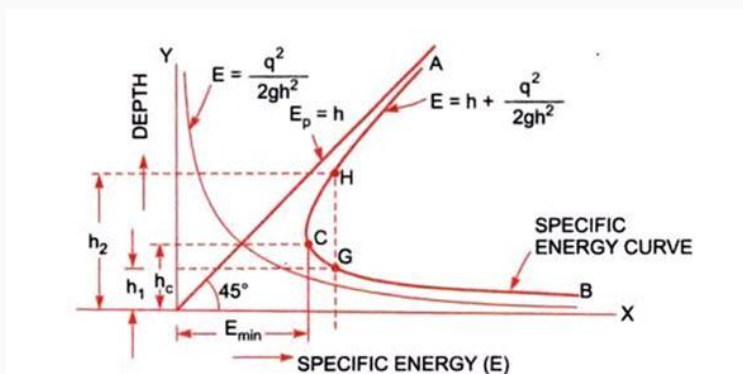

Fig. 11. 2. Specific energy curve. (Source: Bansal, 2009)

11.3 Critical Depth

Critical depth (h_c) is defined as the depth of flow of water at which the specific energy is minimum. In figure 11. 2, curve ACB is a specific energy curve and point C corresponds to the minimum specific energy. The depth of flow of water at C is known as critical depth. The mathematical expression for critical depth is obtained by differentiating the specific energy equation (11.6) with respect to depth of flow and equating the same to zero.

or

$$\frac{dE}{dH} = 0 \tag{11.7}$$

Where,

$$E = h + \frac{q^2}{2gh^2} \qquad \text{From eq. (11.6)}$$

or

$$h = \left(\frac{q^2}{g}\right)^{1/3} \tag{11.8}$$

But when specific energy is minimum, depth of flow is known as critical depth. Hence, critical depth is

$$h_c = \left(\frac{q^2}{g}\right)^{1/3} \tag{11.9}$$

11.4 Minimum Specific Energy in Terms of Critical Depth

From eq. (11.6), specific energy of a flowing fluid is given by,

$$E = h + \frac{q^2}{2gh^2} \qquad (11.10)$$

However, at critical depth the value of specific energy of a flowing fluid is minimum. Hence, the above eq. (11.10) for critical depth of flow can be written as,

$$E_{min} = h_c + \frac{q^2}{2gh_c^2} \qquad (11.11)$$

From eq. (11.8) we have,

$$h_c = \left(\frac{q^2}{g}\right)^{1/3} \text{ or } h_c^3 = \frac{q^2}{g} \qquad (11.12)$$

or

Substituting the value of in eq. (11.11), we get

$$E_{min} = \frac{3h_c}{2} \qquad (11.13)$$

11.5 Specific Force

Specific force may be explained from the momentum equation discussed in the Lesson-10 which is as follows;

$$\gamma \frac{Q}{g}(\beta_2 V_2 - \beta_1 V_1) = P_1 - P_2 + W \sin\theta - F_f \qquad (11.14)$$

Where, $_{p1}$ ansd $_{p2}$ are the resultants of pressure acting on two sections (i.e. hydrostatic forces); W is the weight of water enclosed between section 1 and 2; F_f is the total external force of friction and resistant action along the surface of contact between the water and the channel; b_1 and b_2 are the momentum coefficient at section 1 and 2; Y is the unit weight of water; Q is the discharge; V_1 and V_2 is the mean velocity of flow at section 1 and 2;

If the channel is short, horizontal and prismatic, F_f and are neglected and

$$\beta_1 = \beta_2 = 1$$

Now, the momentum equation can be simply written as:

$$\gamma \frac{Q}{g} (V_2 - V_1) = P_1 - P_2 \qquad (11.15)$$

the hydrostatic pressure forces P_1 and P_2 are respectively,

$$P_1 = \gamma A_1 \overline{z_1} \text{ and } P_2 = \gamma A_2 \overline{z_2} \qquad (11.16)$$

and

Where, $\overline{z_1}$ and $\overline{z_2}$ and are the distances to the centroid below the surface of the flow at section 1 and 2.

$$V_1 = \frac{Q_1}{A_1} \text{ and } V_2 = \frac{Q_2}{A_2}$$

Also,

Then the momentum equation reduces to

$$\frac{Q^2}{gA_1} + A_1\overline{z_1} = \frac{Q^2}{gA_2} + A_2\overline{z_2} \qquad (11.17)$$

The two sides of the above equation are analogous and hence may be reduced to a general form for a short channel section as;

$$F = \frac{Q^2}{gA} + A\overline{z} \qquad (11.18)$$

The first term of the above equation represent rate of change of momentum of the flow passing through the channel section per unit weight of water and the second term is the force per unit weight of water.

Since both terms are essentially force per unit weight of water, their sum is known as the specific force and equation (11.18) is known as specific force equation

11.6 Hydraulic Jump and its Application

When the depth of flow changes rapidly from a low stage to a high stage, it results in an abrupt rise of water surface. This local phenomenon is known as 'hydraulic jump'. It frequently occurs in a canal below a regulating sluice, at the toe of a spillway, at downstream of narrow channel or at the place where a steep channel slope suddenly turns flat. (For hydraulic jump see lesson-12)

Hydraulic jump has many practical applications. It is used,

1) To dissipate the high kinetic energy of water near the toe of the spillway and to protect the bed and banks of a river near a hydraulic structure.
2) To raise water level for irrigation and other distribution works.
3) To increase the discharge of a sluice by holding back tail water.
4) To mix of chemicals in water purification plant.
5) To remove air pockets from water supply lines and prevents air locks.
6) To aerate the stream polluted by bio-degradable materials.

Solved Examples:

Problem 11.1: For a discharge of 10 cumec through a rectangular channel of 5 m width what will be the specific energy of the flowing water if the depth of water in the channel is 3m.

Solution:

Width of the channel, b = 5 m; Discharge, Q = 10 cumec; Depth of water, h = 3 m.

$$\text{Specific energy,} \ = h + \frac{v^2}{2g},$$

Hence,

Where,

$$v = \frac{\text{Disharge } (Q)}{\text{Cross sectional area } (b \times h)}$$

$$E = 3 + \frac{\left(\frac{10}{5\times3}\right)^2}{2\times9.81} = 3.0226 \text{ m Ans.}$$

= 3.0226 m Ans.

Problem 11.2: What would be the critical depth of the water flowing through a rectangular channel of 4 m width, when discharge is 16 m³/s.

Solution:

$$\text{Discharge per unit width, } q = \frac{\text{Disharge}}{\text{Width of the chanel}} = \frac{16}{4} = 4 \text{ cumec/m}$$

From eq. (11.9) Critical depth is given by,

$$h_c = \left(\frac{q^2}{g}\right)^{1/3} = \left(\frac{4^2}{9.81}\right)^{1/3} = 1.177 \text{ m}$$

Ans.

Problem 11.3: A rectangular channel carries a discharge of 18 m³/s and the width of the channel is 6 m. Find the minimum specific energy of the flowing water.

Solution:

Minimum specific energy at critical depth. Hence, first we will calculate the value of critical depth.

Discharge per unit width, $\quad q = \dfrac{\text{Disharge}}{\text{Width of the chanel}} = \dfrac{18}{6} = 3 \quad$ cumec/m

From eq. (11.9) Critical depth is given by,

$$h_c = \left(\frac{q^2}{g}\right)^{1/3} = \left(\frac{3^2}{9.81}\right)^{1/3} = 0.97168 \, \text{m}$$

Now from eq. (11.13) The value of minimum specific energy,

$$E_{min} = \frac{3h_c}{2} = \frac{3 \times 0.97168}{2}$$

$$= 1.4575 \text{ m of water} \qquad \textbf{Ans.}$$

Lesson 12. Hydraulic Jump or Standing Wave

Hydraulic jump defined as, the rise of water level, which takes place due to the transformation of unstable shooting flow (super-critical) to the stable streaming flow (sub-critical flow).It frequently occurs in a canal below a regulating sluice, at the toe of a spillway, at downstream of narrow channel or at the place where a steep channel slope suddenly turns flat.

Hydraulic jump defined as, the rise of water level, which takes place due to the transformation of unstable shooting flow (super-critical) to the stable streaming flow (sub-critical flow).

It frequently occurs in a canal below a regulating sluice, at the toe of a spillway, at downstream of narrow channel or at the place where a steep channel slope suddenly turns flat.
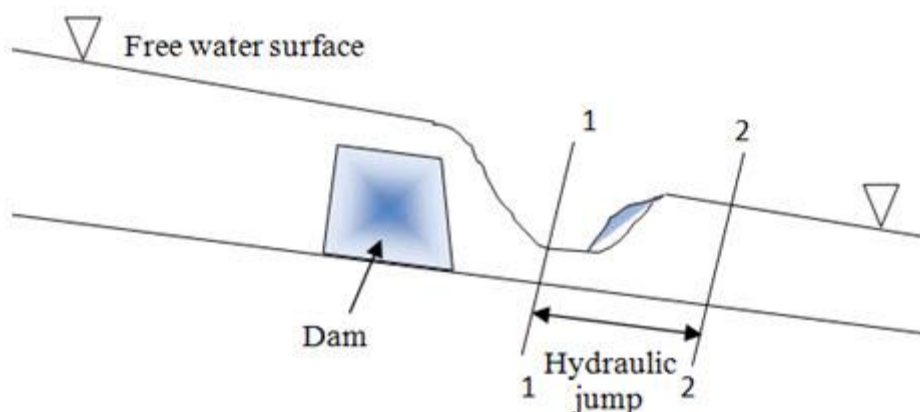

Fig. 12.1. Hydraulic Jump.

Consider the flow of water over a dam as shown in Fig. 12.1. The depth of water at the section 1-1 is small, but it increases towards downstream rapidly over a short length of the channel. This is because at the section 1-1, the flow is a shooting flow as the depth of water at section 1-1 is less than the critical depth. Shooting flow is an unstable type of flow and does not continue on the downstream side. Then this shooting will convert itself into a streaming or tranquil flow and hence

depth of water will increase. This sudden increased of depth of water is called a hydraulic jump or a standing wave.

When hydraulic jump take place, a loss of energy due to eddy formation and turbulence occurs.

12.1 Expression for Depth of Hydraulic Jump

Following assumption are made before deriving the expression for the depth of hydraulic jump,

1. Flow is uniform and pressure distribution is due to hydrostatic before and after the jump.

2. Losses due to friction on the surface of the bed of the channel are negligible

3. The slope of the bed of the channel is small, so that the component of the weight of the fluid in the direction of flow is negligibly small.

Consider a hydraulic jump formed in a channel of horizontal bed as shown in figure 12.2.

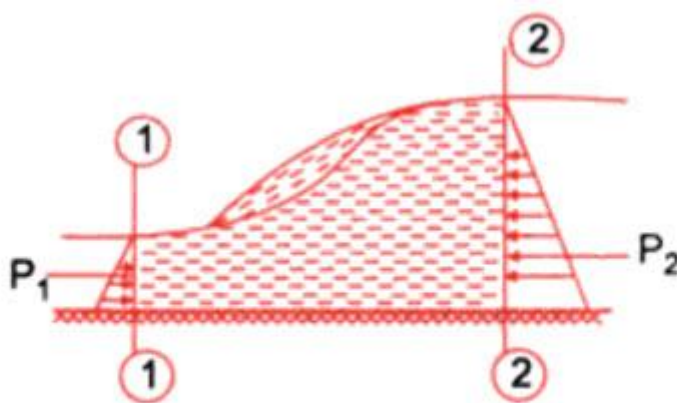Consider there are two sections 1-1 and 2-2 before and after hydraulic jump.

Fig.12.2. Hydraulic jump. (Source: Bansal, 2009)

Let d_1 is the depth of flow at section 1-1 i.e. initial depth; d_2 is the depth of flow at section 2-2 i.e. depth of flow after the hydraulic jump, also known as sequent depth or theoretical tail water depth. The depth pair at section 1-1 and 2-2 are called conjugate depth.

v_2 is the velocity of flow at section 1-1; v_2 is the velocity of flow at section 2-2; z_1 is the depth of centroid of area at section 1-1 below free surface; z_2 is the depth of centroid of area at section 2-2 below free surface; A_1 is the cross sectional area at section 1-1; A_2 is the cross sectional area at section 2-2.

Consider unit width of the channel.

The force acting on the mass of water between sections 1-1 and 2-2 are:

(i) Pressure force P_1 and P_2 on section 1-1 and 2-2 respectively.

(ii) Frictional force on the floor of the channel, which assumed to be negligible.

Let q = discharge per unit width

$$= v_1 d_1 = v_2 d_2 \qquad\qquad (12.1)$$

Now, pressure force on section 1-1

$$P_1 = \rho g A_1 \overline{Z_1} = \rho g \times (d_1 \times 1) \times (d_1/2)$$

(Since, we are considering unit width hence, $A_1 = d_1 \times 1$)

$$= \frac{\rho g \, d_1^2}{2}$$

Similarly pressure force on section 2-2

$$P_2 = \frac{\rho g \, d_2^2}{2}$$

Net force acting on the mass of water between section 1-1 and 2-2

$$= P_2 - P_1$$
$$= \frac{\rho g}{2} [d_2^2 - d_1^2] \qquad\qquad (12.2)$$

But from the momentum principle, the net force acting on a mass of fluid must be equal to the rate of change of momentum in the same section.

Rate of change of momentum in the direction of force

∴Rate of change of momentum in the direction of force

= mass of water per second × change of velocity in direction of force

Now, mass of water per second

= ρ × discharge per unit width × width

= $\rho \times q \times 1 = \rho q\, m^3/s$

Change of velocity in the direction of force $v_1 - v_2$

Hence, rate of change of momentum in the direction of force

$$= \rho q(v_1 - v_2) \qquad (12.3)$$

Hence, according to the momentum principle, the expression given by eq. (12.2) is equal to the expression given by eq. (12.3)

or
$$\frac{\rho g}{2}[d_2^2 - d_1^2] = \rho q(v_1 - v_2) \qquad (12.4)$$

But from equation (1),
$$v_1 = q/d_1 \text{ and } v_2 = q/d_2$$

Substituting the value of and in eq. (12.4) and by solving we get,

$$d_2 + d_1 = \frac{2q^2}{g d_1 d_2} \qquad (12.5)$$

Multiplying both sides by d_2 , we get

$$d_2^2 + d_1 d_2 = \frac{2q^2}{g d_1}$$

$$d_2^2 + d_1 d_2 - \frac{2q^2}{gd_1} = 0 \qquad\qquad (12.6)$$

By solving eq. (12.6) we get

$$d_2 = \frac{-d_1 \pm \sqrt{d_1^2 + \frac{8q^2}{gd_1}}}{2}$$

Neglecting the negative sign, we get

$$d_2 = -\frac{d_1}{2} + \sqrt{\frac{d_1^2}{4} + \frac{2q^2}{gd_1}}$$

$$= -\frac{d_1}{2} + \sqrt{\frac{d_1^2}{4} + \frac{2(v_1 d_1)^2}{gd_1}}$$

$$= -\frac{d_1}{2} + \frac{d_1}{2}\sqrt{1 + \frac{8v_1^2}{gd_1}} \qquad\qquad (12.7)$$

Now, depth of hydraulic jump = $(d_2 - d_1)$

$$F_{r1} = \frac{v_1}{\sqrt{gd_1}}$$

Froude number on the upstream side is given by

Substituting the value in equation (12.7) we get,

$$d_2 = \frac{d_1}{2}\left(-1 + \sqrt{1 + 8F_{r1}^2}\right) \qquad\qquad (12.8)$$

12.2 Energy Dissipation due to Hydraulic Jump

When hydraulic jump take place, a loss of energy due to eddies formation and turbulence occurs. This loss of energy is equal to the difference of specific energies at sec. 1-1 and 2-2.

Let E_1, and E_2 are the energy at section 1-1 and 2-2 respectively. Loss of energy due to hydraulic jump

$$\Delta E = E_1 - E_2$$

$$= \left(d_1 + \frac{v_1^2}{2g}\right) - \left(d_2 + \frac{v_2^2}{2g}\right)$$

$$= \left(\frac{v_1^2}{2g} - \frac{v_2^2}{2g}\right) - (d_2 - d_1)$$

Since $v_1 = q/d_1$ and $v_2 = q/d_2$

$$= \left(\frac{q^2}{2gd_1^2} - \frac{q^2}{2gd_2^2}\right) - (d_2 - d_1)$$

$$= \frac{q^2}{2g}\left(\frac{d_2^2 - d_1^2}{d_1^2 d_2^2}\right) - (d_2 - d_1) \qquad (12.9)$$

$$q^2 = gd_1 d_2 \left(\frac{d2 + d1}{2}\right)$$

But from eq. (5),

Substituting the value of in eq. (12.9) and solving the expression, we get

$$\Delta E = \frac{(d_2 - d_1)^3}{4d_1 d_2} \qquad (12.10)$$

12.3 Jump Efficiency

The ratio of specific energy after the jump (E$_2$) to the energy before the jump (E$_1$) is defined as the efficiency of the jump,

i.e.,   Efficiency of the jump $= \dfrac{E_2}{E_1} = \dfrac{d_2 + \dfrac{v_2^2}{2g}}{d_1 + \dfrac{v_1^2}{2g}}$

$$= \dfrac{\dfrac{d_2}{d_1} + \dfrac{v_2^2}{2gd_1}}{1 + \dfrac{v_1^2}{2gd_1}}$$

Replacing $\left(\dfrac{d_2}{d_1}\right)$ by eq. (12.8) and v_2 by $v_1\left(\dfrac{d_1}{d_2}\right)$ and simplifying further, $\left(\dfrac{E_2}{E_1}\right)$ is expressed as,

$$\dfrac{E_2}{E_1} = \dfrac{\left(\sqrt{1+8F_{r1}^2}\right)^3 - 4F_{r1}^2 + 1}{8F_{r1}^2\left(2 + F_{r1}^2\right)} \tag{12.11}$$

Eq. (12.11) indicates that the jump efficiency is a dimensionless function of Froude number f_{r1} before the jump.

12.4 Relative Loss of Energy

Relative loss of energy is the ratio of loss of energy due to hydraulic jump to the specific energy prior to hydraulic jump, it can be written as,

$$\dfrac{\Delta E}{E_1} = \dfrac{E_1 - E_2}{E_1}$$

$$= \dfrac{\dfrac{(d_1 - d_1)^3}{4d_1 d_2}}{d_1 + \dfrac{v_1^2}{2g}}$$

$$= \dfrac{\left(\dfrac{d_2}{d_1} - 1\right)^3 \left(\dfrac{d_1^3}{4d_1 d_2}\right)}{\dfrac{d_1}{2}\left(2 + \dfrac{v_1^2}{2gd_1}\right)}$$

$$= \frac{\left(\frac{d_2}{d_1} - 1\right)^3 \left(\frac{d_1^2}{4d_2}\right)}{\frac{d_1}{2}(2 + F_{r1}^2)}$$

Since, $F_{r1} = \frac{v_1}{\sqrt{gd_1}}$

$$= \frac{\left\{\frac{1}{2}\left(-1 + \sqrt{1 + 8F_{r1}^2}\right) - 1\right\}^3}{\frac{2d_2}{d_1}(2 + F_{r1}^2)}$$

Since, $d_2 = \frac{d_1}{2}\left(-1 + \sqrt{1 + 8F_{r1}^2}\right)$

$$\models \frac{1}{8} \frac{\left(-3 + \sqrt{1 + 8F_{r1}^2}\right)^3}{\left(-1 + \sqrt{1 + 8F_{r1}^2}\right)(2 + F_{r1}^2)} \qquad (12.12)$$

Equation (12) gives the relative energy loss equation in hydraulic jump

12.5 Types of Hydraulic Jump

Several distinct types of hydraulic jump in a horizontal floor are classified, mainly on the basis of Froude number and shape of the water surface formed in the jump zone. According to the study of USBR (United States Bureau of Reclamation), these types are called;

(i) Undular jump: It occurs in the range of from 1 to 1.7. Water surface shows undulations (figure 12.3a). Here efficiency is very small, is also small.

(ii) Weak jump: A series of small rollers develop on the surface of the jump, but downstream water surface remains smooth (figure 12.3b). Velocity throughout is fairly uniform and energy loss is low. Efficiency varies from 15 to 18%. varies from 1.7 to 2.5. Therefore, this jump is called weak jump.

(iii) Oscillating jump: There is an oscillating jet entering the jump bottom to surface and back again with no periodicity. Each oscillation produces a large wave of irregular period which, very commonly in canals, can travel for kilometres doing unlimited damage to earthen banks and ripraps (figure12.3c). varies from 2.5 to 4.5. Here efficiency is about 45%.

(iv) Steady jump: Here varies from 4.5 to 9. Efficiency or energy dissipation varies from 45% to 70%. The jump is well balanced and performance is at the best (figure 12.3d).

(v) Strong jump: It occurs when > 9. It is rough jump, but effective since energy dissipation may reach from 70% to 85% (figure 12.3e).
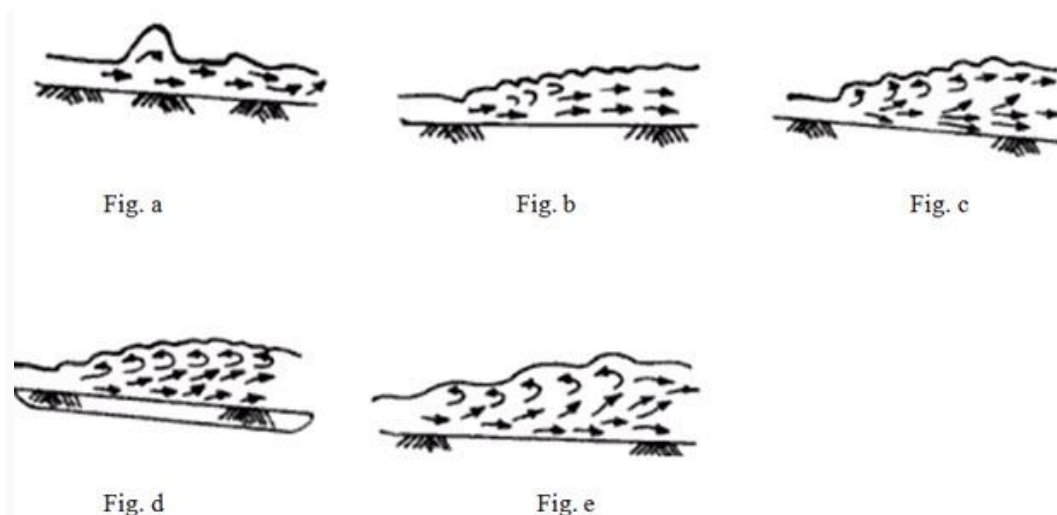
Fig. a   Fig. b   Fig. c

Fig. d   Fig. e

Fig. 5. Types of hydraulic jump. (Source: Das, 2008)

Solved   Examples:
Problem 12.1: Find the depth of flow of water after the hydraulic jump in a rectangular channel of 4 m width having a discharge of 16 m^3/s. The depth of water in the channel before hydraulic jump was 0.5 m.

Solution:

Width of channel, b = 4 m; Depth of flow before jump, d_1 = 0.5 m; Discharge, Q = 16 m^3/s.

Let the depth of flow after hydraulic jump is d_2, hence from eq. (12.8)

$$d_2 = \frac{d_1}{2}\left(-1 + \sqrt{1 + 8F_{r1}^2}\right)$$

$$= \frac{d_1}{2}\left(-1 + \sqrt{1 + 8\left(\frac{v}{\sqrt{gd1}}\right)^2}\right)$$

$$= \frac{d_1}{2}\left(-1 + \sqrt{1 + 8\left(\frac{Q}{A\sqrt{gd1}}\right)^2}\right)$$

$$= \frac{d_1}{2}\left(-1 + \sqrt{1 + 8\left(\frac{Q}{b \times d1\sqrt{gd1}}\right)^2}\right)$$

Now, substituting the values of Q, b, d_1 and g we get,

$d_2 = 2.316$m   Ans.

Problem 12.2. Determine the energy loss after the hydraulic jump for the previous problem.

Solution:

From eq. (12.10) the energy loss due to hydraulic jump is given by,

$$\Delta E = \frac{(d_2 - d_1)^3}{4 d_1 d_2}$$

$$= \frac{(2.316 - 0.5)^3}{4 \times 2.316 \times 0.5}$$

$= 1.29$ m**Ans.**

Module 3. Design of Permanent Gully Control Structures

Lesson 13. Design Requirements of Gully Control Structures

A structure installed across an active gully to stabilize the gully through control of erosion of gully bottom and banks is called gully control structure.

13.1 Design Requirements of Gully Control Structure

The gully control structures primarily designed for safe disposal of excess runoff generated from the watershed. While designing gully control structures three major points are considered, namely, (i) structure must have sufficient provision for safe discharge, (ii) the structure should have sufficient strength to withstand the pressure exerted by flowing water and (iii) the structure should be protected from erosion due to the flow passing over it. These points refer to hydrologic design, structural design and hydraulic design of structures. However, the design vigor depends on the type of gully control structures such as permanent structures require more vigorous design than temporary structures.

13.2 Functions or Purpose

The primary function of gully control structures is to provide safe passage to the flow through intervening into the prevailing slope of natural channel. The other purpose or function of gully control structures are listed below.

1. To reduce runoff erosive forces by stacking off the gradient of a gully and by controlling the course of flow to minimize ill effect on the banks.

2. To store water in the upstream as channel storage for irrigation.

3. To block sediment to keep it from damaging the downstream environment.

4. To maintain the stability of soil when vegetative cover is being established.

13.3 Causes of Failures

The structure failure caused mainly due to faulty hydrologic, hydraulic or structural design either alone or combination of these. However, there are cases when structures failed because of

· Insufficient capacity of structure

· Insufficient provision for dissipation of kinetic energy within the confine of structure

· Unprotected banks near to upstream of structures,

· Improper foundation causing uplift pressure to prevail over the body of the structure.

13.4 Types of Structures

Gully control structures can be grouped into two categories, namely, temporary gully control structures and permanent gully control structures. Temporary gully control structures are made up of locally available material and are designed for 3-10 years. Most of the check dams come under the category of temporary gully control structures. On the other hand, permanent gully control structures are designed for 25 to 50 years period.

13.4.1 Temporary Structures

All temporary structures should be practiced in G-1 type gullies. The purpose of this kind of dam is to temporarily maintain the stability of a gully and to make possible the establishment of vegetative cover. These are not durable and need frequent maintenance, though these are inexpensive and easy to build. These structures are constructed using the locally available material. The life of temporary structure is limited to 3-8 years.

Design Criteria of Temporary Structures

1. The overall height of a temporary check should not ordinarily be more than 75 cm. An effective height of about 30 cm is usually considered sufficient. Minimum 15 cm freeboard is necessary.

2. Life of the check dams under ordinary conditions should be 3-8 years and hence should be design for rainfall of 10 years return period.

3. Since the purpose of check dams in gully control is to eliminate grade in the channel, check dams theoretically should be spaced in such a way that the crest elevation of one will be same as the bottom elevation of the adjacent dam up-stream. Hence horizontal interval depends on the channel slope and height of check dam. Check dams of lesser height in higher slope will require more frequent check dams down the stream.

4. Suitable apron should be provided to avoid the souring due to the flow passing over these check dams. For this purpose, rip-rap is provided in the length of 1 to 1.5 meter downstream of the check dams. The gap between the rip-rap can be seeded with grass.

13.4.1.1 Woven Wire Dams

These dams are used in gullies of moderate slope and small drainage area. They help in the establishment of vegetation for permanent control of erosion. To construct a woven-wire dam a row of posts is set along the curve of the proposed dam at about 1.2 m intervals and 60-90 cm deep. Heavy gauge woven wire is placed against the post with the lower part set in a trench (15-20 cm deep) so that

25-30 cm projects above the ground surface along the spillway interval. Rock, brush or sod may be placed for approximately up to a length of 1.2 m to form the apron. For sealing the structure, straw, fine brush or similar material should be placed against the wire on the upstream side to the height of spillway crest. Fig.13.1 shows woven wire dam.
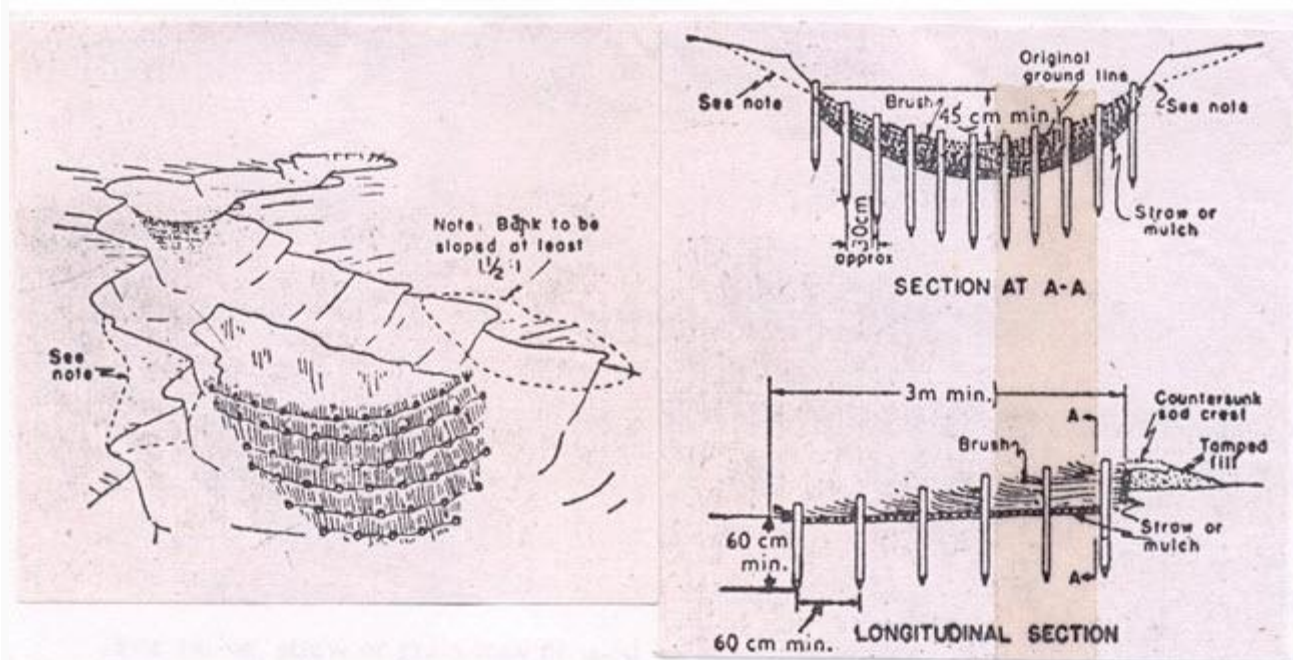

Fig. 13.1. Woven wire dam. (Source: Michael and Ojha, 1978)

13.4.1.2 Brush Dams

They are cheap and easy to build, but least stable of all types of check dams. They are best suited for gullies with small drainage area. The center of the dam is kept lower than the ends to allow water to flow over the dam rather than around it. For a distance of 3-4.5 m along the site of structure, sides and bottom of the gully are covered with thin layer of straw or similar fine mulch. Brushes are then packed closely together over the mulch to about one half of the proposed height of dam. Several rows of stakes are then driven crosswise in the gully, with rows 60 cm apart, and stakes 30-60 cm apart in the rows. Heavy galvanized wire is used to fasten the stakes in a row, as well as to firmly compress the brushes in places. Sometimes large stones are also placed on top of brush to keep it compressed and in close contact with the bottom of the gully. The major weakness is the difficulty of preventing the leaks and constant attention is required to plug openings of appropriate size with straw as they develop. Fig.13. 2 shows brush dam.
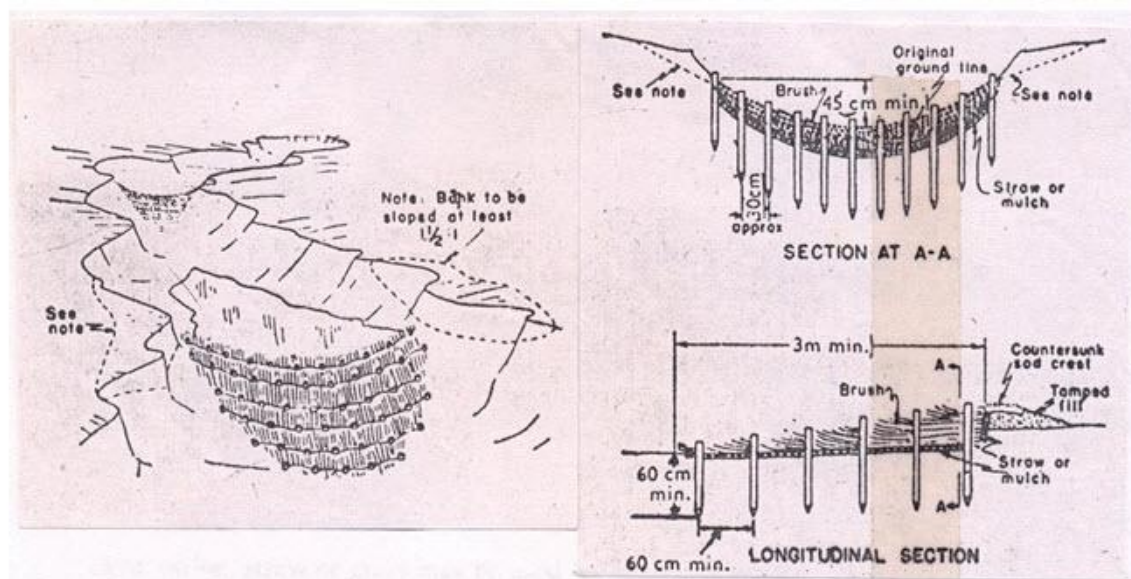
Fig. 13.2. Brush dam. (Source: Michael and Ojha, 1978)

13.4.1.3 Loose Rock Dam

Loose rock dams are suitable for gullies have small to medium size drainage area. They are used in areas where stones or rocks of appreciable size and suitable quality are available. Flat stones are the best choice for dam making as they can be laid in such a way that the entire structure is keyed together. If round or irregular shed stones are used, structure is generally encased in woven-wire so as to prevent outside stones from being washed away. If the rocks are small, they should be enclosed in a cage of woven-wire. To construct the dam, a trench is first made across the gully to a depth of about 30 cm. This forms the base of the dam on which the stones are laid in rows and are brought to the required height. The center of the dam is kept lower than the sides to form spillway. To serve as an apron, several large flat rocks may be countersunk below the spillway, extending about 1 m down-stream from the base of the dam. Fig.13.3 shows loose rock dam.
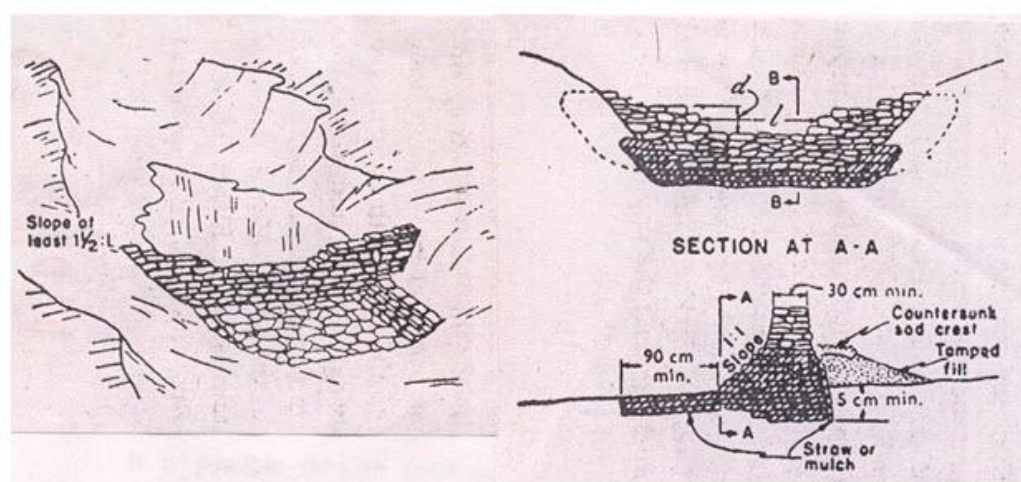

Fig. 13.3. Loose rock dam. (Source: Michael and Ojha, 1978)

13.5 Permanent Structures

Permanent structures are constructed when the benefits from such structures are justifiable compared to the cost of construction. General requirements of the permanent structures for gully control are: (1) they should be constructed with permanent materials, (2) they should have adequate capacity to handle the runoff, and (3) they should help in stabilizing the gully and store water wherever necessary. For the purpose of gully control, following types of structures are adopted.

1)   **Drop structures**: The drop structures are recommended for G-2 type of gully where the downstream fall of water is limited to 3.0 meter. The drop structures are used along the gully bed to act as control points so that the gully bed is not eroded below the crest level of the structure. Fig. 4 shows drop spillway.
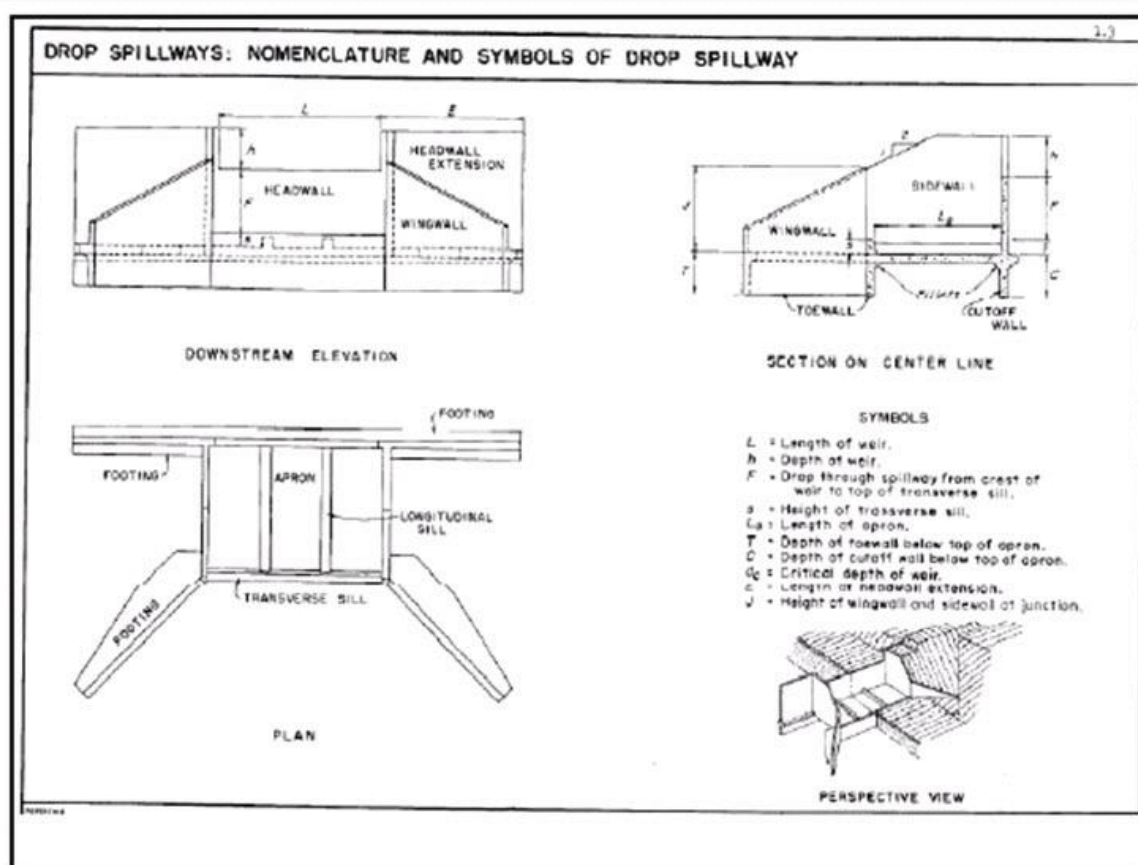

Fig. 13.4. Drop structure. (Source: Singh et al. 1978)

2) Chute structure: It is permanent type gully control structures in which the excess runoff are passed through chute spillway. This type of structure is recommended for G-3 type of gully where the gully depth is more than 3 meter. Fig. 5 shows chute spillway.
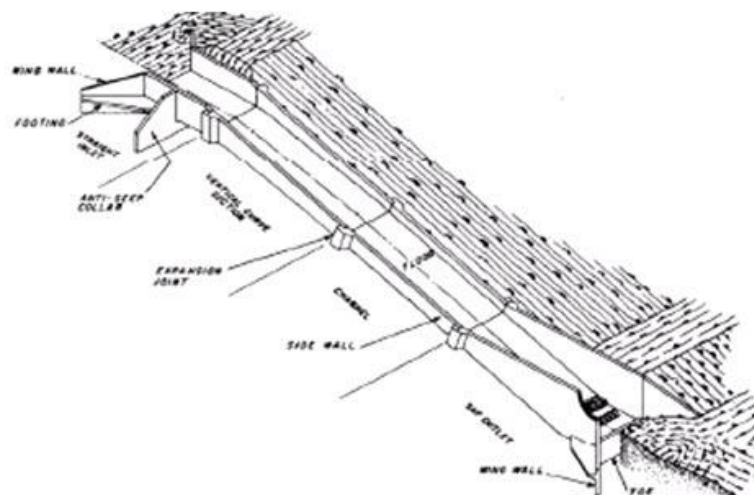
Fig. 13.5. Chute structures. (Source: SCS, 1984)

3) **Drop inlet structures:** These structures should be practiced in G-4 type of gullies where the depth is more than 9 meters and steep side slope exist. In this conditions the drop structures and chute structures are difficult to construct and practically not feasible. In this type of structures the runoff is guided through under lying pipe towards downstream. Fig. 6 shows drop inlet structure.

Fig. 13.6. Drop inlet spillway. (Source: SCS, 1984)

13.6 Planning for Gully Control

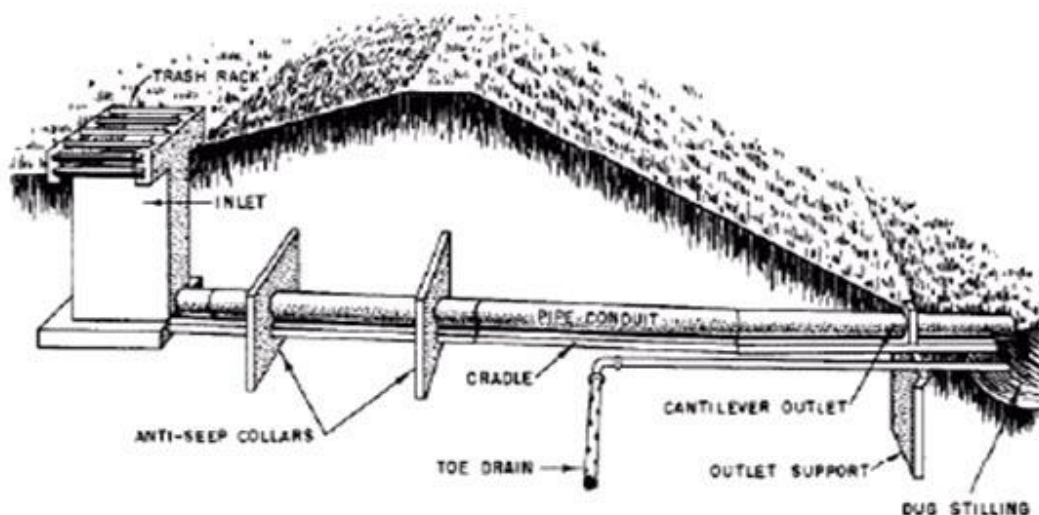Control of gully erosion is done both by taking appropriate measures in the gully beds as well as in the catchment area. The first step in planning the gully control programme is to plan to control the runoff from the catchment area. This may be done by using good land and crop management practices, such as contouring, strip cropping and terracing. Gully control measures should be considered when the plan for the entire watershed is prepared.

Control of gullies may be an extensive operation and the cost of the gully control must be balanced by the benefits. Benefits include the protection of the adjoining areas, reduction of sediment load to the river system, storage of water and sometimes reclamation of the gully beds for cultivation purposes.

Lesson 14. Design Procedure for Spillways

The gully control structures are essentially constructed across the well-defined natural channels. The temporary structures do not need meticulous and exhaustive design and is mostly constructed at G1 type gully. The height of the structure is generally less than 1m. These structures are low cost and made up of locally available material. The permanent structures are constructed at the places where temporary structures are not feasible. These structures are designed to carry heavy discharge and thus require exhaustive design. These structures are cost intensive and economy also figures as a major factor while designing these structures.

14.1 Components of Gully Control Structures

The gully control structures has three major components namely, inlet that allows the flow to enter the structure, embankment or main body that conveys the flow and outlet that safely dispose the flow.

14.1.1 Inlet

It is the part of structure through which water enters the structure. It may be either in the form of box or weir. The primary purpose of the inlet is to maintain the flow in steady state conditions so that its impact on the main structure could be minimized.

14.1.2 Main Body

It is that part, which, receives the flow from the inlet and leaves the same to the outlet through a conduit. The conduits can be either closed form (box type)or open form (rectangular channel).

14.1.3 Outlet

It is downstream component of the structure, which discharges the flow into the gully with safe velocity. It dissipates the kinetic energy of falling discharge within the structure to prevent the downstream channel as well as structure from erosion. Different types of inlet, conduit and outlets are shown in Fig. 14.1.
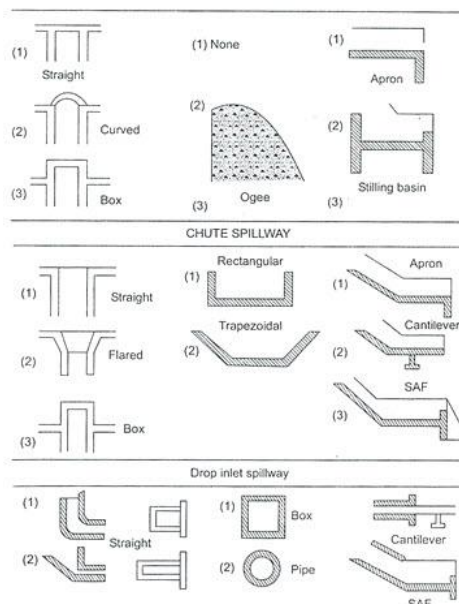
Fig. 14.1. Types of inlet, conduit and outlets. (Source: Suresh, 2009)

The design feature of gully control structure requires a firm foundation. The foundation should be sufficient to hold the weight of the structures and strong enough the save the structure from overturning and buckling. Since these type of structures has to be remained in moist soil, adequate drainage system are required to avoid the phenomena of piping and uplift pressure that may cause structure unstable.

## 14.2 Selection Criteria of Gully Control Structures

The different permanent gully control structures are similar in many respect expect the provision for conduit of the main body. These are divided into drop spillway, chute spillway and drop inlet spillway. The major criteria remain the height of fall for selecting the permanent gully control structures e.g. for less than 3 m fall, drop spillways are considered, for 3-6m fall, chute spillways and for more than 6m fall, drop in-let spillways are considered. For less than 1 m fall, temporary structures are recommended. Fig. 14.2 shows general guideline for selection of suitable structure (SCS, 1984).

14.3 Design Procedure (For Permanent Gully Control Structures)

The first step in the design procedure is being the on-site investigation and site selection. Site is finalized after conducting various surveys and investigations. These include topographic survey, sedimentation, land use, soil characteristics, socio-economic conditions and geologic investigation.

The channel profile survey using topographic maps is conducted for locating the suitable site after considering the native slope and uniform cross-section of the channel. This survey also indicates about the submergence area and storage volume upstream of the proposed structure. The sediment survey includes estimation of expected sedimentation rate. Land use survey includes worth of structure from the benefits (tangible and intangible) thus obtained. Socio-

economic conditions refer to relative impact on the people in the vicinity of the proposed structures. The geologic investigation determines the suitability of the site for foundation and thus stability of the structure. The sites are selected after optimizing for all these conditions.
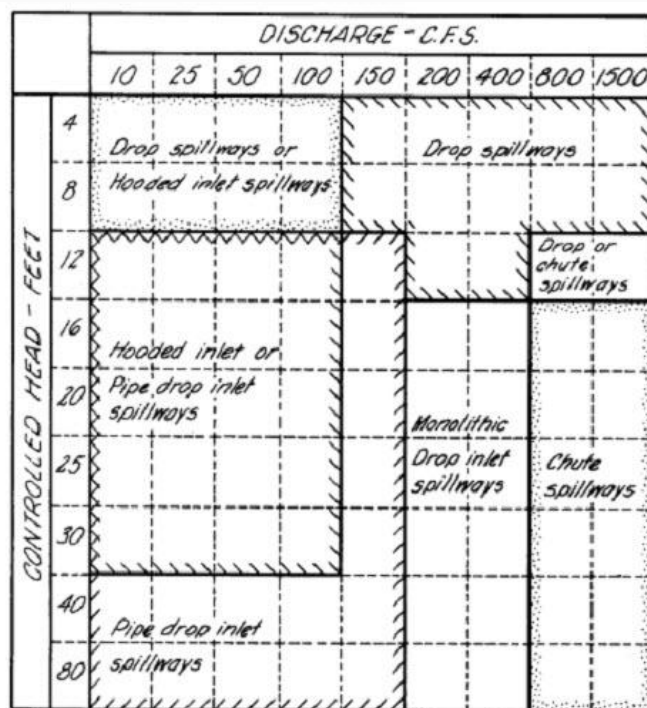
Fig. 14.2. General guide for suitable structure selection.

(Source: SCS, 1984)

After site is selected, engineering design of the structures is carried out. This includes, hydrologic design that describes design inflow rate and amount, hydraulic design that includes determination of suitable dimension of various components of the structure and structural design that includes determination of specification and required strength of construction material considering desired safety.

14.3.1 Hydrologic Design

The hydrologic design includes estimation of peak flow that the structure is required to handle. This is essential in cased of drop and chute spillway, whereas the runoff volume information is required in case of drop inlet spillway. The design includes the probability analysis of rainfall for determining frequency or return period. The design return periods for different structures are presented in Table 14.1.

Table 14.1. Design frequency for various types of structures

Type of structure	Frequency (years)
Storage and diversion dams having permanent spillways	50-100
Earth fill dam-storage having natural spillways	25-50
Stock water dam	25
Small permanent masonry gully control structure	10-15
Terrace outlet and vegetated waterways	10
Field diversion	15

Based on Table 14.1 the frequency for the particular structures isselected. Using the intensity-duration- frequency relationships (described in lesson 5)rainfall intensity is determined for selected location and peak runoff rate is estimated using the rational method.

14.3.2 Hydraulic Design

Hydraulic design consists of determining the dimensions of different components of the structure, on the basis of expected maximum runoff rate, that has been estimated in case of hydrologic design phase. The dimension of the structure should be sufficient to carry the design flow. Hydraulic design also deals with the study of the effect of flow on upstream and downstream reaches of the channel and dissipation of the kinetic energy of discharge falling towards downstream face of the structure. This phenomena helps in controlling the channel erosion below the structure. In this design standard principles of hydraulic and fluid mechanics are used which have been described in lessons 7 through 12. The hydraulic design example will be elaborated in lesson 16 for drop spillway.

14.3.3 Structural Design

Structural design involves the determination of strength and stability of different parts of the structure. It involves the analysis of various forces acting on the structure. They are mainly the water pressure which may be static or dynamic acting on the structure, forces developed due to overflow over the structure and the effect of water flow underneath the structure that is seepage and subsurface flow. The structure must be stable against these forces. In addition, the dimension of the structure should also be such that, the internal stress developed in the structure, must be resisted by the construction materials. The main objective for

structural design is to determine the dimension of different components of the structures and material specification to withstand the various possible forces namely hydrostatic water pressure, self-weight of structure and up-lift pressure due to wet foundation. The design example is given in lesson 17 for drop spillway.

Lesson 15. Drop Spillway

The drop spillway is a weir structure. Flow passes through the weir opening, drops to an approximately level apron or stilling basin and then passes into the downstream channel. Drop spillway is one of the most commonly used gully control structures. It is mainly used at the gully bed to create a control point. Several such drop structures are constructed across the gully width throughout the length at fixed intervals. The series of such structures, develop a continuous break to flow of water, causing deposition of sediments and thus to filling the gully section. Sometimes, the drop structures are also used at the gully head to pass the flow safely and controlling the gully head. The different components of drop structures are shown Fig. 15.1.
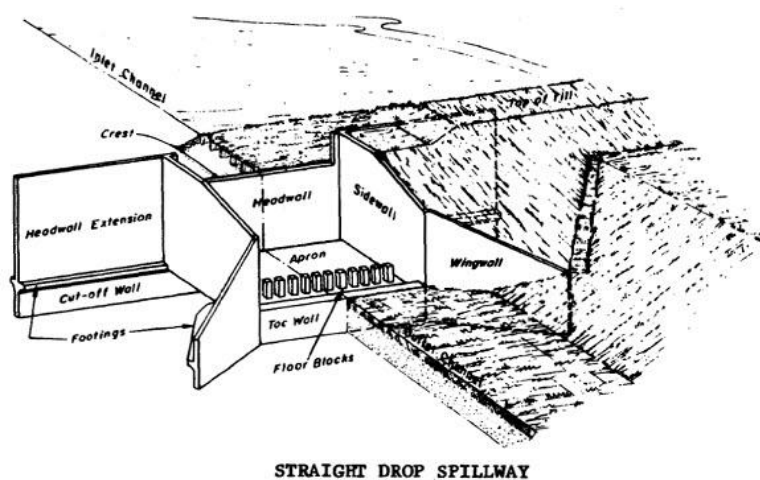

STRAIGHT DROP SPILLWAY

Fig. 15.1. Various components of drop spillway. (Source: SCS, 1984)

15.1 Different Components of Drop Structures

1. Head wall

2. Head wall extension

3. Side walls

4. Wing walls

5. Apron

6. Longitudinal sills

7. End sills

8. Cut-off walls

9. Toe wall

1. Head wall: It acts as a front wall against runoff in the drop spillway. It is constructed across the gully width. Headwall is provided with weir for passing through the flow.. Rectangular weir is most commonly used. The size of the weir should be sufficient to pass the design discharge safely.

2. Head wall extension: It is the extended portion of head wall into the gully sides. Its main function is to provide structural strength against sliding of the structure.

3. Sidewalls: These are constructed in the side along the gully walls. These two walls determine the apron section. The function of sidewalls is to prevent splashing of water over the gully banks and to confine the water flow within the apron.

4. Wing walls: These are constructed at the rear end of the structure, with some inclination, usually at 45° from the vertical. These walls are extended up to the gully sides and perform the function of preventing the flow backward into the space left between gully wall and side wall of the structure.

5. Apron: It is one of the main downstream components of the straight drop spillway. It receives the gully flow with high velocity and dissipates the kinetic energy of the flow to protect downstream channel from erosion.. It includes several blocks, which are elevated by some height and make the apron surface rough. This feature of apron is responsible for dissipating the maximum kinetic energy of falling water by creating hydraulic jump, as a result the velocity of outgoing water gets significantly reduced.

6. Longitudinal Sills: These are constructed in the apron section. They are constructed lengthwise parallel to the side walls. The sills are useful to make the apron, stable. Usually the sill height is 5-10 cm.

7. End Sills: End sills are the elevated portion of rear end of the apron. This is usually 10-30 cm height and make 45° angle to the apron surface. Its main function is to obstruct the water, going directly in to the channel, below.

8. Cut-off Walls: These are constructed to provide structural strength against sliding of the structure. They increase frictional resistance of the structure, which opposes the force causing to slide. In other words, cut-off walls acts as a key for the structure.

9. Toe Walls: These low walls are constructed at the bottom of the structure to prevent slippage or spreading of soil.

The inflow capacity of the straight drop structure is controlled by the size of the inlet i.e. notch, used. The notch is in form of a rectangular weir, in which flow is directly proportional to the length of the weir. The drop structure is used to control the velocity of runoff in a channel by lowering the water abruptly from one level to another.

15.2 Uses of Drop Structures

The various functional uses of this spillway are given as under:

1. Used for grade stabilization in lower reaches of waterways and outlets.

2. Used as an outlet structure in tile drainage system

3. Used for controlling the tail water at the outlet section of the conduit or spillway.

4. Used in the water distribution system for controlling irrigation.

5. Used as an outlet for disposing surface water from large areas, especially where drainage ditches exists.

Adaptability: The straight drop spillway is an efficient structure for controlling relatively low heads, normally up to 3m.

15.3 Material for Construction

Drop structures can be constructed with reinforced concrete, plain concrete, rock masonry, concrete blocks (with or without reinforcement)or sheet piling of steel, timber, and prefabricated metals.

Construction Requirement

1. Reinforced concrete is used in construction. The steel required for reinforcement is about 1-1.5% (based on experience of hilly watersheds) of the body weight of the structures.

2. The basin floor should be leveled transversely and longitudinally.

3. Upstream and or downstream channel transitions may be needed.

4. Concrete floor and wall thickness is usually 30-45 cm depending upon the flow velocity of the runoff and approach length available for inlet box. If approach length is more than 15 m, 30 cm thick wall can be recommended.

5. The depth of the concrete footing should not be less than 50 cm in any case for small- and medium- sized natural channel.

6. May need riprap or other form of erosion protection upstream and downstream of the drop structure where earthen channel exists.

7. The approach channel bed elevation should be same as the spillway crest elevation at the headwall.

15.4 Advantages

1. In this structure, the danger of undermining by rodents is not possible.

2. Straight drop spillways are less susceptible to get structural damaged, than the other structure.

3. As in other spillways (especially in drop-inlet) the conduit is likely to be clogged by debris, but in the drop spillway there is no such problem.

4. Its construction is very easy.

15.5 Disadvantages

There are some disadvantages of the drop spillways.

1. In the areas where discharge is less than 3 m^3/s and total water head or drop exceeds 3 m, the construction of straight drop spillway proves to be costly affair, should not be preferred.

2. If the gully grade below the structure is not stable then it is impossible to construct a drop spillway.

3. The place where temporary storage is required to obtain a reduction in discharge, the construction of this structure is not technically justified.

Lesson 16. Drop Spillway Design

16.1 Introduction

In the previous lecture, design considerations of permanent gully control structures are discussed. These include hydrologic design, hydraulic design and structural design. In additions, details about drop spillway are also discussed. In this lecture our focus would be on hydrologic and hydraulic design of drop spillway. The hydraulic structures are failed mainly due to their insufficient hydraulic capacity and lack of provision for dissipating the energy of falling water into the structure. That is why, design of these structures is done considering not only to have sufficient discharge handling capacity, but also for dissipating the kinetic energy of falling discharge within the structure in such a manner and to limit that will protect the structure and downstream channel from the erosion.

16.2 Principle of Hydrologic Design

It involves the estimation of design runoff rate and flood volume, which the structures is expected handle safely. This is done by computing the peak runoff rate for 25 to 30 years return period. Runoff rate is calculated using the Rational method:

$$Q_{peak} = \frac{1}{36} CIA \qquad (16.1)$$

Where, Q_{peak} is the peak runoff rate, m^3/s; C is the runoff coefficient; I is the rainfall intensity (cm/h) for the duration equal to time of concentration of watershed and for a given recurrence interval; A is the watershed area, ha.

Hydraulic design consists of determining the dimension of different components of the structure, on the basis of expected maximum runoff rate determined using the Rational method. The dimension of the structure should have sufficient capacity and also provision for dissipating the kinetic energy of discharge falling towards downstream face of the structure. This phenomenon helps in controlling the channel erosion below the structure.

16.2.1 Design of Drop Spillway

The hydraulic design of drop structures includes three major steps namely, inlet design, conduit (mostly rectangular weir) and outlet design. The different views of drop spillway, e.g., plan, side view and downstream elevation are presented in Fig. 16.1.
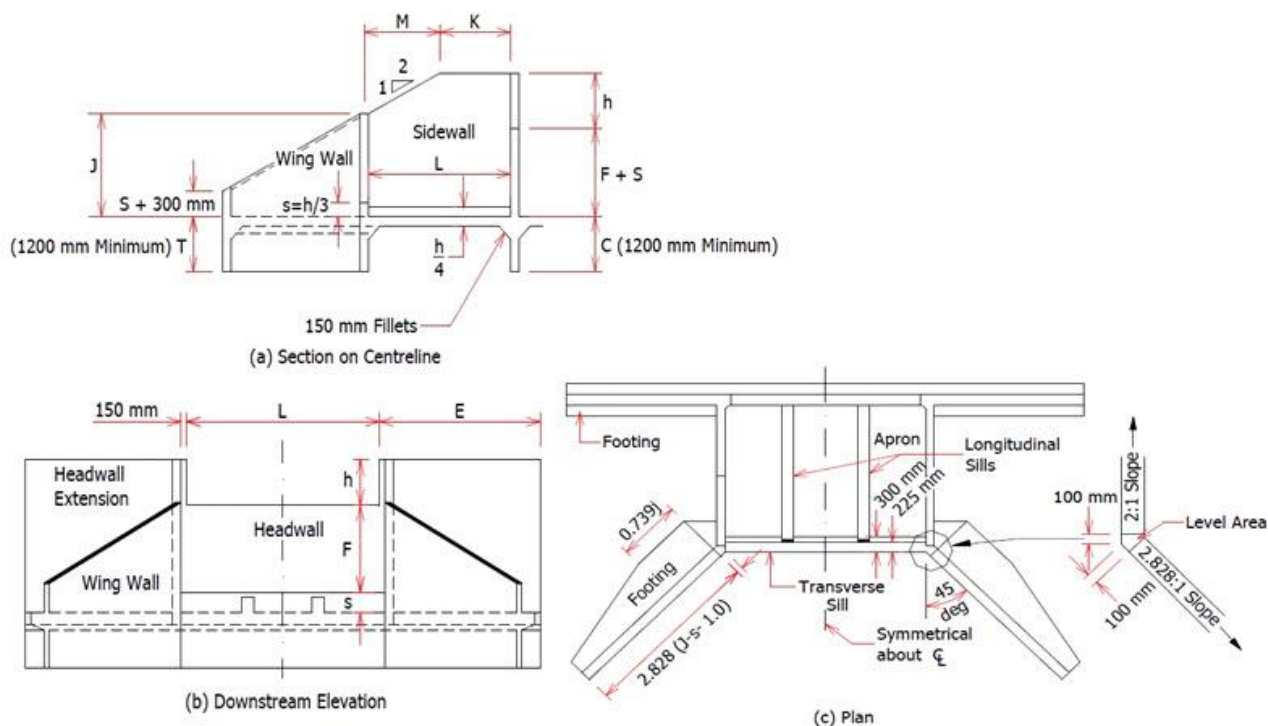
Fig. 16.1. Different view of drop spillway. (Source: Bansal, 2010)

16.2.2 Computation of Length and Depth of Weir

Straight drop spillway consists of straight type inlet that is why its name became as straight drop spillway. This type of inlet is most suitable for wide and shallow gullies to handle small to medium flows.

To calculate the inflow capacity of straight drop spillway, the following weir formula may be used:

$$Q = \frac{1.711 L h^{3/2}}{(1.1+0.01 F)} \qquad (16.2)$$

Where, Q is the maximum discharge capacity of the weir (cumec); L is the length of weir (m); h is the total depth of weir (m); and F is the net drop from top of transverse sill to crest (m).

The site condition best describes the values of L and h. The values of L,h and F can be taken from standard table which are given in Table 1. The ratio between depth of flow (h) and net drop (F) influences the scouring. Higher ratio increases the scouring tendency and lower values of this ratio will result into lower system

performance. So for most economical design, this ratio is recommended to be lower than 0.5 and should not be more than 0.75 in any case.

Table 16.1. Discharge capacities for straight drop spillways with net drop, F = 2 m. multiply by correction factor for other drops (discharge capacity Q, cumec)

h, m	Length of weir (L, m)										
	1	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
0.4	0.39	0.5	0.77	0.97	1.16	1.35	1.55	1.74	1.93	2.13	2.32
0.5	0.54	0.81	1.08	1.35	1.62	1.89	2.16	2.43	2.7	2.97	3.24
0.6	0.71	1.07	1.42	1.78	2.13	2.49	2.84	3.2	3.55	3.91	4.26
0.7	0.89	1.34	1.79	2.24	2.68	3.13	3.58	4.03	4.47	4.92	5.37
0.8	1.09	1.64	2.19	2.73	3.28	3.83	4.37	4.92	5.47	6.01	6.56
0.9	1.3	1.96	2.61	3.26	3.91	4.57	5.22	5.87	6.52	7.17	7.83
1	1.53	2.29	3.06	3.82	4.58	5.35	6.11	6.87	7.64	8.4	9.17
1.1	1.76	2.64	3.52	4.41	5.29	6.17	7.05	7.93	8.81	9.69	10.57
1.2	2.01	3.01	4.02	5.02	6.02	7.03	8.03	9.04	10.04	11.05	12.0

										5
1.3	2.26	3.4	4.53	5.66	6.79	7.93	9.06	10.19	11.32	12.45	13.59
1.4	2.53	3.8	5.06	6.33	7.59	8.86	10.12	11.39	12.65	13.92	15.18
1.5	2.81	4.21	5.71	7.02	8.42	9.82	11.23	12.63	14.03	15.44	16.84

Correction factor for the other drops from 0.5 m to 3.0 m

Net Drop F (m)	0.5	0.75	1.0	1.25	1.50	1.75	2.0	2.25	2.5	2.75	3.0
Correction factor	1.114	1.112	1.009	1.007	1.004	1.002	1.000	0.989	0.996	0.993	0.99

The ratio of L/h should not be less than 2 for rectangular weir. The dimension L, h and F are decided by the above mention procedure. Once these dimensions are finalized, the dimensions of other components of the drop spillway are calculated using the following guiding procedure.

16.2.3 Computation of Hydraulic Components

Height of longitudinal and end sill, $\quad\quad H_L = h/4 \quad\quad\quad\quad\quad\quad$ (16.3)

Height of transverse sill, $\quad\quad\quad\quad S = h/3 \quad\quad\quad\quad\quad\quad\quad$ (16.4)

Minimum length of head wall extension,

E (m) = (3h+0.6) or 1.5 F, whichever is greater. $\quad\quad\quad\quad$ (16.5)

Length of stilling basin or length of apron,

$$= F\left(2.28\frac{h}{F} + 0.52\right) \text{or } 2[F + S + 0.3 - J], \text{whichever is greater} \qquad (16.6)$$

Height of wing wall and side wall at junction,

$$F + h + s - [\frac{L_B + 0.1}{2}], \text{whichever is greater} \qquad (16.7)$$

Height of headwall extension (H_E)

$$H_E = F + S + h \qquad (16.8)$$

The components M and K, as shown inFig.16.1 (a) are calculated by the following equations

$$M = 2\,(F + s + h - J) \qquad (16.9)$$

$$K = L_B - M \qquad (16.10)$$

Depth of cutoff wall: The depth of cutoff wall (C) and toe wall (T) are considered to be same. It can be calculated using the following formula given by USDA,

$$C = T = \frac{1.65(S + 0.4F + 0.75)}{4} \qquad (16.11)$$

Apron thickness: Varies from 0.2 to 0.3 m depending upon the depth of fall. Thickness of apron in plain concrete for different values of overfall F, can be determined from the Table 2.

Table 16.2. Apron thickness

Overfall, F (m)	0.5- 0.75	1.0 – 1.25	1.5 – 1.75	2.0 – 2.25	2.5 – 3.0
Apron thickness (cm)	20	25	25	30	30

For masonry and gabion structures, these thickness may be increased by 1.5 and 2 times respectively.

Wall thickness: Top widths and base width of headwall, sidewall, wingwall and headwall extensions for different wall heights for masonry construction are given in Table 3.

Table 16.3. Drop spillways in stone and masonry

Description	Headwall	Sidewall	Wingwall&Headwall Extension Headwall extension
Minimum top width (m)	0.45	0.3	0.3
Height of headwall (H)	**Recommended base width**		
0.5	0.45	0.3	0.3
1	0.67	0.55	0.4
1.5	1	0.82	0.6
2	1.33	1.1	0.8
2.5	1.67	1.37	1
3	2	1.65	1.2
3.5	-	-	1.4
4	-	-	1.6
4.5	-	-	1.8

Example 16.1:

Design a drop structure which is to be constructed across the gully. The catchment area of the gully is 50 ha. The maximum rainfall intensity was recorded

as 12 cm/h once in 50 - years return period, for the period equal to Tc. The drop of bed is 2 m. (Assume necessary data required).

Solution:

1. Computation of peak discharge

$$Q_{peak} = \frac{CIA}{36} = \frac{0.35 \times 12 \times 50}{36} = 5.83 \text{ m}^3/\text{s}$$

 Assuming the value of runoff coefficient as 0.35.

2. Computation of combinations of L and h for the discharge Q as 5.83 m³/s: it is calculated using following equation.

$$Q = \frac{1.711 L h^{3/2}}{(1.1 + 0.03\, F)}$$

 The different values of 'L' are assumed and the values of 'h', h/F and L/h are computed, as shown in following table.

L (cm)	3	3.5	4	4.5	5
h (cm)	1.17	1.06	0.97	0.90	0.84
h/F	0.56	0.53	0.49	0.45	0.42
L/h	2.56	3.30	4.12	5.0	5.95

For selection of suitable combination of L and h, the value of h/F >0.5 is not suitable. Further, in all case, L/h ratio is greater than 2. Hence L ranges from 4.0 to 5.0 m, may be selected as suitable combinations. Let

Length of crest, (L) = **4.0 m**

and corresponding to this L value, the h = **0.97 m**

3. Computation of design of different hydraulic parts of the drop structure

a.   Minimum length of head wall extension

   E = 3h + 0.6 or 1.5 F whichever is greater

      = 3 x 0.97 + 0.6 or 1.5 x 2

      = **3.51 or 3.0 m**

Between above two values of E, the higher value is 3.51 m, hence it is selected as the minimum length of head wall extension.

b.   Length of apron: The length of apron and stilling basin (L_B) are the same, which is computed by the following equation

$$L_B(m) = F(2.28\frac{h}{F} + 0.52)$$

$$= 2(2.28\frac{0.97}{2} + 0.52) \ = \textbf{3.25 m}$$

c. Height of transverse sill

$h_t = h/3 = 0.97/3 = \textbf{0.32 m}$

d.   Height of longitudinal sill

$S = h/4 = 0.97/4 = \textbf{0.24 m}$

e. Height of wing wall and side wall at junction:

$$J (m) = 2h \text{ or } F + h + s - [\frac{L_B+0.1}{2}], \text{ whichever is greater}$$

$$J = 2 \times 0.97 = 1.94 \text{ or } 2 + 0.97 + 0.24 - [\frac{3.25+0.1}{2}] = 1.58$$

The values of J are obtained as **1.94** m and **1.58** m. Between them the higher value i.e., **1.94** m is selected as the height of wing wall and side wall at the junction point.

f. Dimensions of M and K

$$M = 2 (F + s + h - J)$$

$$= 2 (2 + 0.24 + 0.97 - 1.94)$$

$$= \textbf{2.54 m}$$

and $K = (L_B) - M$

$$= (3.25) - 2.54$$

$$= \textbf{0.71 m}$$

g.   Cutoff wall and toe wall

C = T =

$= (1.65 (0.24 + (0.4 \times 2) + 0.75))/4$

$= \textbf{0.73 m}$

Lesson 17. Structural Design of Drop Spillway-I

17.1 Structural Design

The structural design or stability of any hydraulic structure depends upon the interaction among the water, foundation soil and construction materials. Reinforcement, concrete, brick and masonry (stone) are most commonly used for construction. In addition to this, the dimensional proportion of different components of the structure other than those determined hydraulically, also affect the stability of the structure.

Failure of the head wall is the major problem for drop structure and this may occur due to the following reasons (Table 17.1):

Table 17.1. Factors responsible for different type of failure of hydraulic structure

Failure due to	Factor responsible
Overturning	Inappropriate hydrologic design and substandard construction material
Sliding	Inappropriate structural design unable to sustain the fluid and earth pressure
Tension	Inappropriate reinforcement
Compression	Faulty reinforcement and concretes are not mixed homogenously causing differential settlement
Piping	Internal erosion due to poor construction material and inappropriate foundation.

The structural design of the drop spillway involves thorough check for its stability to withstand above failures. Table 17.2 presents the function of different structural components from stability point view.

Table 17.2. Functions of components of drop spillway

Structural components	Functions
Head wall Extension	Assist in creating stable fill
	Prevents piping under the structure
	Reduce uplift pressure
	Resist sliding of the structure
Toe wall	Prevents piping under structure
	Prevents undermining of the apron
Side wall	Helps in holding a stable fill
	Protects the fill against erosion from the flowing water, Passing over the spillway.
Wing wall	Helps in holding stable fill
	Protects the fill and gully banks against score by flowing Water.
Head wall	It is the main component of the drop spillway and is protected by the above components.

17.2 Check against Overturning

The major factor behind overturning is the hydrostatic pressure caused by water column upstream of structure and active earth pressure due to filling. The structure should be checked for safety against overturning about toe of the structure. There are three scenarios that is to be tested for overturning, these include

Case 1: No earth fill against the head wall; i.e., only hydrostatic pressure exists. In this case the pressure of the water on the structure is maximum at the bottom and nil at the top and the pressure acts at the Centre of gravity of the pressure triangle.

Case 2: Soil filled up to the crest level; both hydrostatic and saturated soil pressure exists and the structure is only subjected to the saturated soil pressure.

Case 3: Soil filled up to the crest elevation and water is flowing over the crest. In this case the structure is subjected to the soil pressure, standing water pressure and pressure due to water flow above the crest.

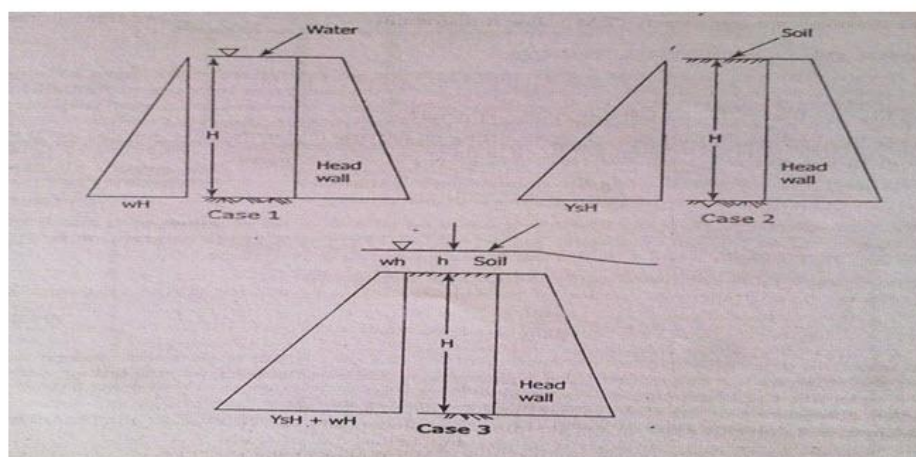For structural design of drop spillways, case 3 stands more appropriately.

Fig. 17.1. Load on the head wall of a drop structure. (Source: Murthy, 1985)

In cases 2 and 3 the pressure (P_s) due to soil fill over the headwall as presented by Murthy (1985) is as follows:

$$P_s = \frac{WH}{2}\left(\frac{1-\sin\theta}{1+\sin\theta}\right) \qquad (17.1)$$

Where, P_s is the lateral pressure acting in horizontal direction; W is the vertical pressure of soil fill given $(\gamma_s H)$, γ_s is the density of earth fill or soil; H is the height of earth fill; θ is the internal friction angle of soil.

After substitution of $W=\gamma_s.H$, the above equation can also be written as:

$$P_s = \frac{\gamma_s H^2}{2}\left(\frac{1-\sin\theta}{1+\sin\theta}\right) \tag{17.2}$$

The forces P_s attempt to induce turning moment (T.M) to the structural components on which it is acting. The weight of different structural components produces retaining moment (R.M.) that counteracts the turning moment.

The factor of safely is given as ratio of restoring moment (R.M) to the turning moment (T.M) i.e.

$$F.S_0 = \frac{\sum R.M.}{\sum T.M.} \geq 1.2 \tag{17.3}$$

If the value of factor of safety against overturning is more than 1.2 (for concrete) and 2.5 for masonry structure, then structure is assumed to be safe.

Restoring moment is the result of the weight of structure and turning moment is due to water and uplift pressure. The turning moment also depends upon the condition or nature of the sub soil strata. The uplift pressure is generally considered equal to the head of the water against headwall at u/s side) and zero at the downstream side. Its line of action is vertically upward from the centre of the structure. For safety point of view, the restoring moment should always be greater than the turning moment.

There may be site situation where the desired factor of safety of the structure is difficult to achieve but it is necessary to construct the structure. In this case, the anchoring is provided to balance the deficit moment between turning moment and restoring moment. In anchor design the deficit moment are balanced using tie rod (steel) which can sustain the pull due to this deficit moment.

17.3 Check against Sliding

The horizontal force acting on the structure in downstream direction may cause failure of structure by sliding. There are two types of forces resisting sliding; one is frictional resistance of the foundation and other is cohesion force of the soil and foundation, both. The sum of horizontal forces ($\sum H$) should always be less than the resisting forces of the structure, for controlling the sliding action.

It is expressed as:

$$\sum H < (\tan \theta . \sum V + C.A.) \qquad (17.4)$$

Where, f is the coefficient of friction (tan θ) of the soil; θ is the angle of repose or internal friction of the soil; $\sum V$ is the total vertical force of the foundation material; A is the area of plane sliding.

To ensure the safety of structure against sliding, the value of sum of all resisting force must be equal to 1.5 times the sum of horizontal forces (i.e$\sum$ $V + C.A = 1.5. \sum$ Hf.). In case, if it is not satisfied, then the cutoff wall and toe of the wall should be constructed for greater depth. This yields more cohesive force. In addition to it, the base of the structure should also be made uneven, to increase the friction resistance of the structure.

Sliding of the structure may also be predicted by a factor, known as factor of safety against sliding. It is given as:

$$F.S_S = \frac{\sum H}{\sum V - \mu} \qquad (17.5)$$

Where, F.S$_s$ is the factor of safety against sliding; $\sum H$ is the sum of all horizontal forces which tend to slide the structure;is the sum of all vertical forces and is the uplift force.

The value of factor of safety against sliding should be greater than or equal to 0.8 for concrete and 0.75 for masonry.

17.4 Check against Piping

The piping type failure in hydraulic structure takes place due to removal of materials from the foundation by seepage water as it emerges from the soil below

the structure. Removal soil material below the structure will ultimately leads to failure.

There are two basic thoughts about piping phenomenon, in which one related with the flow of water through the foundation materials and the other is related with the line of creep. The line of creep is the contact surface between the structure and soil at the base of the foundation as of least resistance to flow.

Several studies have reported that most of the structures are failed due to piping, mainly along the line of creep. The value of safe weighted creep ratio (C_w) of different soil types can be calculated, using the following formula and can be have an idea about the failure of structure due piping. The formula is given as:

$$C_w = \frac{\sum L_H + 2\sum L_V}{3H} \qquad (17.6)$$

Where, C_w is the weighted creep ratio; $\sum L_H$ is the sum of all horizontal or flat contact lengths; $\sum L_V$ is the sum of all vertical or steep contact lengths and His the head between head and tail water depths.

The recommended values of weighted creep ratio for different types of materials are given in Table 17.3. The computed values of C_w should always be either equal to or greater than the recommended values to have stable structure. If it does not satisfy, then depth of cutoff or toe wall should be suitably increased to obtain the desired C_w value.

Table 17.3. Values of weighted creep ratio for different substrata

Soil type	C_w
Clear gravel	5.0
Clean sand or sand and gravel mixture	6.5
Very Fine sand and silt	8.5
Mixture of well graded sand silt and clay(less than 15%)	5.5
Mixture of well graded sand silt and clay(More than 15%)	4.0

Firm clay	2.3
Hard clay	1.8

(Source: USDA, 1957)

17.5 Check against Tension

In case of bricks or stone masonries structure, it should be ensured that tension should not be developed at the base of the head wall, as these materials are not capable to bear tensile stress. This is done by ensuring the resultant force acting on the head wall, passing through the middle third of the base of structure. The position of resultant forces may be calculated by using following equation.
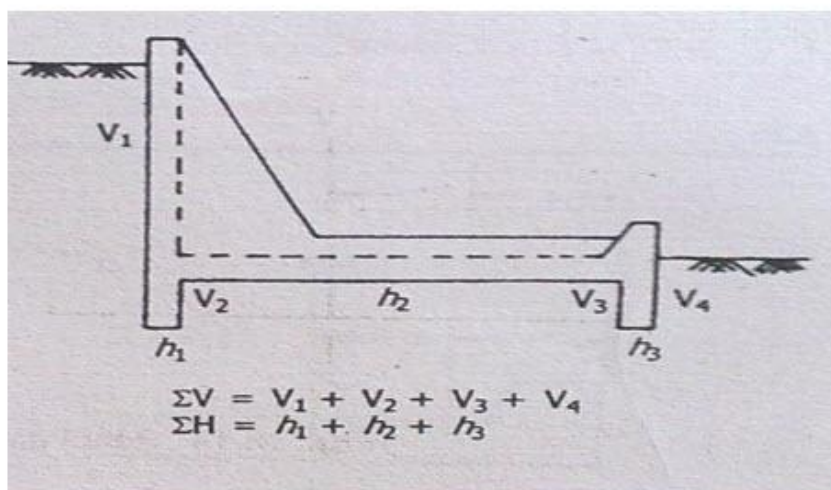

$$\Sigma V = V_1 + V_2 + V_3 + V_4$$
$$\Sigma H = h_1 + h_2 + h_3$$

Fig. 17.2. Calculation of weighted creep (Source: Murthy, 1985)

$$Z = \frac{\Sigma M}{\Sigma V} \tag{17.7}$$

Where, Z is the position at which resultant forces is acting i.e. horizontal distance measured from the wall to the point of action of vertical force.is the sum of all the horizontal forces; is the sum of all vertical forces

The eccentricity (e) is given as:

$$e = \frac{d}{2} - z \tag{17.8}$$

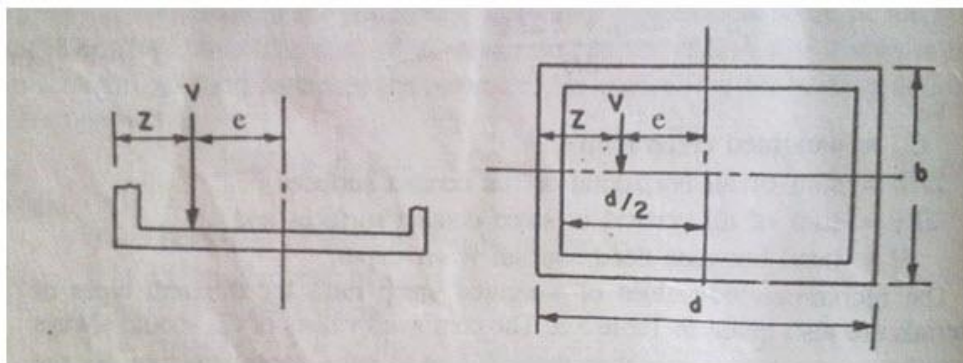It should not be greater than d/6 in which d is denoted as base length.

Fig. 17.3. Resultant forces acting in horizontal and vertical direction on a structure. (Source: Suresh, 1993)

17.6 Check for Compression

It is performed by determining the resultant forces, acting on the structure. The resultant force is due to the load transmitted on the foundation of the structure, as the result of all horizontal and vertical forces acting on the structure. This is also called contact pressure. It is acting vertically downward i.e. compressive nature. This makes the structure stable against floating. The contact pressure may be calculated, using the following formula:

$$P = \frac{\Sigma V}{A}\left(1 \pm \frac{6e}{d}\right) \qquad (17.9)$$

Where, P is the contact pressure; ΣV is the sum of all vertical forces including uplift forces; A is the base area of the structure; e is the eccentricity (i.e. longitudinal distance from the centroid of the base area to the point of application of the resultant vertical forces) and d is the base length of the structure.

<h2>Lesson 18. Structural Design of Drop Spillway-II</h2>

In the previous lectures, concepts of hydrologic, hydraulic and structural design of drop spillway are discussed. In this lecture, structural design will be carried out considering the hydrologic and hydraulic designs carried out in example 16.1.

18.1 Design Example

Design the structure of a straight inlet drop spillway with a straight apron outlet, to be installed for gully control. The expected peak runoff rate through the gully is 5.83 m³/s. For dimension of the structure refer example 16.1. Assume fill condition for the design.

Soil

- Angle of internal friction = 25°

- Cohesion resistance = 500 Kg/m³

- Unit weight of soil = 1900 Kg /m³

Brick

- Unit weight of masonry = 1900 Kg /m³
Foundation

- Made of firm clay. Its creep ratio, C_w = 2.3
Water

- Unit weight of water = 1000 Kg/m³
Solution:

From example 16.1

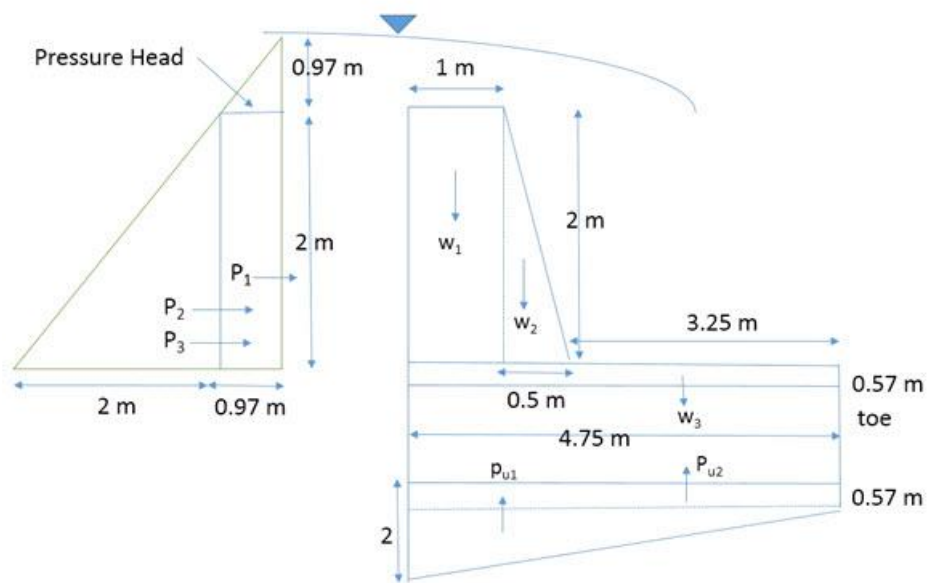The crest length of weir, L = 4.0 m

and   h = 0.97 m

Apron length, L_b= 3.25 m

Cutoff wall / toe wall = 0.567 m = 0.57 m

Total uplift pressure head = F = 2 m

Analysis of forces

Forces	Volume of component (m³)	Material Density (Kg/m³)	Force/unit length of structure (Kg/m)	Lever arm length (m)	Moment kg	Type of moment
P_1	2 x 0.97 x 1	1000 (water)	1940	At 0.5 x 2 = 1.0 from base	1940	overturning
P_2 and P_3	0.5 x 2 x 2 x 1	1000 (water), 1900 (soil)	2000+3800 =5800	At 1/3 x 2 = 0.66 from base	3862.8	Overturning
P_{u1}	(0.5 x 1.43 x 4.75) x 1	1000 (water)	3396.25	At 2/3 x 4.75 = 3.16 from toe	10754.8	Overturning

P_{u2}	0.57 x 4.75 x 1	1000 (water)	2707.5	At 0.5 x 4.75 = 2.375	6430.31	Overturning
Total overturning moment						**22987.91**
W1	2 x 1 x 1	2100 (Bick masonry)	4200	At 4.75 – 0.5 4.25 from toe	17850	Restoring
W2	0.5 x 2 x 1 x 0.5	2100 (Brick masonry)	1050	At 3.25 + (2/3) x 0.5 = 3.55 from toe	3762.5	Restoring
W3	4.75 x 0.57 x 1	2100 (Brick masonry)	5685.75	At 0.5 x 4.75 = 2.375 from toe	13503.65	Restoring
Total restoring moment						**35116.15**

Safety against Overturning: For safety against overturning,

Total restoring moment > Total overturning moment

i.e.,
$$\frac{\text{restoring moment}}{\text{overturning moment}} \geq 1.5$$

$$\frac{35116.15}{22987.91} \geq 1.5$$

Therefore, the structure designed is safe against overturning.

Safety against Sliding: For safety against sliding, ΣH shouldn't be greater than f.ΣV + CA, where horizontal forces, H= P_1 +P_2+ P_3, Vertical forces, V = W_1 +W_2 + W_3 – P_{u1} –P_{u2}, and f = tan θ(angle of internal friction of soil), C is the cohesion resistance of soil, and A = area of the plane of sliding.

ΣH = 1940 + 5800 = 7740 kg/m

ΣV = 4200+1050+5685.75 – (3396.25+2707.5) = 4832 kg/m

fΣV + CA = 0.466 x 4832 + 500 x 4.75 x 2 =7001.712 kg/m

As this is greater than ΣH, the structure is safe against sliding.

Safety against Tension

Assuming restoring moments as positive and turning moments as negative, the value of z is calculated as:

z = ΣH/ΣV

= (**35116.15- 22987.91) /** 4832 = 2.50

e = 2.50 – (4.75/2) = 0.125 m

width (d)/6 = 4.75/6 = 0.791 m

This shows that, e = 0.125 is less than 0.791 and structure is therefore safe against tension.

Safety against Compression

For safety against compression, the forces should be compressive in nature, so that there is no floatation of the structure.

Contact pressure,

$$= \frac{\Sigma V}{A}\left(1 \pm \frac{6e}{d}\right)$$

$$= \frac{4832 \text{ Kg/m x 2m}}{4.75 \text{ X } 2}\left(1 \pm \frac{6 \text{ X } 0.125}{4.75}\right)$$

= 1017.26 +160.62 Kg / m^2

= 1177.9 Kg / m^2 at upstream side

=856.64 Kg / m^2 at downstream side

Hence the resultant pressure is downward and, therefore the structure is safe against compression.

Safety against Piping

For safety against piping, the weighted creep ratio C_w should be higher than the recommended value.

$$C_w = \frac{\sum L_H + 2\sum L_V}{3H}$$

Assume depth of cutoff walls on the upstream and downstream sides = 1 m

Here,

$\sum L_h$ = 4.75 m

$2\sum L_v$ = 2 x (2 x 1.57 + 2 x 1) = 10.28 m

H = 2 m

Therefore,
$$C_w = \frac{4.75 + 10.28}{3 \times 2}$$
= 2.505

and for clay C_w = 2.3 which is less than 2.505 therefore, the structure is safe against piping. Hence the dimensions calculated are satisfactory.

Lesson 19. Drop Inlet Spillway

19.1 Components of Drop Inlet Structures

The drop inlet structures are the combination of 3 major components, namely, inlet, conduit and outlet. The structural design includes the determination of specification of these components. The usual function of a drop inlet spillway is to convey a portion of the runoff through or under an embankment without erosion.
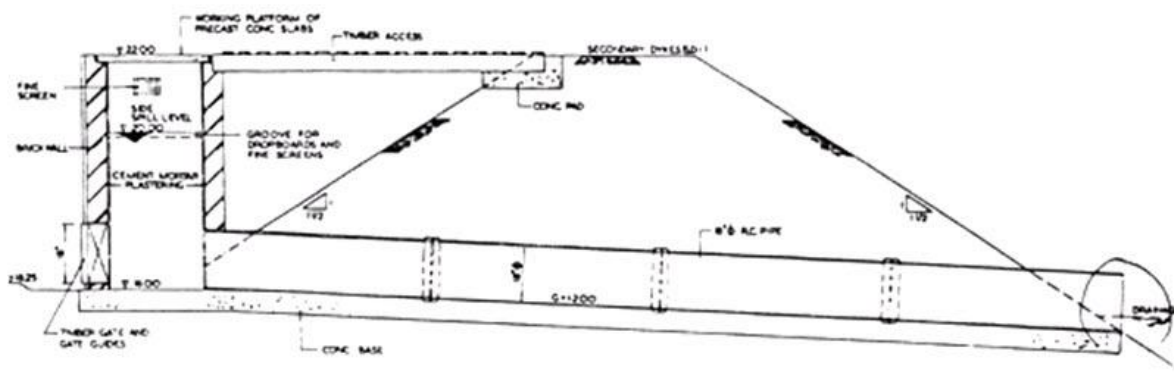

Fig. 19.1. Sketch of drop inlet structures.

(Source: http://www.fao.org/docrep/field/003/ab744e/AB744E09.gif)

19.1.1 Inlet

The inlet is installed having some drop which causes to flow of water from inlet to conduit. Various shapes of the inlet are used in the structures. Those include, box type, catch-pit and funnel shaped. The funnel shaped also known as morning glory or glory hole are widely used in the conditions where large amount of the flow to be handled. Design of inlet includes the determination of size and selection of shape in order to receive the excess flow and transfer it to conduit. The flow carrying capacity of the inlet and conduit and outlet must align adequately for proper functioning of the drop inlet structures. The size of the inlet largely depends on the amount of excess flow to be handled, however the shape depends upon the amount of excess flow as well as site conditions.

19.1.2 Conduit

The flow can be divided in two; first flow over weir in case of inlet and then it transforms to pipe or orifice flow in case of conduit. The flow in the conduit is governed largely by the slope of the conduit. This also influence the energy dissipated in term of head loss due to friction. The term natural slope is defined as

the hydraulic slope for which the head loss due to friction is equated with the head gain due to elevation difference. These are given as

$$H_f = LK_c \frac{v^2}{2g} \qquad (19.1)$$

Head loss due to friction,

In which H_f is head loss due to internal friction; L is length of the conduit, V is velocity of flow and K_c is head loss coefficient.

Natural slope (S_n) can be expressed as

$$S_n = \frac{H_f}{L} = K_c \frac{v^2}{2g} \qquad (19.2)$$

Slope of pipe can be taken as the sine of elevation and length of pipe. The friction factor for different pipe material is given in Table 19.1.

Table 19.1: Friction factor values of different pipe materials

Pipe materials	Friction factor
Aluminium	0.34
Aluminium coated with steel	0.25
Concrete	0.03
Steel Pipe	0.23

19.1.3 Outlet

Different flow regimes at outlet can be expected depending upon the conditions of pipe slope and natural slope. For natural slope being greater than pipe slope and the inlet is submerged, the conduit will flow full and the capacity can be given as:

$$Q = \frac{a\sqrt{2gH}}{\sqrt{1+K_e+K_cL}} \qquad (19.3)$$

In which is the area of cross-section of conduit and is the coefficient of entrance loss at inlet.

For natural slope being less than the pipe slope and outlet is not submerged the flow is controlled by the inlet section of the conduit and can be given by standard orifice formula.

$$Q = aC\sqrt{2gH} \qquad (19.4)$$

In which is the coefficient of discharge for the orifice.

19.2 Uses of Drop Inlet Spillways

A drop inlet spillway is normally used to drop low to medium volumes of water over a sharp incline (30%). The incline height is normally greater than 1 m with no upper limit. Common functional uses are given as:

- Gully control
- Surface water inlets to open ditches and terrace inlets
- Principal spillways for farm ponds or reservoirs
- Grade stabilization of gullies
- Lower end of water disposal system
- Principal spillways for debris basins
- Used as culverts in roadway structures
- Flood prevention structures

19.3 Adaptability

It is a very efficient structure for controlling relatively high gully heads, usually above 3 m. It is well adapted to sites providing an appreciable amount of temporary storage above the inlet. It may also be used for relatively low heads, as in the case of a drop inlet on a road culvert, or in passing surface water through a soil bank along a drainage ditch.

19.4 Advantages

- For high heads, it requires less construction material than a drop spillway.
- Where an appreciable amount of temporary storage is available, the capacity of the spillway can be materially reduced.

- Reduction in construction cost.

- A favorable factor in downstream channel grade stabilization and flood prevention.

19.5 Limitations

Small drop inlets are subject to stoppage by debris. It is limited to locations where satisfactory earth embankments can be constructed. Other disadvantages are as given below:

- The entry point of the spillway normally concentrates the water-flow to a small area. This point can often plug with debris or local scouring can occur due to the high velocity of the water.

- A proper design is required to prevent water from channeling along the sides of the pipe.

- A head (or stage) of water is normally required to obtain full capacity of the inlet (this may make the berm height unreasonably high).

- Spillway systems can be more expensive than comparable system for high flow rates.

19.6 Types of Drop Inlet Spillway

19.6.1 Drop Pipe Structure

This structure consists of two components, a vertical pipe and a horizontal pipe. The drop pipe can be square or round in cross-section which can be constructed of concrete, steel or plastic. The limiting flow factors are the flow over the crest of the vertical pipe and the capacity of the horizontal pipe. The flow over the crest is dependent on the circumference of the vertical pipe and the stage of water over it. The inlet should be as non-restrictive as possible since any obstructions will adversely affect the flow rate. The horizontal pipe is installed into the bottom side of the vertical pipe. The flow through this pipe is dependent on the head from the vertical pipe and the length and roughness of the pipe material. The horizontal pipe is normally smaller in diameter than the vertical pipe since the water running through it is under a higher pressure due to the increased head.

19.6.2 Sloped Pipe Structure

This structure consists of one component, a sloped pipe. Capacity is normally determined by the length and internal roughness of the pipe. Slope of the pipe has very little effect since under most circumstances the "critical slope" (slope at which flow capacity does not increase with increase of slope) is exceeded. Since the water is not forced into the pipe under a high head (as is the case with the horizontal pipe in a drop pipe structure) this structure has a much lower capacity.
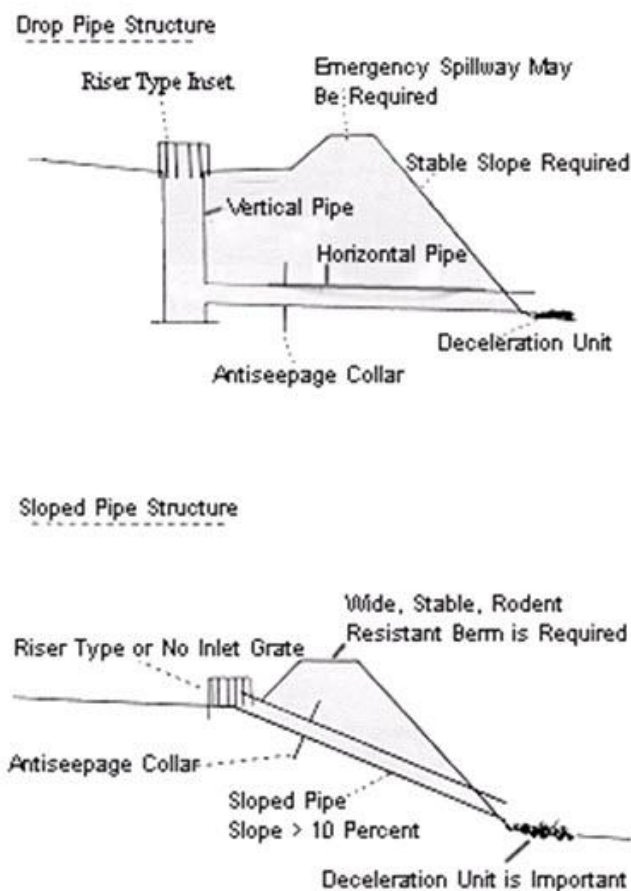
Fig. 19.2. Diagram showing Drop pipe structure and Sloped pipe structure.

(Source: http://www.omafra.gov.on.ca/english/engineer/facts/85-057.html)

19.7 Hydrologic Design

The Hydrologic design consists of knowing both the peak rate of runoff expected and also the inflow hydrograph. The hydrologic design procedure is similar as discussed in case of drop structures. The outflow will not be same as the inflow like other structures.

19.8 Hydraulic Design

It involves the design of earth dam and pipe spillway, as these two components are the main in drop inlet spillway.

19.8.1 Design of Earthen dam

The design of earth dam is also performed by considered all the designs steps – hydrologic, hydraulic and structural design, as in other hydraulic structures. The design of earthen dam suitable to drop inlet spillway is described as follows:

- The upstream and downstream side slopes commonly used in earth dam are 3:1 and 2:1, respectively.

- Top width of dam varies with its height. The minimum top width should be equal to 1.8m for the height of 3.5m. When top width of dam is used as a road, then it should be kept between 2.5 and 3.0m. In addition, there should also be added 30 cm additional top width for each 60 cm dam height.

- In order to make the dam safe against overtopping, there must be added 5% of theoretical dam height or more to the dam height as a settling allowance and 60 cm as freeboard.

- Bottom width of dam is calculated on the basis of side slopes and its height. The bottom width should match the length of conduit, used.

- The side slopes of dam should be protected against erosion. This is performed by making rip- rap, using heavy gravels or rocks when upstream side slope is badly eroded by wave action. The downstream side slope is not affected by wave action, so vegetation can be grown to protect this side.

19.8.2 Design of Pipe Spillway

In drop inlet spillway, the pipe spillway has a vertical section towards upstream face of the dam, called riser, which is connected to the conduit passing through the dam. The top of the riser may be raised up to desired height, as per requirement for providing the grade to conduit and protecting the gully head. The flow capacity may be controlled by the inlet and conduit, both. To follow the hydraulic design of spillway, the types of flow occur in conduit should be considered. A typical discharge characteristic curve of drop inlet spillway is shown in Fig.19.3.
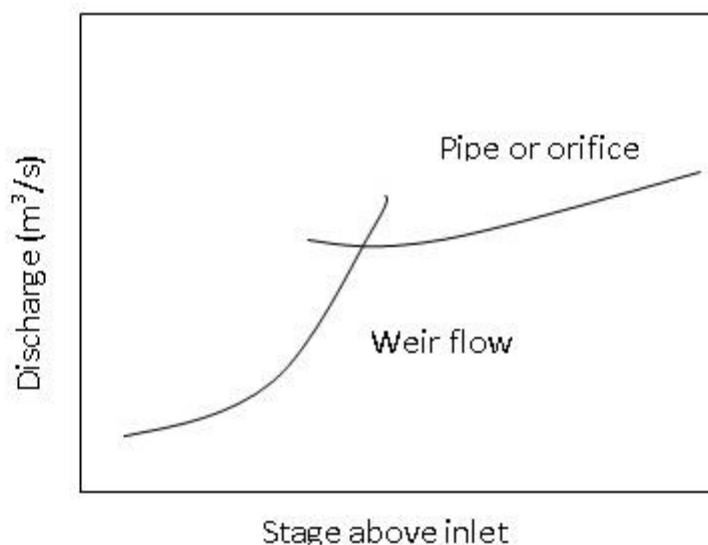

Fig. 19.3. Discharge characteristics curve (Source: Murthy 1994)

Example 19.1: Design Problem

Determine the size of concrete pipe needed in a drop inlet spillway for a peak flow of 3 cu. m per second and a total head of 3m. Determine the slope to be given to the pipe to flow full. Length of pipe = 12m, entrance loss coefficient K_e= 0.5 and friction loss coefficient K_c= 0.03

Solution:

$$\text{Velocity of water in pipe } v = \frac{\sqrt{2gh}}{\sqrt{1+K_e+K_c.l}}$$

$$= \frac{\sqrt{2\times9.81\times3}}{\sqrt{1+0.5+(0.03\times12)}}$$

$$= 5.63 \text{ m/s}$$

$$A = \frac{Q}{v}$$

$$A = \frac{3}{5.63}$$

$$= 0.533 \text{ m}^2$$

Therefore, dia. of pipe, d = 0.82 m; Select 85 cm dia. pipe

Neutral slope,

$$S_n = \frac{K_c\frac{v^2}{2g}}{\sqrt{1-\left(K_c\frac{v^2}{2g}\right)^2}}$$

(Neglect second term in denominator)

$$S_n = \frac{0.03\times5.63^2}{2\times9.81} = 0.048$$

$$S_n =$$

The downstream end of the pipe is kept 30 cm below the upstream end.

Actual slope, S= $\dfrac{0.3}{12}$ = 0.025;   As S< Sn, the pipe will flow full.

Design Problems

1. A 50cm corrugated metal pipe of 75 m length is used in construction of drop inlet spillway. Given the head is 2m, entrance loss coefficient is 0.5. Estimate the peak discharge under pipe flow.
2. Determine the diameter of a concrete pipe needed in a drop inlet spillway for a peak discharge of 12 m³/s if the actual slope of the pipe is 0.025.

Determine the size of concrete pipe needed in a drop inlet spillway for a peak flow of 5 m³/s and a total head of 2m. Determine the slope to be given to the pipe to flow full. Length of pipe = 20m.

Lesson 20. Drop Inlet Spillway Construction

In the previous lecture, general description about the drop inlet spillway along with its applicability, limitation and design considerations are discussed. The design issue for drop inlet structures includes design of inlet, earthen embankment and emergency spillway, selection and placement of pipe and provision to grip the pipe to minimize the piping effect within the body of embankment (anti-seep collars), and design on outlet. The design of a drop inlet spillway cannot be made independently of the design of the earth embankment, emergency spillway, and other elements of the total structure.

20.1 Constructional Features

20.1.1 Anti- Seep Collars

The anti-seep collars are primarily provided to check the adverse effect of piping phenomena by increasing the pipe length and cutting short the capillary gradient along a pipe conduit, which extends through an embankment. It also provide grip to the pipe for firm placement inside the embankment. The anti-seep collar can be of several types depending on the use and extent of piping potential. These are shown in Fig. 20.1.

(a)   (b)

Fig. 20.1. (a) Anti-seep collar using single sheet metal. (b) Anti-seep collar using rubber/plastic sheet.

 The length of collar is kept about 30% of the total seepage length. The collars are constructed in the size of at least 2.5 m width and 2 m in height, centered around the conduit pipe. For concrete anti-seep collars, thickness should be 25 cm and these are reinforced with 1.2 cm dia steel rods at 30 cm interval. The number of collars to be required depends on the length of conduit used in the spillway.

20.1.2 Installation of Conduit

Generally, two types of conduits, namely, concrete pipe (drain pipe) and corrugated pipe (helical metal pipe) are used in drop inlet structure (Figure 20.2).
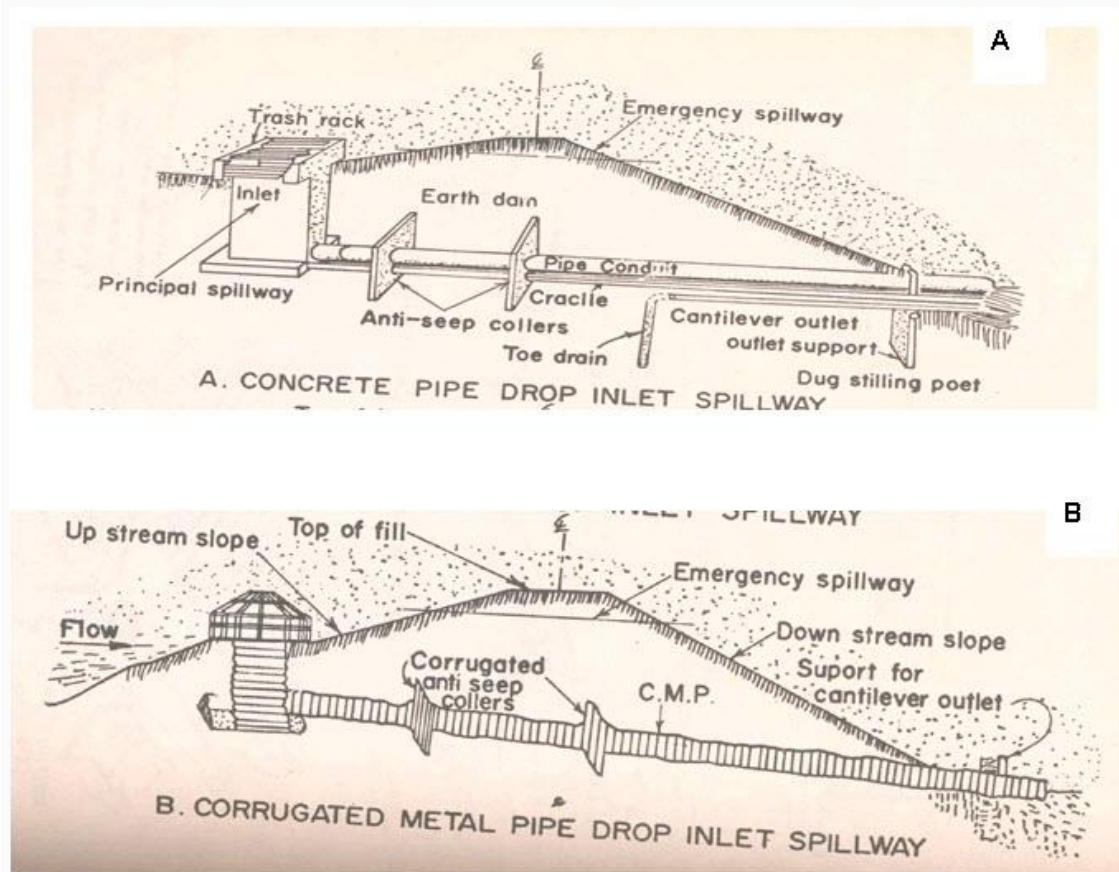

Fig. 20.2. Different type of pipe in let spillway; (a) Concrete pipe, (b) Corrugated metal pipe. (Source: Singh et al., 1990)

For smaller quantity of flow, helical metal pipe are used whereas for large quantity of flow, concrete pipes are used. In case of helical pipe usage, heavy pipe should be used and a coating of water repellent such as bitumen should be performed to increase the life of the pipe. The joints must be sealed to water tight using material such as rubber gasket fixed with Anti-seep collar to prevent the seepage along with the pipe as well as through the joints. The strength of the pipe and collar should be sufficient to bear the load of the embankment. If using cement concrete anti-seep collar, care must be taken for perfect alignment of the pipe because even little distortion will cause rupture of the collar. For ensuring uniform settlement along the pipe, attempt should be made to complete the installation of pipe in one go. When large size cement concrete pipe are planned, care should be taken to ensure that the concrete pipe has proper steel reinforcement and possess sufficient compressive strength to bear the load of the embankment and water flowing within it. The other care given below should be taken as well. Anti-seep collars,

when used, should be installed around all conduits through earth fills according to the following criteria:

- Enough collars should be placed to increase the seepage length along the conduit by a minimum of 15%. This percentage is based on the length of pipe in the saturation zone.

- Maximum collar spacing should be 14 times the minimum projection above the pipe. The minimum collar spacing should be 5 times the minimum projection.

- Anti-seep collars should be placed within the saturation zone. In cases where the spacing limit will not allow this, at least one collar should be in the saturation zone.

- All anti-seep collars and their connections to the conduit should be watertight and made of material compatible with the conduit.

- Collar dimensions should extend a minimum of 2 feet in all directions around the pipe.

- Anti-seep collars should be placed a minimum of 2 feet from pipe joints unless flanged joints are used.

20.1.3 Cradle to the Conduit

To prevent uneven settlement and to develop hoop stress in the concrete pipes a cradle of masonry or concrete is provided to the conduit. Concrete pipes withstand more loads when hoop stress is developed than otherwise.

20.1.4 Earthen Embankment

Earthen embankment is briefly described in previous lecture and also separately discussed in later.

20.1.5 Emergency Spillway

If the runoff exceeds the design runoff, there is overtopping of the embankment and failure of the structure. To prevent such an occurrence, an emergency spillway is located on the embankment at the convenient location. It leads to downstream of the structure. The channel of the emergency spillway is protected with grass or stone pitching. The flood routing procedure gives the elevation at which the emergency spillway is to be located. Emergency earth spillways should have the capacity to discharge the peak flow from the watershed, resulting from a storm expected to occur once in 25 years, where the flow in the principal spillway is appreciable, the design capacity of the emergency spillway may be reduced by that amount.

20.1.6 Stone Pitching

It is recommended on the upstream side on the embankment and downstream side beyond the outlet to prevent soil erosion.

20.2 Maintenance

Any erosion control system needs regular attention to prevent any weak points and consequent failure. A checklist for spillways should be followed as below:

1. Obstructions in the inlet or the spillway should be removed. If these obstructions reoccur frequently a different inlet should be installed.

2. Watch for cracks in the berm or spillway foundation. If cracks occur, immediate repair will be required. Often the back slope will have to be decreased to prevent further failure.

Maintenance and inspection is especially important in the first couple of years after installation since the vegetation will not have developed fully and earth settlement may still be taking place.

Design Problem

The design discharge for drop inlet spillway are determined by the following relationship (Singh, et al. 1990):

$$\frac{V_s}{V_r} = 1 - 2\frac{Q_o}{Q_i} + 1.8\frac{Q_o^2}{Q_i} - 0.8\frac{Q_o^3}{Q_i} \qquad (20.1)$$

Where, V_s is the volume of temporary storage (ha-m), V_r is the volume of runoff (ha-m), Q_o is there quired principal spillway discharge (cumec) and, Q_i is thepeak flow from design storm (cumec).

The above polynomial equation is solved for different $\frac{V_s}{V_r}$ and $\frac{Q_o}{Q_i}$ and the values are given in Table 20.1 for quick solution. For intermittent values of $\frac{V_s}{V_r}$ and, $\frac{Q_o}{Q_i}$ do the interpolation.

Table 20.1. Estimating principal spillway discharge allowing for temporary storage (for watershed of less than 100 ha)

$\dfrac{V_s}{V_r}$	$\dfrac{Q_o}{Q_i};$									
	0.00	**0.01**	**0.02**	**0.03**	**0.04**	**0.05**	**0.06**	**0.07**	**0.08**	**0.09**
0.0	1.00	0.99	0.98	0.96	0.95	0.94	0.92	0.91	0.90	0.88
0.1	0.87	0.85	0.84	0.82	0.81	0.79	0.78	0.76	0.74	0.73
0.2	0.72	0.70	0.68	0.67	0.65	0.64	0.62	0.61	0.60	0.58
0.3	0.57	0.55	0.54	052	0.51	0.50	0.49	0.47	0.46	0.45
0.4	0.44	0.43	0.42	0.41	0.40	0.39	0.38	0.37	0.36	0.35
0.5	0.34	0.33	0.32	0.31	0.30	0.29	0.28	0.27	0.27	026
0.6	0.25	0.24	0.23	0.23	0.22	0.21	0.20	0.20	0.19	0.18
0.7	0.18	0.17	0.16	0.15	0.15	0.14	0.14	0.13	0.12	0.12
0.8	0.11	0.11	0.10	0.09	0.09	0.08	0.08	0.07	0.07	0.06
0.9	0.05	0.05	0.04	0.04	0.04	0.03	0.02	0.02	0.01	0.01

Fig. 20.3 along with Table 20.2a and 20.2b can be used to determine the capacity of 20 cm and 30 cm corrugated metal (CM) pipe drop inlets, as well as the height of riser required to provide the capacities for larger concrete pipes of 20 m length with correction factors for other lengths given in Table 10.3. The reinforced concrete pipes are used as drop inlet for embankment height of more than 6m. The concrete pipe must be cradled and bedded properly.

Size of riser pipe in relation to conduit pipe is determined as follows:

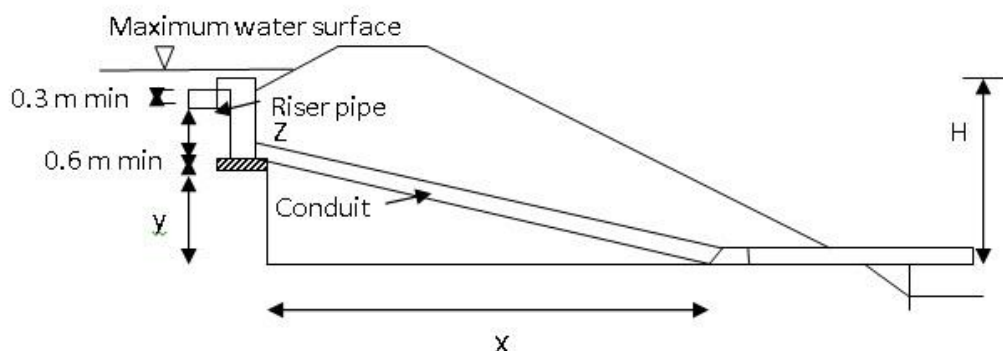| Inlet proportion					
Pipe conduit dia, D (cm)	20-30	45	60	75	90
Pipe riser dia, D (cm)	45	60	75	90	120

Fig. 20.3. Cross-section of dam and pipe spillway.

Determining height of riser: Using Table 20.2a and predetermined slope, find value of discharge for the selected conduit size. Compare this value of discharge with values given Table 20.2b.

(a)   If Table 20.2 discharge value > table 20.2b discharge value (for the design head, conduit length and conduit size) riser height (z) should be minimum 5 times diameter of riser pipe (5D) to provide full pipe flow.

(b)   If discharge is equal to or less than the table value of 20.2b, the riser height (z) should be less than 5D (minimum height = 0.6 m).

The outlet of the drop inlet spillway should be in line with the downstream channel. The layout providing the shortest conduit will exist when the conduit is straight and at a 90° angle with the centerline of the embankment.

For other diameter concrete drop inlet and conduit with 20 m length, the discharge capacity can be estimated using Table 20.3. For other length of conduit, the correction factors given in Table 20.3 are used to determine various dimension of pipe conduit.

Table 20.2a. Capacity for 20 and 30 cm dia pipes

Slope (%)	Discharge (cumec)		Slope (%)	Discharge (cumec)	
	20 cm pipe	30 cm pipe		20 cm pipe	30 cm pipe
5	0.04	0.116	18	0.076	0.221
6	0.042	0.127	19	0.076	0.227
7	0.048	0.139	20	0.079	0.232
8	0.051	0.147	21	0.082	0.238
9	0.054	0.156	22	0.082	0.244
10	0.057	0.164	23	0.085	0.249
11	0.059	0.173	24	0.088	0.255
12	0.062	0.181	25	0.088	0.26
13	0.065	0.187	26	0.091	0.263
14	0.068	0.19	27	0.093	0.269
15	0.068	0.201	28	0.093	0.275
16	0.071	0.207	29	0.096	0.28
17	0.074	0.215	30	0.096	0.286

Table 20.2b. Capacity (cumec) for pipe inlet

Head (H)		20 cm conduit-45 cm riser for pipe length of			30 cm conduit-45 cm riser for pipe length of		
Ft.	m.	50' or 15.24m	70' or 21.34m	90' or 27.43m	50' or 15.24m	70' or 21.34m	90' or 27.43m
5	1.52	0.051	0.045	0.04	0.14	0.12	0.11
6	1.83	0.057	0.048	0.042	0.16	0.14	0.12
7	2.13	0.059	0.054	0.048	0.17	0.15	0.13
8	2.44	0.065	0.057	0.051	0.18	0.16	0.14
9	2.74	0.068	0.059	0.054	0.19	0.17	0.15
10	3.05	0.074	0.062	0.057	0.2	0.18	0.16
11	3.35	0.076	0.065	0.059	0.21	0.19	0.17
12	3.66	0.079	0.068	0.062	0.22	0.2	0.18
13	3.96	0.082	0.071	0.065	0.23	0.2	0.18
14	4.27	0.085	0.074	0.065	0.24	0.21	0.19
15	4.57	0.088	0.076	0.068	0.25	0.22	0.2
16	4.88	0.091	0.077	0.071	0.25	0.22	0.2
17	5.18	0.093	0.082	0.072	0.26	0.23	0.21
18	5.49	0.096	0.085	0.076	0.27	0.24	0.22

| 19 | 5.79 | 0.099 | 0.088 | 0.076 | 0.28 | 0.25 | 0.22 |
| 20 | 6.1 | 0.102 | 0.088 | 0.079 | 0.29 | 0.25 | 0.23 |

Table 20.3. Discharge capacity Q, in cumec (Full pipe flow assumed) for R/C drop inlet, $K_e + K_b$ = 0.65 with 20 m of R/C conduit; n = 0.013.

(Note: Multiply by correction factors for other pipe lengths)

Head dia.(m)	30cm	45cm	60cm	75cm	90cm
1.0	0.17	0.44	0.83	1.36	1.93
1.5	0.20	0.54	1.02	1.65	2.47
2.0	0.24	0.61	1.18	1.92	2.85
2.5	0.26	0.68	1.30	2.16	3.18
3.0	0.28	0.75	1.44	2.36	3.50
3.5	0.33	0.81	1.55	2.54	3.79
4.0	0.34	0.86	1.66	2.71	3.83
4.5	0.35	0.92	1.77	2.87	4.27
5.0	0.37	0.97	1.85	3.03	4.51
5.5	0.40	1.02	1.95	3.18	4.74
6.0	0.41	1.06	2.04	3.36	4.97
6.5	0.44	1.12	2.12	3.46	5.13

7.0	0.45	1.16	2.19	3.58	5.32
7.5	0.47	1.19	2.27	3.70	5.50
L(m)	Correction factors for other pipe lengths				
15	1.076	1.055	1.034	1.024	1.014
20	1.000	1.000	1.000	1.000	1.000
25	0.938	0.947	0.957	0.959	0.961
30	0.889	0.905	0.917	0.935	0.937

Lesson 21. Chute Spillway

21.1 Chute Spillway and Its Uses

Chute spillway is an open channel like structure, which is constructed on steep slope of the gully face with a suitable inlet and outlet. It usually consists of an inlet, vertical curve section, steep-sloped channel and outlet. The major part of the drop in water surface takes place in a channel. Flow passes through the inlet and down the paved channel to the floor of the outlet. It handles the flow having supercritical velocity.

The chute, specially the concrete chute is adapted to high overfall, where a full flow structure is required and where site conditions do not permit the use of a detention type structure. It may also be used with detention dams, taking advantage of the temporary storage to reduce the required capacity and the cost of the chute. It is usually more economical than a drop-inlet structure, when large capacities are required. For high drop (3 to 6 m) and discharge capacity, chute spillways are cheaper than drop spillways as they require less construction material. A typical chute spillway is shown in Fig. 21.1.

Chute spillway is generally used

(i) to control gully head;

(ii) to convey the runoff from upstream areas into the gully, very smoothly without erosion;

(iii) as a structure for flood prevention, water conservation and collection of sediments and

(iv) for controlling the gradient of natural or artificial channels.

Fig. 21.1. One of Alqueva dam chute spillway, in Portugal.

21.2 Adaptability

The chute spillways are suitable for following conditions:

(i) For high overfalls, where a full flow structure is required.

(ii) Where site conditions are not suitable for constructing drop spillway.

(iii) This spillway can also be constructed in combination of check dams and other detention type structures.

21.3 Limitations

The various limitations of chute spillway are as under:

(i) There is considerable danger of undermining due to rodents. For which, additional precautions are required.

(ii) In poorly drained areas, there is problem of seepage. Such areas are not suitable for chute spillways as seepage tends to weaken the foundation. In such areas if constructing of chute spillway is very essential and no other substitutes are available, then provisions to control the seepage problem are essentially made.

(iii) From safety point of view, the construction site should be compacted very well or when earth filling is done that should also be compacted thoroughly. It is an additional work, which takes time and money both.

21.4 Components of Chute Spillway

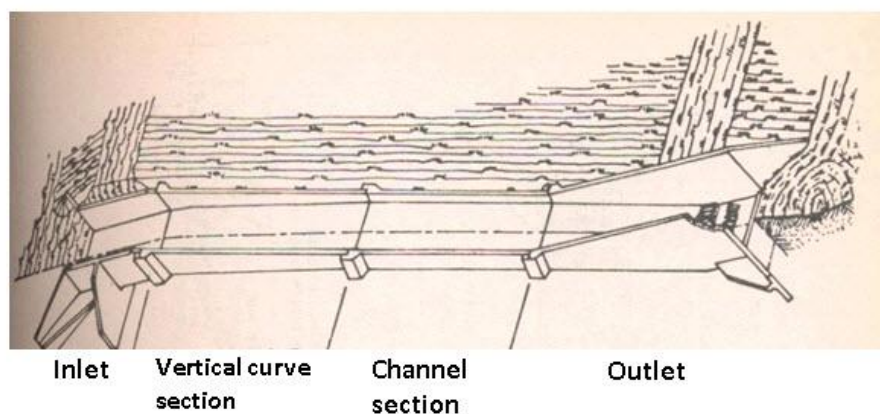The various components of standard chute spillways are shown in Fig. 21.2.

Fig. 21.2. Various components of chute spillway.

21.4.1 Inlet

The following three types of inlets are used with chute spillways:

(i) Straight inlet

(ii) Flared or side channel inlet

(iii) Box (rectangular) type inlet

The box type inlet is generally used when straight type inlet is not sufficient to handle the runoff at desire drop. The inlet section governs discharge capacity of structure. Vertical walls extending into the soil foundations under the inlet are known as cutoff walls. Their main purpose is to prevent water seepage under the structure. Similar walls, extending laterally from the inlet to prevent seepage and erosion around the ends of the structure, are called headwall extensions. These walls also protect against burrowing animals.

The hydraulic design of inlet involves the design of weir length. It is determined by using the weir formula:

$$Q = \frac{2}{3} \times C_d \times \sqrt{2g} \times L \times h^{3/2} \qquad (21.1)$$

Where, Q is the peak discharge rate, m³/s; C_d is the coefficient of discharge = 0.6; L is the weir length, m and h is the head of flow, m.

21.4.2 Conduit or Chute Discharge Carrier

The conduit is the part of chute spillway which convey water from inlet to the outlet section. It can be rectangular or a trapezoidal channel. Usually the conduit section is adopted considering the dimensions of the inlet section. Sometimes, more or less same section as that of inlet is used for the conduit also. The side walls of conduit confine the flow rate and discharge distribution, too, within the conduit section. The top wall of conduit is constructed in such a way, that it may be flushed with the embankment slope. Manning's formula is used for the design of channel capacity. Design of channel cross-section is similar to the design of open channel, in which bottom width, top width, side slope and depth are determined for a given discharge rate.

21.4.3 Outlet

Outlet section of chute spillway is located at the downstream end. It is also called as energy dissipater because, it dissipates the energy of falling water from higher to lower elevation By decreasing the velocity of flow. Thus it protects downstream area from the soil erosion. At the outlet section, energy dissipates on the concept of hydraulic jump. This hydraulic jump is the formed on the horizontal part of the basin. The Froud number (F) of incoming flow into the outlet and corresponding downstream water depth (d_2) should satisfy the following equation.

$$\frac{d_2}{d_1} = \frac{1}{2}(\sqrt{1 + 8F^2} - 1) \qquad (21.2)$$

Where, V_1 is the sequent depth ratio that means ratio of depth of flow after hydraulic jump and before hydraulic jump.

$$F = \text{Froud number} = \frac{v_1}{\sqrt{gd_1}} \qquad (21.3)$$

Here, V_1 is the flow velocity before occurrence of hydraulic jump and g is acceleration due to gravity.

Chute spillway outlet may include Chute blocks, baffle blocks, stilling basin, end sill and side (training) walls. It is preferable to keep them vertical on water side for the satisfactory formation of hydraulic jump. When the velocity at entry of stilling basin is high, chute and baffle blocks are omitted. The outlet's capacity is verified by different considerations of critical depth of flow. Straight apron can also be used for small structures. Scour at the outlet is one of the important factors leading to failure of structure. Scour may be controlled by giving proper consideration in the design to the:

1. Stability of the grade below the structure.

2. Velocities occurring in the downstream channel.

3. Tail water elevations for different flow stages.

4. Dissipation of water energy in the outlet.

Scour below drop spillways or chutes usually are reduced as the tail water elevation is increased.

21.5 Hydrologic Design

The peak flood for which the chute spillway is to be designed will govern the size and capacity. It involves the estimation of run-off rate and flood volume depends on several factors associated with the runoff. The design peak runoff volume using Rational method are computed for the design rainfall intensity of 25-30 years return period.

21.6 Hydraulic Design of Components

Hydraulic design involves determining the dimension of different components of structure, on the basis of expected maximum runoff rate, that has been estimated

in hydrologic design phase. The dimension of structure should be able to handle design runoff. The detailed structural design of chute spillway will be discussed in lesson 22.

Hydraulic design of inlet and conduit is already discussed earlier. Chute spillways are used for different purposes.. These could be as large as spillways for huge dam of river valley projects, or small size structure for gully control and conveyance of water from canals. For gully control as per the recommendation of USDA (Agr. Handbook, 135), the following four types of outlets are generally used.

1. Straight Apron

2. Cantilever

3. SAF (Saint Anthony Falls)

4. Baffle

A straight apron type of outlet is simplest of all, and its design is same as drop spillways. The cantilever type is used where channel grade and soil below the structure are unstable. The SAF (Saint Anthony Falls) stilling basins, developed at Saint Anthony Falls hydraulics laboratory (USA) is the most common of all stilling basins widely used in several applications of chute spillways. The design of SAF will be discussed in next chapter.

Example 21.1:

The chute spillway is to be provided with a straight inlet with peak flow discharge 3.57 m³/s. The depth of flow is to have 1.28 m. What should be the weir length? (C_d = 0.6)

Solution:

Given:

Head = 1.28 m

Q = 3.57m³/s

C_d = 0.6

The discharge over the weir is given by the equation as

$$Q = \frac{2}{3} \times C_d \times \sqrt{2g} \times L \times h^{3/2}$$

$$3.57 = \frac{2}{3} \times 0.6 \times \sqrt{2 \times 9.81} \times L \times h^{3/2}$$

L= 1.39 m **Ans.**

Example 21.2:

A hydraulic jumps forms at the downstream end of chute spillway carrying flow velocity 6 m/s. If the depth of flow before jump is 0.4m. Determine whether a hydraulic jump will occur, and if so, find its depth after hydraulic jump.

Solution:

Velocity of flow, V_1 = 6 m/s

Depth of flow, d_1 = 0.4 m

Froud number on the upstream side, $= \dfrac{V_1}{\sqrt{g \times d_1}} = \dfrac{6}{\sqrt{9.81 \times 0.4}} = 3.0289$

As Froude no. is more than one, the flow is shooting on the upstream side. Shooting flow is unstable flow and it will convert itself into streaming flow by raising its height and hence hydraulic jump will take place.

From eqn. 21.2;   Hence, d_2 **= 1.525 m Ans.**

Lesson 22. Design of Chute Spillway

In the previous lecture, chute spillway and its components of chute spillway are discussed. In this lecture, design of SAF stilling basin and structural design of chute spillway is discussed.

22.1 Design Layout of Chute Spillway

The structural details of straight inlet, straight channel and SAF type outlet are presented in Fig. 22.1.
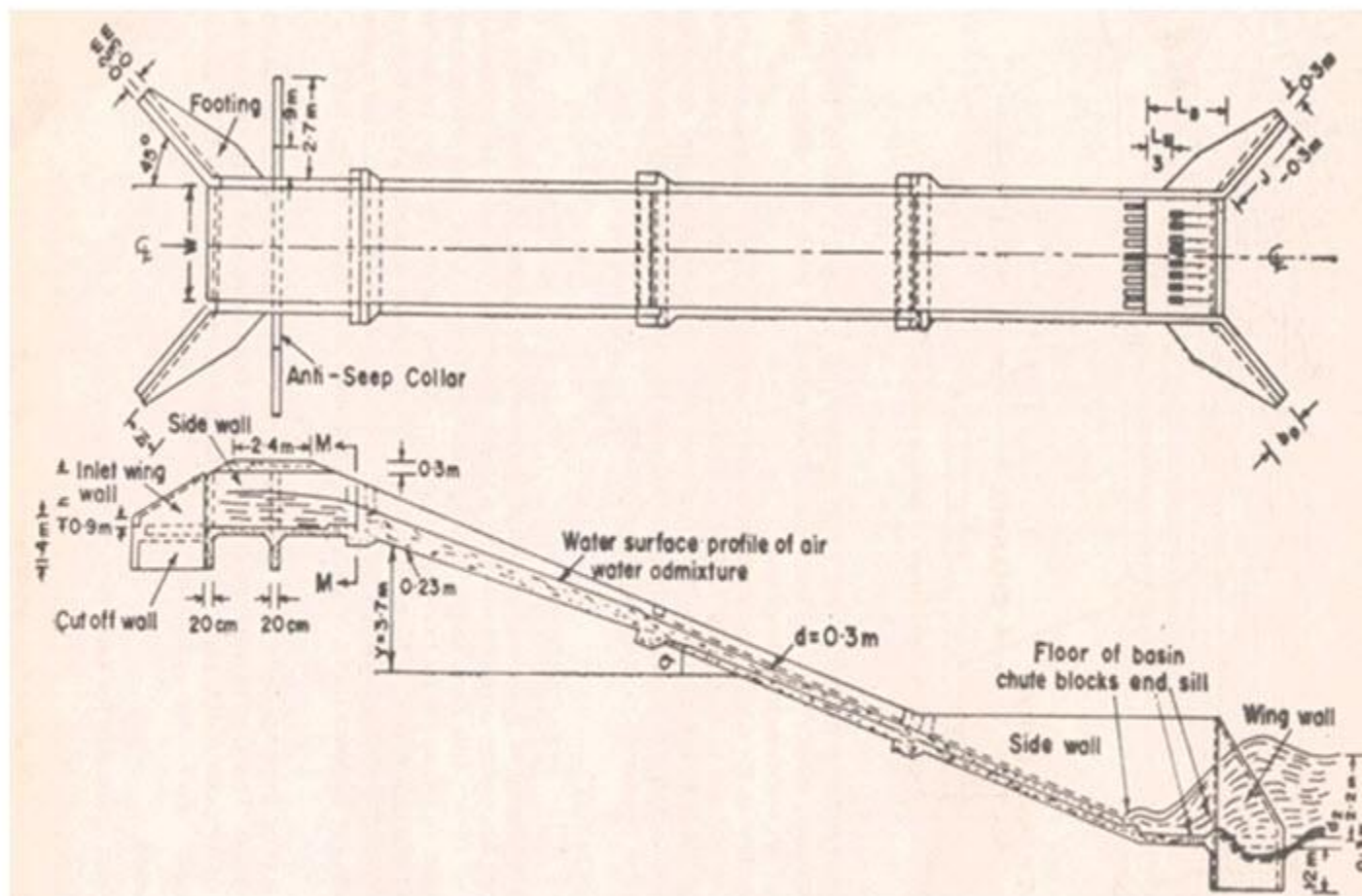

Fig. 22.1. Structural details of a chute spillway.

(Source: Singh et al. 1990)

Figs. 22.2 and 22.3 present the inlet and channel details of chute spillway. The design procedure for inlet is more or less similar to that of drop structures except

design of stilling basin and tail water. The channel is designed in such a way that the flow remains sub-critical.
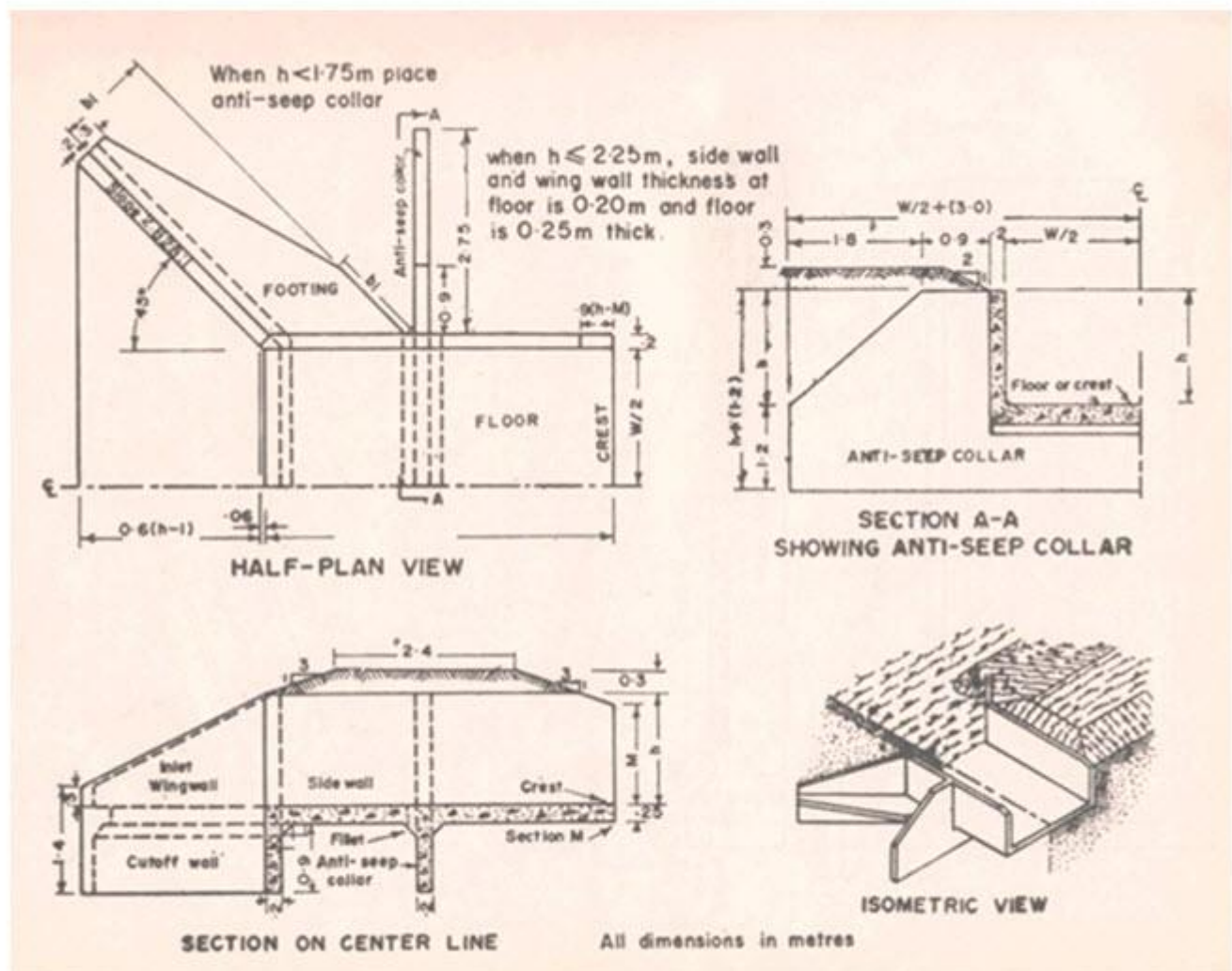

Fig. 22.2. Chute spillway – Inlet details. (Source: Singh et al., 1990)

22.2 SAF Stilling Basin Design

The Saint Anthony Falls (SAF) stilling basin, shown in Fig. 22.4, provides chute blocks, baffle blocks, and an end sill that allows the basin to be shorter than a free hydraulic jump basin. It is recommended for use at small structures such as spillways, outlet works, and canals where the Froude number at the dissipater entrance is between 1.7 and 17. The reduction in basin length achieved through the use of appurtenances is about 80 percent of the free hydraulic jump length. The SAF stilling basin provides an economical method of dissipating energy and preventing stream bed erosion.
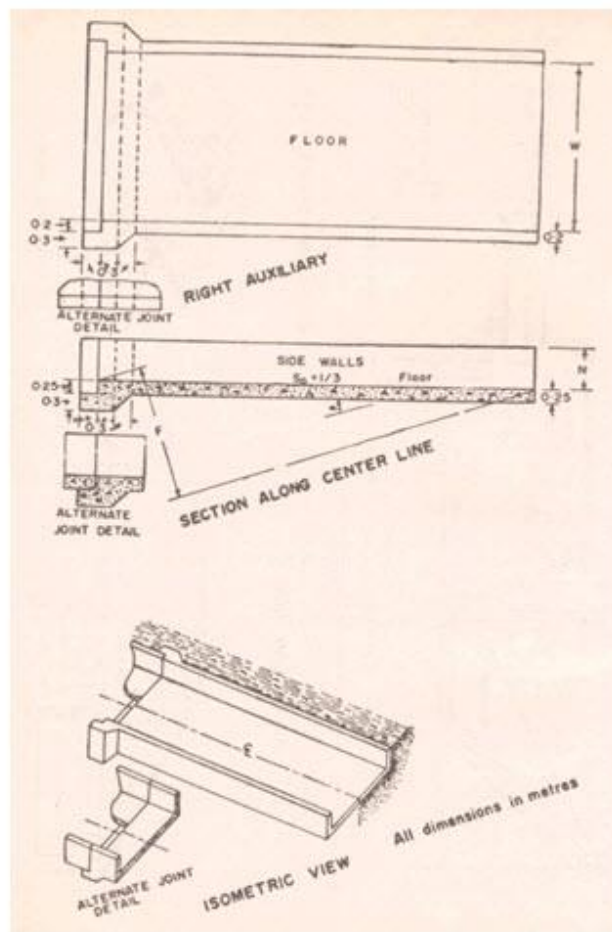
Fig. 22.3. Chute spillway – Channel details. (Source: Singh et al., 1990)
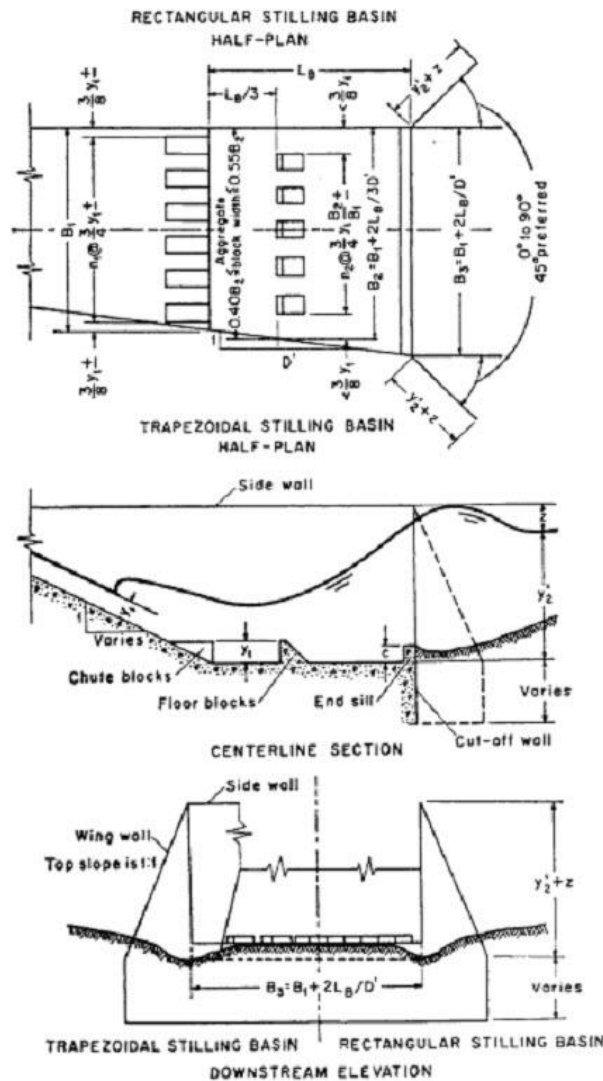
Fig. 22.4. SAF Stilling Basin. (Source: Blaisdell, 1948)

Referring Fig. 22.4, the dimensions of different components of SAF stilling basin are given under (Blaisdell, 1948):

1. Length of stilling basin (L_B) is given by

$$LB = \frac{4.5\, y_2}{F_1^{\,0.76}} \qquad\qquad (22.1)$$

This equation is valid for the Froud number ranging between 1.7 to 17. The Froud number can be calculated by the equation, $F_1 = \frac{V_1}{\sqrt{g y_1}}$, in which y_1 represents the height of chutes and floor blocks.

2. Width and spacing of the chutes as well as floor blocks = $0.75 y_1$

3. Distance from the upstream end of the stilling basin to the floor block = $\frac{L_B}{3}$

4. No floor blocks should be place closer tothe side walls than $\frac{3}{8}$ y_1.

5. The floor blocks should be placed at downstream side from the openings between the chute blocks.

6. The height of the end sill should be equal to 0.07 y_2,

7. The actual depth of tail water above the stilling basin can be computed by the equation,

for F_1= 1.7 to 5.5   (22.2a)

for F_1= 5.5 to 11.0   (22.2b)

for F_1= 11 to 17   (22.2c)

8.The height of the side wall above the maximum tail water depth is given by

$$Z = \frac{y_2}{3} \qquad (22.3)$$

9.Height of wing wall should be equal to the height of stilling basin's side wall.

10.The top of wing wall should have a slope of 1:1.

11. The wing wall may be placed at an angle of 45^0 to the centre line of the outlet.

12. The width of stilling basin at the downstream end (B_3) is given by

$$B_3 = B_1 + \frac{2L_B}{D'} \qquad (22.4)$$

In which, D' is the slope of flare given to the side wall and B_1 = basin width at upstream equal to chute channel width.

13. In order to make structure safe against sliding, a cutoff wall of nominal depth should be provided at the end of stilling basin.

14. In the design of stilling basin the effect of entrained air should be neglected.

22.3 Structural Design

Structural design of chute spillway deals with the determination of the thickness of stable chute floor. The thickness of floor is initially assumed to check the

stability at every section. The structure is considered to be stable, when the weight of the structure (ΔW) is greater than the uplift pressure due to water (Δu) (i.e. $\Delta W > \Delta u$). The uplift pressure over the structure is calculated by drawing the pressure diagram. The pressure is assumed equal to the depth of flow and zero at the beginning of the structure and at the outlet end, respectively. The pressure diagram is then divided into a number of equal parts and area of each part is calculated. The uplift pressure is obtained by multiplying the density of water with area of pressure diagram. The detail is shown in Fig. 22.5.
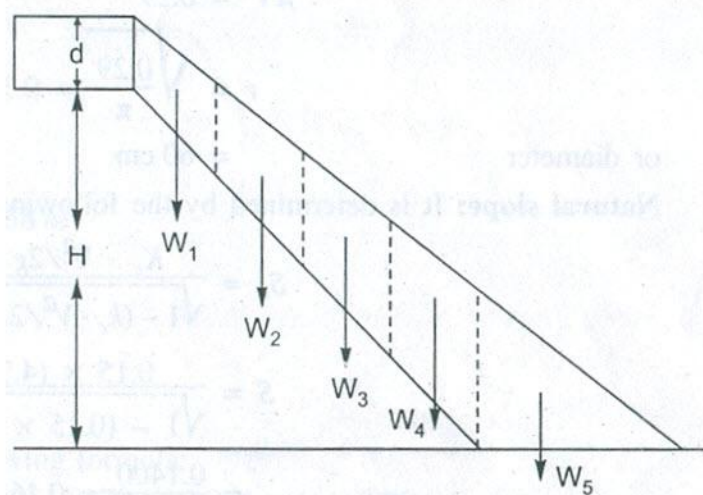

Fig. 22.5. Pressure diagram of chute spillway for determining ΔW and Δu.

(Source: Suresh, 2012)

Example 22.1:

Design a chute spillway for a gully head with a drop of 3.5 m, laid on a 1.5:1 slope. The slope of flare given to the side wall is 1.5:1. The width of the gully is 4.5 m and the catchment area is 25 ha. From the records, it has been found that the maximum intensity of rainfall based on a 50 – year recurrence interval is 10cm / h, for the duration equal to the time of concentration. The runoff coefficient for the rational formula is 0.35. The spillway is to have a straight inlet with a depth of flow of 0.70 m, and a SAF stilling basin at the outlet.

Solution:

The peak flow rate for the design of the structure is determined by using the Rational formula:

$$Q = 0.027\ CIA$$

$$Q = 0.027 \times 0.35 \times 10 \times 25 = 2.36\ m^3 / s$$

Inlet width

The spillway is to be provided with straight inlet, therefore, the flow discharge through it is

$$Q = Q_p = 1.77 \, LH^{3/2}$$

$$2.36 = 1.77 \, L \times (0.70)^{3/2}$$

$$L = 2.28 \, m \, (Approximately)$$

Flow velocity at the stilling basin toe is $$v = \sqrt{2gh_e}$$

Where h_e is the head loss at the toe = (Drop in head) - (10% drop in gully)

$$= 3.5 - \frac{10}{100} \times 3.5 = (3.5 - 0.35)$$

$$= 3.15 \, m$$

Therefore,
$$v = \sqrt{2 \times 9.81 \times 3.15}$$

$$= 7.86 \, m/s$$

Since, Q = (area of the water body on the crest) x (Velocity of flow)

$= L \times y_1 \times v,$ where y_1 is the initial depth.

Therefore, the initial depth of flow,

$$y_1 = \frac{Q}{Lv} = \frac{2.36}{2.28 \times 7.86} = 0.1316$$

$$F_1 = \frac{v}{\sqrt{g\,y_1}} = \frac{7.86}{\sqrt{9.81 \times 0.1316}} = 6.92$$

Froud number,

Sequent depth y_2, the depth of water after hydraulic jump is determined by the following formula;

$$\frac{y_2}{y_1} = \frac{1}{2}\left[\sqrt{1 + 8\,(F_1)^2} - 1\right]$$

$$y_2 = (0.1316) \times \frac{1}{2}\left[\sqrt{1 + 8\,(6.92)^2} - 1\right]$$

$$= 1.223 \text{ m}$$

Design Dimensions

1. Floor and chute blocks height = y_1 = 0.1316 m
2. Spacing and width between floor and chute blocks = $0.75y_1$
$$= 0.75 \times 0.1316 = 0.0987 \text{ m}$$

3. Distance (minimum) of the floor blocks from the side walls = 0.375 x

$$= 0.375 \times 0.1316$$

$$= 0.05 \text{ m}$$

4. Tail water depth over the stilling basin,

$$y'_2 = 0.85 y_2 = 0.85 \times 1.223 = 1.039 \text{ m}$$

5. Stilling basin length = LB $= \dfrac{4.5\,y_2}{F^{0.76}} = \dfrac{4.5 \times 1.223}{6.92^{0.76}} = 1.265 \text{ m}$

6. Side wall and stilling basin height = $l = \dfrac{y_2}{3} + y'_2 = \dfrac{1.223}{3} + 1.039 = 1.44 \text{ m}$

7. Transverse sill height = $0.07 \times y_2 = 0.07 \times 1.223 = 0.0856$ m

8. Side wall freeboard to be provided above the tail-water depth = $\dfrac{y_2}{3} = \dfrac{1.223}{3} = 0.41$ m

9. Wing wall height = stilling basin side wall height = 1.44 m

10. Width of stilling basin

$$B_3 = B_1 + \frac{2L_B}{D'} = 2.28 + \frac{2 \times 1.265}{1.5} = 3.96 \text{m}$$

Structural Design

The weight of the structure should be more than the uplift force. Assume a rectangular channel section, and make thickness of the material 15 cm. The details of the structure pressure diagram shown in Fig. 22.6. The uplift pressure head on the channel of the chute is divided into four sections. Since the channel slope is 1.5:1, therefore, the total horizontal width of the sloping channel section is 5.25 m. Dividing it into 4 equal sections, the water body in each section will have a horizontal width of 1.3125 m.
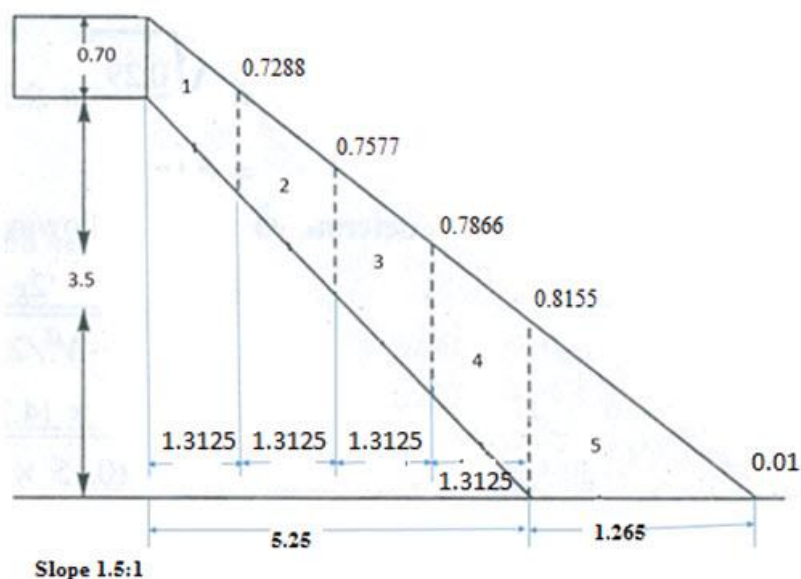

Fig. 22.6. Pressure diagram for chute spillway.

The head at these sections are calculated as under, for example, the head in the first case is

$$= \left[\frac{(3.5 + 0.7)}{5.25 + 1.265} \times (6.515 - 1.3125) \right] - \left[\frac{2}{3} (5.25 - 1.3125) \right] = 0.7288 \text{ m}$$

Similarly, the values of the heads for rest of the sections are calculated (see Fig. 22.6).

The area of water in each section which will have the shape of a trapezium,

$$= \frac{1}{2}(0.7 + 0.7288) \times 1.3125 = 0.9376 \text{ m}^2$$

Therefore the uplift force in the first section = water area × width of the channel × density of water

$$= 0.9376 \; m^2 \times 2.28 \; m \times 1000 \frac{kg}{m^3} = 2137 \; kg$$

The length of total channel

$$\text{The length of total channel} = \sqrt{3.5^2 + 5.25^2} = 6.31 \, m.$$

Length of each section = 6.31/4 = 1.577 m.

Width of channel section = 2.28 m.

The weight of the section of the structure

= Cross-sectional area of the section x thickness of concrete x density of concrete
=1.577 x 2.28 x 0.15 x 2300 = 1240 kg

Weight of the two sides of the wall on each section = 2(1.577×1.44×0.15×2300)

=1567 kg

Total weight of each section = (1567+1240) = 2807 kg

Weight of the Stilling Basin

Weight of the base = L_B x B_3 x Thickness X Density of concrete

Weight of the base = 1.265 x 3.96 x 0.15 x 2300 =1728 kg

The height of the side walls of the stilling basin = 1.44 m

Weight of the side walls = (1.265×1.44×0.15×2300) ×2 = 1257 kg

Total weight of the stilling basin = 1728 +1257 = 2985 kg

The computations pertaining to each section and the associated structure stability are all shown in a tabulated form.

Table 22.2. Cumulative value for each section

Section no.	Head of each section upstream (m)	Head of each section downstream (m)	Horizontal width of the water body (m)	Each section area (m²)	Uplift force (kg)	Cumulative uplift force (kg) Δ u	Weight of each section (kg)	Cumulative weight of structure (kg) Δ W	Stability of structure
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1.	0.7	0.7288	2.28	0.93765	2137.84	2137.842	2807	2807	Safe
2.	0.7288	0.757751	2.28	0.975549	2224.25	4362.094	2807	5614	Safe
3.	0.757751	0.786627	2.28	1.013498	2310.77	6672.871	2807	8421	Safe
4.	0.786627	0.815503	2.28	1.051398	2397.18	9070.057	2807	11228	Safe
5.	0.815503	0.01	3.96	0.52213	2067.63	11137.69	2985	14213	Safe

Therefore the structure designed is safe.

Module 4. Water Storage Structures

Lesson 23. Earthen Embankment

23.1 Introduction

An earthen embankment is a raised confining structure made from compacted soil to confine runoff either for surface storage or for ground water recharge. These are also used for increasing infiltration; detention and retention of water to facilitate deep percolation and also to provide additional storage as in the case of semi dug-out ponds. The cross-section of embankments is usually trapezoidal in shape. When constructed across natural channel to induce channel storage, the embankment also called earthen dam. Further, the embankment depends on its own weight to resist against sliding and overturning whereas foundation work is also included in case of dam. However, at many instances both terms are used as synonymously.

The earthen embankment meant to collect surface runoff or overland flow can be compared with field bunds in many respect except the difference in height and catchment area. In the case of field bunds, the height are usually less than 50 cm and catchment area is limited up to 0.5 ha whereas in case of embankments the minimum height is 3 m and can be as high as 10m and catchment area can be tens of hectare.

Advantages and disadvantage of small earthen embankment/dam:

The major advantages include.

- Local natural materials are used.

- Design procedures are straightforward.

- Comparatively small plant and equipment are required.

- Foundation requirements are less stringent than for other types of dam. The broad base of an earth dam spreads the load on the foundation.

- Earth fill dams resist settlement and movement better than more rigid structures and can be more suitable for areas where earth movements are common.

The disadvantages are

- An earth embankment is easily damaged or destroyed by water flowing on, over or against it. Thus, a spillway and adequate upstream protection are essential for any earth dam.

- Designing and constructing adequate spillways is usually the most technically difficult part of any dam building work. Any site with a poor quality spillway should not be used.

- If it is not adequately compacted during construction, the dam will have weak structure hence prone to seepage.

- Earth dams require continual maintenance to prevent erosion, tree growth, subsidence, animal and insect damage and seepage.

23.2 Types of Embankment

The type of embankment depends on the purpose of storm water to be used such as detention and retention, ground water recharge, surface water harvesting etc. and the available soil material for construction. Broadly, two type of embankment can be assumed, namely, homogenous embankment and zoned embankment.

23.2.1 Homogeneous

It is composed of one kind of material (excluding slope protection). The material used must be sufficiently impervious to provide an adequate water barrier, and the side slopes must be moderately flat for stability and ease of maintenance (Fig. 23.1). The homogeneous soil material is either keyed in to the impervious base (see Fig. 23.1 A) or impervious soil material can be placed as blanket to the upstream slope of the embankment (see Fig. 23.1 B).
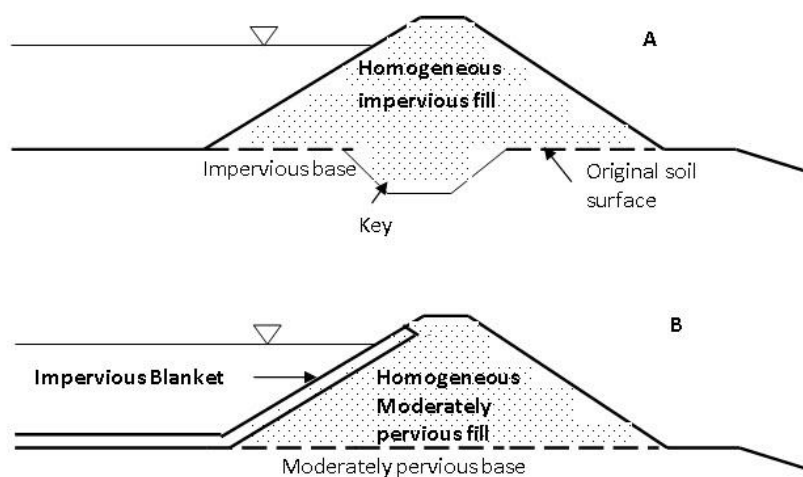

Fig. 23.1. Different types of Homogeneous earthen embankment/dam.
(Source: Samra, et al., 2002)

Diaphragm: The bulk of material used for construction of the embankment is pervious. A thin diaphragm of impermeable material like (concrete, steel, butyl etc.) is provided to act as barrier against seepage through the fill. The full diaphragm condition occurs when the upper crest of diaphragm and design water depth are at the same level (see Fig. 23.2 A). There are cases when the full diaphragm is difficult to adopt, partial diaphragm can be adopted in conjunction

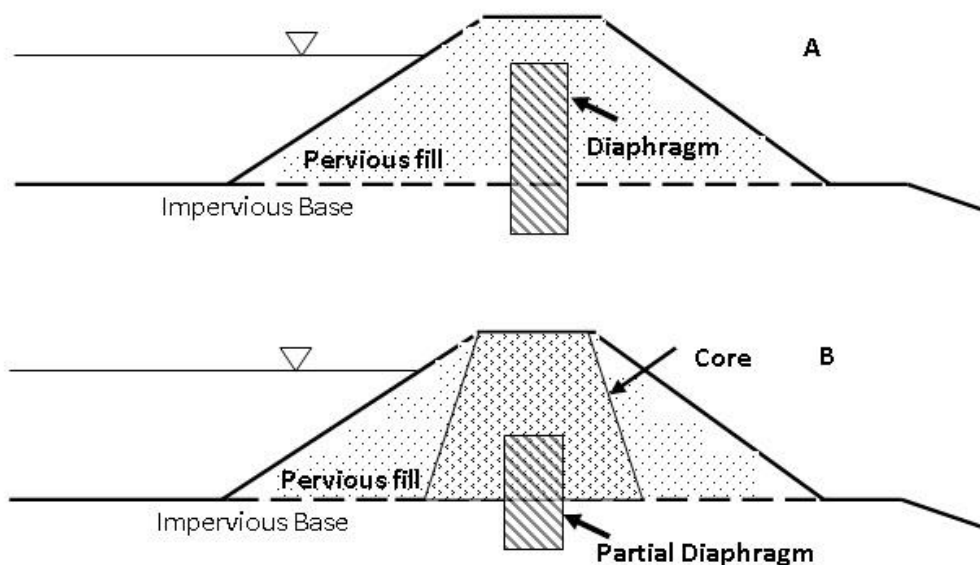with core wall (Fig. 23.2 B).The core walls usually are made of the earth with low hydraulic conductivity.

Fig. 23.2.   Different Diaphragm typed earthen embankment/dam.

(Source: Samra, et.al., 2002)

23.2.2 Zoned Earth Dams

It contains a central impervious core, surrounded by zones of more pervious material, called shells. These pervious zones or shells support and protect the impervious core (Fig. 23.3).
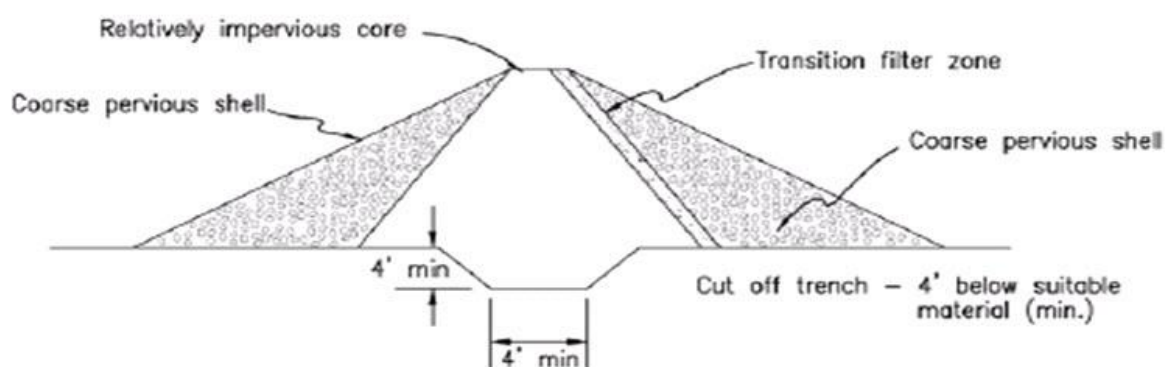

Fig. 23.3. Zoned Earth Dam

23.3 Methods of Construction

Earthen dams are either constructed as rolled fill or hydraulic fill dams.

(a) Rolled Fill Dam: In this type of dams, successive layers of moistened or damp soils are laid one over one another. Each layer not exceeding 20 cm in thickness is properly consolidated at optimum moisture content which is maintained by sprinkling water and compacted by mechanical roller only then the next layer laid.

(b) Hydraulic Fill Dam: In this type of dams, the construction, excavation, transportation of the earth is done by hydraulic methods. Outer edges of the embankments are kept slightly higher than the middle portion of each layer. During construction, a mixture of excavated materials in slurry condition is pumped and discharged at the edges. This slurry of excavated materials and water consists of coarse and fine materials. When it is discharged near the outer edges, the coarser materials settle first at the edges, while the finer materials move to the middle and settle there. Fine particles are deposited in the central portion to form a water tight central core. In this method, compaction is not required.

23.4 Site selection and Investigation for Earthen Embankment/Dam

The embankment should be proposed based on site specific considerations to prevent hydraulic, seepage and structural failure of dam. The common causes of dam failure are illustrated in Fig. 23.4. These are discussed in detail in lecture 24.
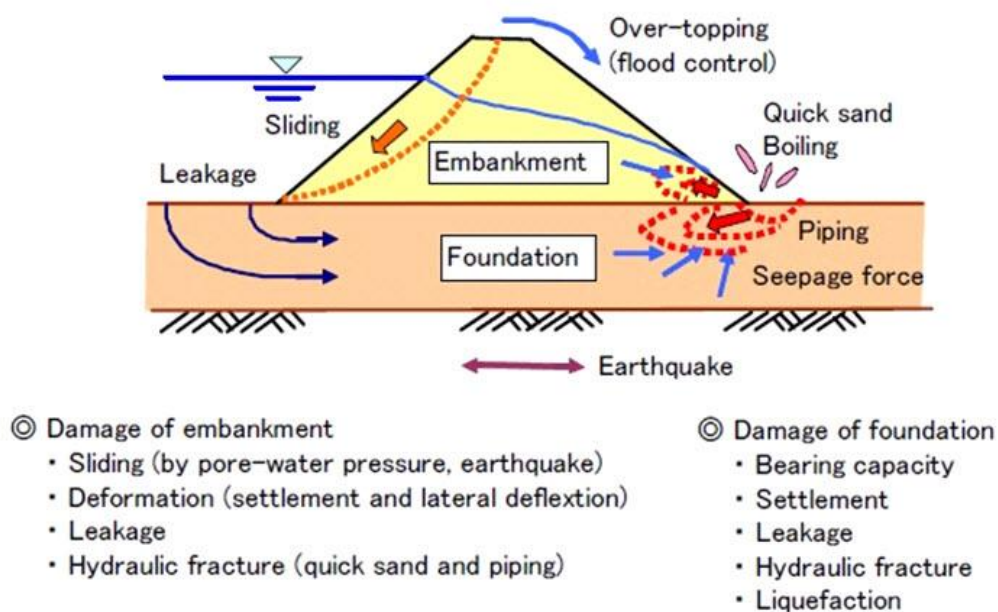

Fig. 23.4. Damages in Embankment and Foundation.

23.5 Design of Earthen Embankment

The various components of earthen embankment includes (a) base/foundation including key trench or cut-off, (b) height, (c) side slopes, (d) top width, (e) free board, (f) toe drains or filter and (g) core wall.

23.5.1 Design Steps

Preliminary Investigations

Although the selection of a suitable site is essentially a field exercise, the use of aerial photographs and large-scale maps can provide a useful assessment of the local topography and hydrological conditions before any field visit takes place. Once possible sites are identified, a field visit is essential. It is important to identify where the water to be stored is to be used: irrigation, for example, involves the conveyance of large quantities of water and, if the dam-site is a long distance away from the cultivated area, much expenditure on pipelines and pumping may be required.

The economic and design implications of each site can be determined from a brief preliminary survey. The survey must be sufficiently accurate and detailed to enable comparative estimates to be made for various heights of dam. The most economic height is usually calculated on the basis of cost per unit volume of water.

Catchment Yield

The catchment yield, 'Y', is based on the expected annual runoff from a catchment and is an important factor in assessing the feasibility of a dam and in determining the required height of the embankment. The latter is important to allow the dam designer to size the dam to suit expected inflow. It is estimated as follows:

- Calculate the annual runoff for the catchment, in mm. This is 'RF'.

- Measure the catchment area 'A' in km^2, upstream of the proposed embankment.

- The annual runoff for the catchment (the catchment yield in an average year), Y, in m^3, is given by:

Y = RF x A x 1 000   (23.1)

Storage Capacity

At this stage, this is worked out as follows:

$$Q = \frac{LTH}{6} \qquad (23.2)$$

Where, Q is the storage capacity in m³ and should not exceed Y above, L is the length of the dam wall at full supply level (FSL) in m, T is the throwback, in m and approximately in a straight line from the wall. H is the maximum height of the embankments, in m, at FSL. 6 is a factor (conservative generally) that can be adjusted (to 5 or 4) with experience and local knowledge.

Equation 23.2 considers the water volume to be an inverted pyramid with a triangular surface area (LT/2) and H/3 for the height/depth, and is a simplification of reality.

Further, general guidelines for homogeneous embankment are given in Table 1 and design steps are given below:

Height of the Embankment

The height of the embankment is determined using depth-capacity curve and reservoir area capacity curve. The other consideration is to obtain the minimum cost of embankment per unit storage. The overall height however, should not exceed 10 m.

Free Board

The free board is the added height to the embankment as a safety measure to prevent the ill effect of waves and runoff generated from storms greater than the designed frequency. The free board is the difference between highest flood level and top level of the embankment after complete settlement taken place. Generally, 10-15% free board is provided to the highest flood level. The free board also depend on length of the embankment. Additional 50 cm free board is provided for the embankment length upto 400m. For 400-800 m length and greater than 800 m length, the additional freeboard of 75 cm and 100 cm is provided respectively.

Top width can be calculated using formula

$$W = \frac{H}{5} + 3 \quad \text{(for very low embankment upto 3 m height)} \qquad (23.3)$$

$$W = 0.55\sqrt{H} + 0.2 H \quad (H \leq 30m) \qquad (23.4)$$

Wave height can be calculated using Hawksley's formula

$$h_w = 0.014(D_m)^{0.5} \qquad (23.5)$$

Where, h_w is wave height (m), D_m is longest fetch distance (m).

Settlement Allowance

Settlement includes the consolidation of the fill material and base material due to self-weight and increased moisture of the embankment body due to storage of water. The settlement allowance varies from 5% (machine compacted) to 10% (manual compacted). The settlement allowance is added to the height of the embankment over and above the calculated free board.

Table 23.1. General guidelines for embankment section (Homogeneous section)

Sl. No	Description	Height upto 5m		Height between 5 and 10 m	
1.	Slopes	U/S	D/S	U/S	D/S
2	Soil				
	Well graded gravel and sand	Not suitable	Not suitable	Not suitable	Not suitable
	Gravel and sand with silt and clay	2:1	2:1	2:1	2:1
	Silt and clay with low compressibility	2:1	2:1	2.5:1	2.5:1
	Silt and clay with high compressibility	2:1	2:1	3.75:1	2.5:1
3	Rock toe height	Not necessary up to 3 m. above 3 m, 1 m rock toe should be provided		Necessary, height should be H/5 where H is the height of embankment	

4	Top width	Minimum 2 m	Minimum 2.5 m
5	Free board	Min 2 m and max 3 m over maximum flood level	

Design Example 23.1:

Design an earthen embankment with the following data

RL of bed surface =116.0 m. RL of Highest Flood Level (HFL) = 122.5 m.

Assume a fetch of 1.5 km and slope of saturation line is 4:1. Length of the embankment is 350 m.

Solution:

Height of water upto HFL =122.5 - 116.0 = 6.5 m.

Calculation of free board

Wave height for a fetch of 1.5 km,

$$h_w = 0.014(D_m)^{0.5}$$

$$h_w = 0.014\sqrt{1500} = 0.542 \text{ m}$$

= 0.542 m

Free board (15%) = 6.5*.15 = 0.98 m

Since, free board is more than height of waves, 0.98 m value is adopted.

Free board adjustment for length of embankment

The length of embankment of 350 m less than 400 m and hence 40 cm to be added to the existing freeboard.

Thus total free board = 0.98+0.40 = 1.38 m

Hence, height of dam = 6.5 + 1.38 = 7.88 m

Assuming 5% settlement allowance, = 7.88*0.05 = 0.394= say 0.40m

Thus overall height of proposed dam =7.88 + 0.40 = 8.28 m say 8.25 m

Top width of the dam = $\frac{8.25}{5} + 1.5$ = 3.15 m (say 3.5 m).

Determining base width of the embankment and its stability

Assuming upstream and downstream slopes of 3:1 and 2.5:1, respectively, the base width is

W_b = 3*overall height of embankment+top width+2.5*overall height of embankment

W_b = 3*8.25+3.5+2.5*8.25 = 48.38m

Normal projection of saturation line over the base = 3*6.5+4*6.5 = 45.5 m.

Since the design base width is greater than the normal projection of the saturation line. Therefore, the d/s face of the embankment will be dry and saturation line will converges within the base width. Hence the design is safe and stable. Typical cross-section of the embankment is shown in Fig..23.5
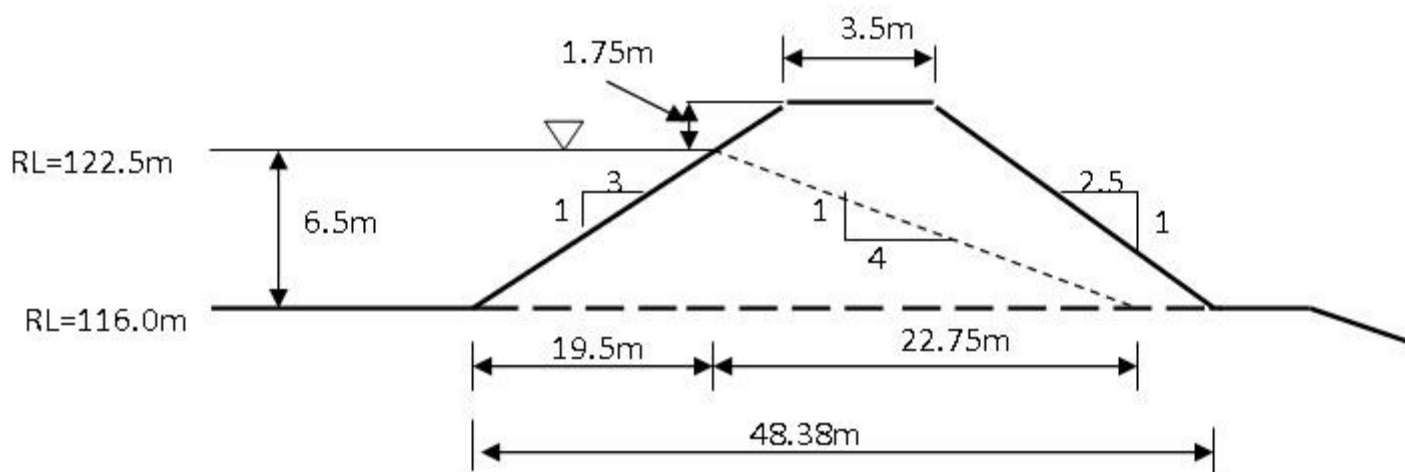

Fig. 23.5. Cross-section of earthen embankment

In case, the seepage line or saturation line does not meet the base of the embankment, then modifications in the different components of the embankment is needed for embankment safety and stability. One such modification can be the case when highest flood level is at 125.25 m instead of present 122.5 m.

Lesson 24. Earthen Dam Drainage and Failure

24.1 Drainage System

Drainage in earthen dams is primarily provided to bring the phreatic line (upper surface of zone of saturation) in the embankment well within the downstream face so that water does not seeps through the body of the dam. It is achieved using the filter material at the downstream end of the earthen embankment (Fig.24.1). It drains the excess water inside the confining structure and thus reduces the pore pressure and thus internal erosion
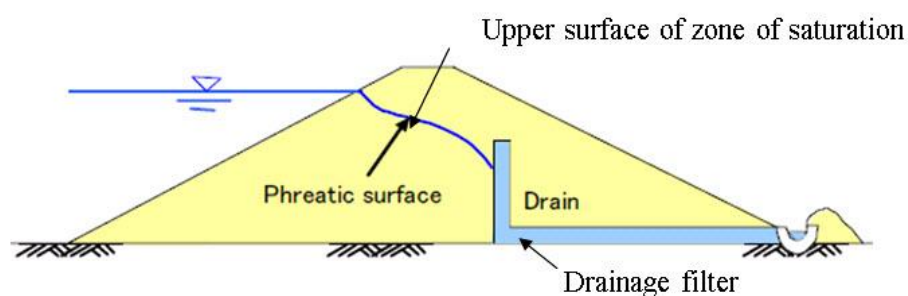

Fig. 24.1. Homogeneous dam showing filter at the downstream side.

(Source: Kunitomo, 2000)

The code of practice recommended by Indian standards (IS 9429:1999) lists different type of filters based of their relative placement in the embankment. These are presented in Fig. 24.2.
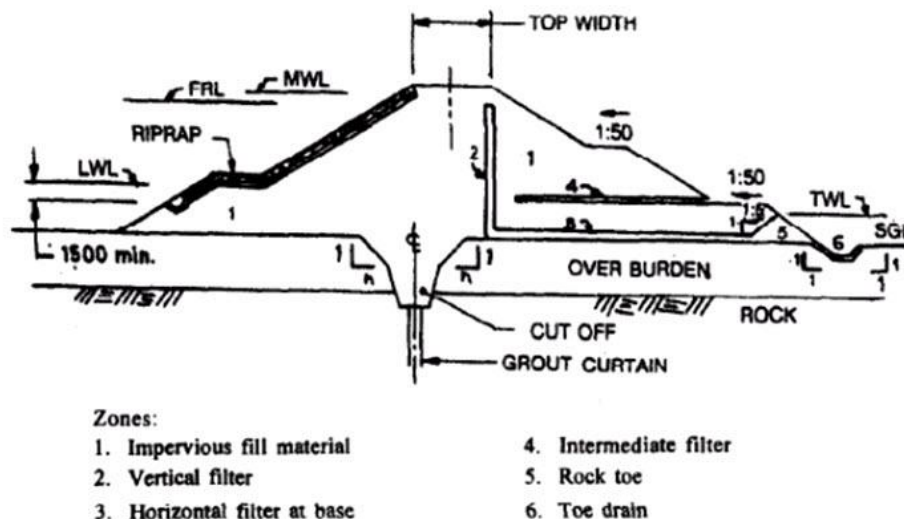
Zones:
1. Impervious fill material
2. Vertical filter
3. Horizontal filter at base
4. Intermediate filter
5. Rock toe
6. Toe drain

Fig. 24.2. Homogenous dam section showing seepage control features.

(Source: Indian standard, IS 9429:1999)

24.2 Criteria for Selection of Drainage Features

The drainage features primarily address the problem of seepage force in form of pore water pressure and piping.

24.2.1 Impervious Fill Material

The seepage control in the dam body is achieved either by providing a barrier or control in drainage. The impervious fill material can act asa barrier to the seepage flow through the body of embankment dam.

24.2.2 Inclined/Vertical Filter with Horizontal Filter

The inclined or vertical filters abutting downstream face of the impervious core are provided to collect seepage water to keep the downstream shell relatively dry. The top level of the vertical filter should be kept as equal to the top of the impervious core level at least. The minimum thickness should be kept as one meter. At the base, this filter should be connected to the horizontal filter that will ultimately carry the seepage water to the toe drain (see Fig. 24.3).
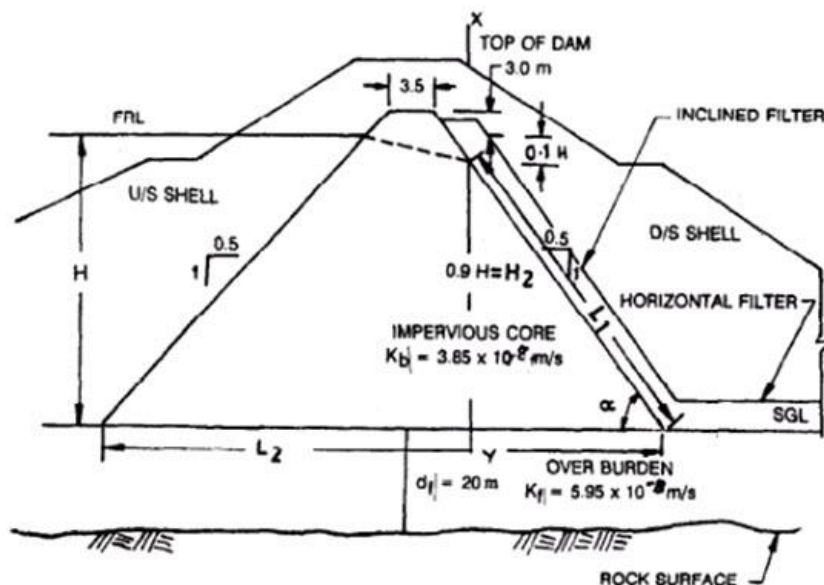
Fig. 24.3. Homogenous dam section showing placement of vertical and horizontal filter. (Source: Indian standard (IS 9429:1999)

24.2.3 Horizontal Filter

Horizontal filter collects seepage water from body of the dam through inclined or vertical filter and transmits it to the toe drain and thus minimize the risk of piping from the dam base. Horizontal filter is usually provided in the downstream side of the homogenous dam. A minimum slope of 1 in 100 towards toe filter has to be maintained for quick disposal of seepage water to the toe drain. A minimum thickness of one meter is required for the horizontal drainage.

24.2.4 Intermediate Filter

Horizontal layer of filter in both upstream and downstream at intermediate level is required in the cases where pore pressure exerted during construction and sudden draw down in expected. Italso helps in reducing pore pressure rise after prolonged rainfall. The thickness and vertical interval are decided on the basis of height, permeability of earth of dam body and filter. However, provision of 0.6 m thickness at 6.0 m interval is considered as adequate for small earthen dam. This filter can be extended upto outer slope of the embankment but care should be taken to avoid the connection with vertical/inclined filters.

24.2.5 Rock Toe

The major function of the rock toe is to protect the lower part of downstream embankment.

24.2.6 Toe Drain

Toe drain (Fig. 24.2) is provided at the downstream side of the earthen 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.
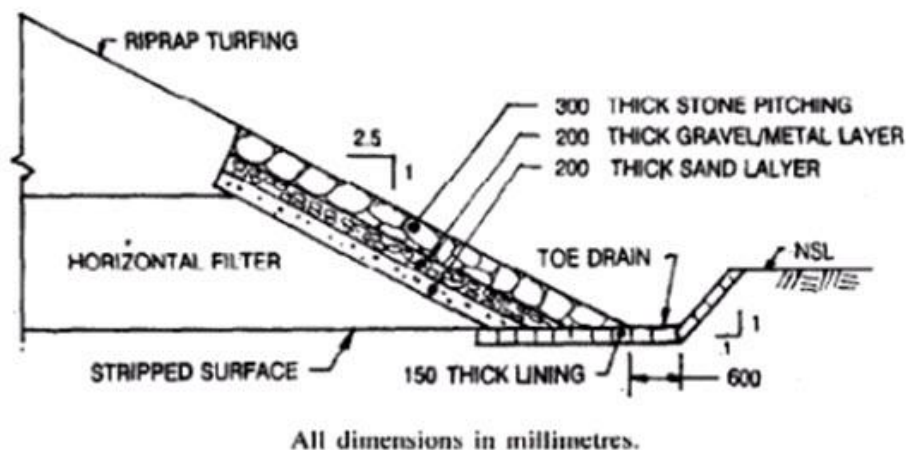
Fig. 24.4. Placement of rock and toe drainage.

(Source: Indian standard, IS 9429:1999)

24.3 Design Criteria for Filter Material

The filter material used for drainage system should satisfy the following criteria.

1. Filter material should be more pervious than base material

2. The particle of base material should not clog the voids of filter material

Based on percentage of particles finer than 75 micron, the base soil are graded (Table 24.1).

Table 24.1. Categories of base soil

Sl. No.	Catagory	% finer than 75 micron
i	1	>85
ii	2	40-85
iii	3	15-39
iv	4	<15

To satisfy the permeability requirement of the filter material, D_{15} of the filter material should be equal to or higher than 5 times D_{15} of base soil and should not be less than 0.1 mm. Table 2 presents filtercriteria for different base material.

Table 24.2. Criteria for filters

Catagory	Base soil description and per cent finer than 75 micron	Filter criteria
1	Fine silt and clay, more than 85% finer	$D_{15}(F) \leq 9D_{15}(B) \geq 0.2$ mm
2	Sand, silt, clay and silty and clayey sand, 40-85% finer	$D_{15}(F) \leq 0.7$ mm
3	Silty and clayey sands and gravel, 15-39% finer	$4D_{15}(B) - 0.7$mm but should not be less than 0.7 mm
4	Sand and gravel, less than 15% finer	$D_{15}(F) \leq 4D_{15}(B)$

The configuration of the filter zones, however, will depend upon the type of embankment:

1. In a modified homogenous dam, the filter is generally placed as a blanket of sand and fine gravel on the downstream foundation area, extending from the cutoff/core trench boundary to the edge of the downstream toe and then taken to safe discharge by the toe drains.

2. In a zoned dam, the filter is placed between the core and the downstream shell zone. A longitudinal 'chimney' drain of gravel material collects the intercepted seepage flow and carries it to the base of the chimney. The transverse drains then conveys the water to the toe drains outside the embankment.

Design criteria has been given by Terzaghi

1. $\dfrac{\text{D15 of filter material}}{\text{D85 of base material}} = 5 \; to \; 40$ (24.

Provided that the filter material does not contain more than 5% material finer than 0.75mm.

2. $\dfrac{D_{15}\,of\,filter\,material}{D_{85}\,of\,base\,material} \leq 5$ if not satisfying the above case.   (24.2

3. $\dfrac{D_{85}\,of\,filter\,material}{Maximum\,opening\,of\,the\,pipe\,drain} \geq 2$ (24.3

 The particle size distribution curve of both i.e. filter and base material should be parallel.

24.4 Causes of Failure of Earthen Dams

Like most of engineering structures, earthen dams may fail due to faulty design, improper construction and poor maintenance practices, etc.

The various causes of failure may be classified as:

a) Hydraulic failure

b) Seepage failure

c) Structural failure

24.4.1 Hydraulic Failure

Hydraulic failure accounts for over 40% of earth dam failure and may be due to one or more of the following:

(i) By Overtopping: When free board of dam or capacity of spillway is insufficient, the flood water will pass over the dam and wash its downstream.
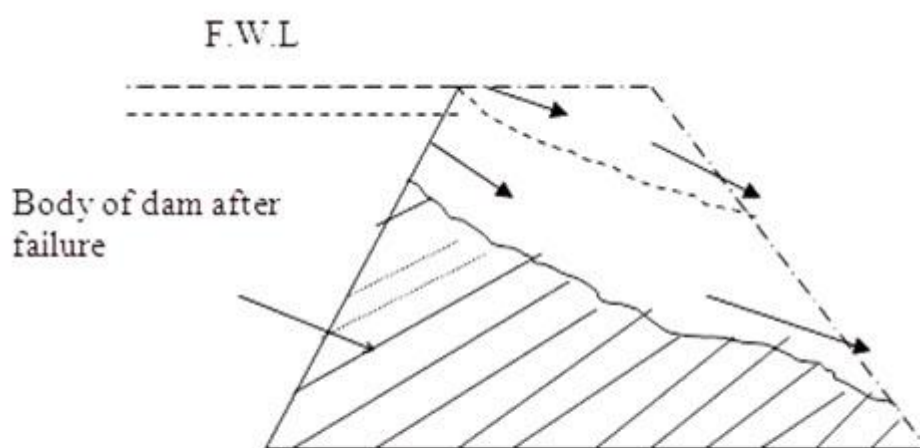

Fig. 24.6. Dam failure by overtopping.

Fig. 24.6. Dam failure by overtopping.

(ii) Erosion of Downstream Toe: The toe of the dam at the downstream side may be eroded due to heavy cross-current from spillway buckets ortail water.   When the toe of downstream is eroded, it will lead to failure of dam. This can be prevented by providing a riprap at downstream side up to a height above the tail water depth. Also, the side wall of the spillway should have sufficient height and length to prevent possibility of cross flow towards the earthen embankment.

(iii) Erosion of Upstream Surface: During winds, the waves developed near the top water surface may cut into the soil of upstream dam face which may cause slip of the upstream surface leading to failure. For preventing against such failure, the upstream face should be protected with stone pitching.

(iv) Erosion of Downstream Face by Gully Formation: During heavy rains, the flowing rain water over the downstream face can erode the surface, creating gullies, which could lead to failure. To prevent such failures, the dam surface should be properly maintained. All cuts\cracks should be filled on time and surface should be well grassed to reduce the effect of surface runoff. Berms could be provided at suitable heights and proper drainage should be maintained.

24.4.2 Seepage Failure

Seepage always occurs in the dams. If the magnitude is within design limits, it may not harm the stability of the dam. However, if seepage is concentrated or uncontrolled beyond limits, it will lead to failure of the dam. Following are some of the various types of seepage failure.

(i) Piping through Dam Body: Seepage starts through the poor soils in the body of the dam, small channels are formed which transport dam's material downstream. As more materials are transported downstream, the channels grow bigger and bigger which could lead to wash out of dam.

(ii) Piping through Foundation: When highly permeable cavities or strata of gravel or coarse sand are present in the dam foundation, it may lead to heavy seepage. The concentrated seepage at high rate will erode soil present in the foundation which will cause increased flow of water and soil. As a result, the dam will settle or sink leading to failure.

Fig. 24.7. Dam failure by piping through dam body.

(iii) Sloughing of Downstream Side of Dam: The process of failure due to sloughing starts when the downstream toe of the dam becomes saturated and starts getting eroded, causing small slump or slide of the dam. The small slide leaves a relative steep face, which also becomes saturated due to seepage and also slumps again and forms more unstable surface. The process of saturation and slumping continues, leading to failure of dam.

24.4.3 Structural Failure

About 25% of failure is attributed to structural failure, which is mainly due to shear failure causing slide along the slopes. The failure may be due to:

(i) Slide in Embankment: When the slopes of the embankments are too steep, the embankment may slide resulting in its failure. This occurs when there is a sudden drawdown or drastic decrease in the upstream water level due to some means, which is critical for the upstream side. Because of this, development of extremely high pore pressures takes place which decreases the shearing strength of the soil. The downstream side can slide especially when dam is full. In this case upstream embankment failure is not as serious as downstream failure.

(ii)Foundation Slide: When the foundation of an earthfill dam is composed of fine silt, clay, or similar soft soil, the whole dam may slide due to water thrust. If fissured rocks, such as soft clay, or shale exist below the foundation, the side thrust of the water pressure may shear the whole dam and cause its failure. In such failure the top of the dam gets cracked and the lower slopes moves outward and forms large mud waves near the dam heel.

(iii) Faulty Construction and Poor Maintenance: If during construction, the compaction of the embankment is not properly done, it may lead to failure.

(iv) Earthquake may Cause the Following Types of Failure to Earth fill Dams:

1. Cracks may develop in the core wall, causing leakages and piping failure.

2. Slow waves may set up due to shaking of reservoir bottom, and dam may fail due to overtopping.

3. Settlement of dam which may reduce freeboard causing failure by overtopping.

4. Sliding of natural hills causing damage to dam and its appurtenant structures.

5. Fault movement in the dam site reducing reservoir capacity and causing overtopping.

6. Shear slide of dam.

7. Failure of slope pitching.

Lesson. 25 Seepage Analysis

Seepage is the horizontal flow of water through the soil pore space. The seepage generally implied to the dominant horizontal flow in porous media. Seepage is the more commonly phenomenon in soil and water conservation structures and hence it assumes great significance in addressing the issue of stability of hydraulic structures mostly earthen type.

25.1 One Dimensional Flow

The flow through the saturated soil mass takes place from point of higher energy to point of lower energy as represented by head. The total head at any point in the soil mass consists of piezometric head, velocity head and elevation head. This can be represented by Bernoulli equation as below:

$$h = \frac{u}{\gamma_w} + \frac{v^2}{2g} + z \qquad (25.1)$$

Where the first term is pressure head, second term is velocity head and the third term is elevation head and also h is the total head, u is the pressure, v is the velocity, g is the acceleration due to gravity and gw is the unit weight of water.

 In soils the velocity of flow is very small and thus velocity head can be neglected in comparison to other heads. Eq. (25.1) becomes:

$$h = \frac{u}{\gamma_w} + z \qquad (25.2)$$

The relationship among pressure, elevation and total heads for the flow through soils is shown in Fig. 25.1. The pressure head at a point is the height of the vertical column of water in the peizometer installed at that point. The elevation head of a point is the vertical distance measured from any arbitrary horizontal reference datum plane to the point. Therefore, total head at points A and B, separated by distance L, is h_A and h_B, respectively.
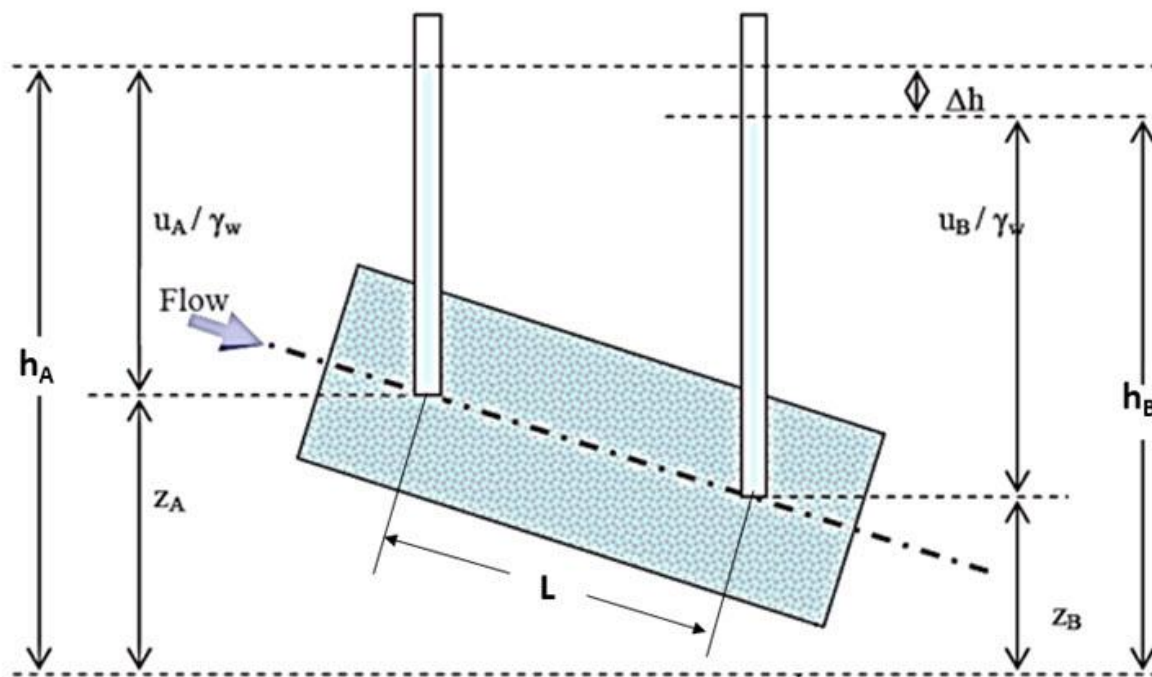
Fig. 25.1. Energy in water during flow through soil.

(Source: Punmia, B.C. 1994)

The loss of head between point andcan be given by

$$\Delta h = h_A - h_B \qquad (25.3)$$

The head loss Δh, can be expressed in a non-dimensional form as

$$i = \frac{\Delta h}{L} \qquad (25.4)$$

Where,I is the hydraulic gradient; and Lis the distance between points A and B, that is the length of flow over which the loss of head occurred.

25. 2 Darcy's Law

Darcy's law states that under the laminar flow conditions velocity of flow is directly proportional to hydraulic gradient and it can be expressed as:

$$v = Ki \qquad (25.5)$$

Where, v is the discharge velocity, which is the quantity of water flowing in unit time through a unit gross cross sectional area of soil at right angles to the direction of flow and k is the hydraulic conductivity.

The Darcy's equation v is discharge velocity and is based on gross cross sectional area. However the actual velocity of water (known as seepage velocity and represented by vs) through the void spaces is higher than v.

25.3 Two Dimensional Flow

In many practical situations, such as flow around a sheet pile wall, under masonry dams and through earth dams, flow is two dimensional, i.e., the velocity components in the horizontal and vertical directions vary from point within the cross section of soil mass. The two dimensional flow of water through soil is governed by Laplace's Equation. The assumption and its implications (in bracket) associated with Laplace's equation are presented below.

1. Darcy's law is valid (Flow is laminar)

2. The soil is completely saturated (Degree of saturation is 100%)

3. The soil is homogeneous (Coefficient of permeability is constant)

4. The soil is isotropic (Coefficient of permeability is same in all directions)

5. During flow, the volume of soil & water remains constant (No expansion or contraction)

6. The soil and water are incompressible (No volume change occurs).

For isotropic conditions, Laplace's Equation representing the two dimensional flow of water through soil can be expressed as:

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial z^2} = 0 \qquad (25.6)$$

This is the Laplace equation for **isotropic soil.** It says, the change of gradient in x-direction plus change in gradient in y-direction is zero. Graphical solution of Lapalce equation results flow net.

25.3.1 Flow Net

When Laplace equation is solved graphically the equation gives flow net consisting two sets of curves intersecting at right angles known as flow lines (or stream lines) and equipotential lines.

25.3.2 Characteristics of Flow Net

1. Flow lines or stream lines represent flow paths of particles of water

2. Flow lines and equipotential line are orthogonal to each other

3. The area between two flow lines is called a flow channel

4. The rate of flow in a flow channel is constant *(Δq)*

5. Flow cannot occur across flow lines

6. An equipotential line is a line joining points with the same head

7. The velocity of flow is normal to the equipotential line

8. The difference in head between two equipotential lines is called the potential drop or head loss *(Δh).*

9. A flow line cannot intersect another flow line.

10. An equipotential line cannot intersect another equipotential line.
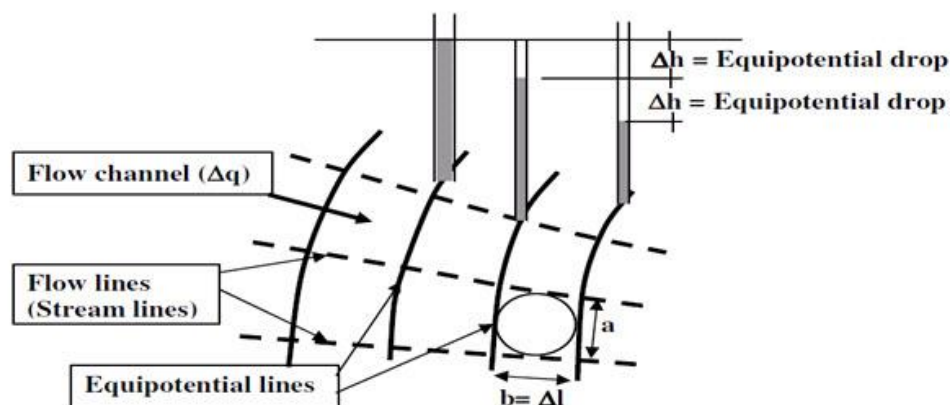
Fig. 25.2. Flow net and its characteristics. (Source: Punmia, B.C. 1994)

25.3.2 Quantity of Seepage

Let *b* and *l* are the width and length of the field respectively; h is the head drop between two equipotential lines; *Q* is the discharge passing through the flow channel of flow net; Δ*h* is the total head, causing the flow.

The seepage rate (*q*) through the dam can be computed from the flownet. According to the Darcy's law, the seepage water through a single flow channel is equal to:

$$\Delta q = KiA$$

Consider   unit   thickness   of   field

$$\Delta q = K\,\frac{\Delta h}{l}. \quad (b.1) \qquad\qquad (25.7)$$

If the field thickness is taken as unit and total number of potential drops in the entire flow net is N_d, then $\Delta h = h/N_d$; and Eq. 25.7 is written as,

$$\Delta q = K\,\frac{h}{N_d}.\frac{b}{l} \qquad\qquad (25.8)$$

Hence total discharge passing through the entire flow net is given by

$$q = \Sigma \Delta q = K\,\frac{h}{N_d}.\frac{b}{l}.N_f \qquad\qquad (25.9)$$

in which N_f is the total number of flow channels in the flow net. Since the field is in square in shape, hence $b = l$. Thus, Eq. (25.9) is reduced to:

$$q = K\,h.\frac{N_f}{N_d}. \qquad\qquad (25.10)$$

This expression can be used for computing the discharge rate passing through the flow net. It is applicable only to the isotropic soils.

Example 25.1:

For a homogeneous earth dam 52 m high and 2 m free board, a flow net was constructed and following results were obtained:

Number of potential drops = 25

Number of flow channels = 4

The dam has a horizontal filter of 40 m length at its downstream end. Calculate the discharge per metre length of the dam if the coefficient of permeability of the dam material is 3×10^{-3} cm/sec

Solution:

The dischrge per unit length is given by

$$q = kH\frac{N_f}{N_d}$$

Hence,

H = water depth = 52-2 = 50 m

$k = 3 \times 10^{-3}$ cm/sec = 3×10^{-5} m/sec

$N_f = 4$

$N_d = 25$

$\therefore$

$q = 3 \times 10^{-5} \times 50 \times \frac{4}{25} = 24 \times 10^{-5}$ m³/sec/m

= 0.00024 cumecs/metre length

Lesson 26. Phreatic Line in Earthen Dam

Phreatic line is the top flow line which separates saturated zone from unsaturated zone and below which positive hydrostatic pressure exists in the dam section. Along the phreatic line, the atmospheric pressure exists. The flow of water, below the phreatic line, reduces the effective weight of the soil, as a result shear strength of the soil is reduced due to generation of greater pore pressure.

26.1 Locating Phreatic Line

It has been noticed from experiments that the line of seepage assumes more or less the shape of a parabola. The hydraulic gradient i is equal to the slope of the free surface and is constant with depth (Dupuit's theory), the resulting solution of the phreatic surface is parabola. In some sections a little divergence from a regular parabola is required particularly at the surfaces of entry and discharge of the line of seepage. The properties of the regular parabola which are essential to obtain phreatic line are depicted in Fig. 26.1.
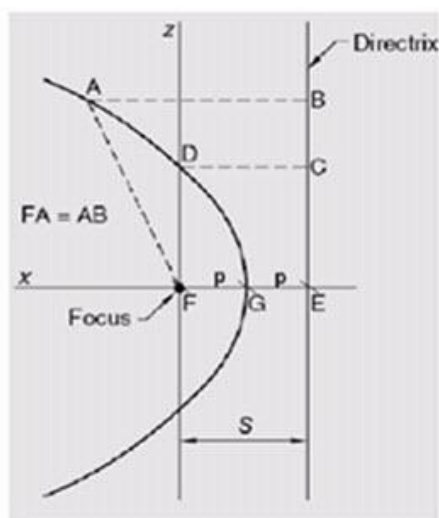
Every point on the parabola is equidistant from focus and directrix.

Therefore,

$$FA = AB$$

Also, $FG = GE = p = \dfrac{s}{2}$

Focus = (0, 0)

Any point A on the parabola is given by,

$$A = A(x, z)$$

$$x^2 + y^2 = (2p + x)^2$$

Or, $X = \dfrac{z^2 - 4p^2}{4p}$

Fig. 26.1. Geometrical properties of parabola. (Source: Punmia, 1994)

Determination of Phreatic Line:

Before drawing the flow net for the earth dam section, it is necessary to plot the phreatic line more accurately. Following methods are used:

1. Casagrande's graphical methods for determining phreatic line with and without horizontal drainage filter

2. Analytical methods for phreatic line for embankments with inclined discharge face without horizontal filter.

Only graphical methods are discussed here

26.1.1 Casagrande's Graphical Method (with filter)

Casagrande assumed that the phreatic line is a base parabola whose focus is at *F* which is the starting point of the filter *FE*. To determine the phreatic line following procedure is followed:
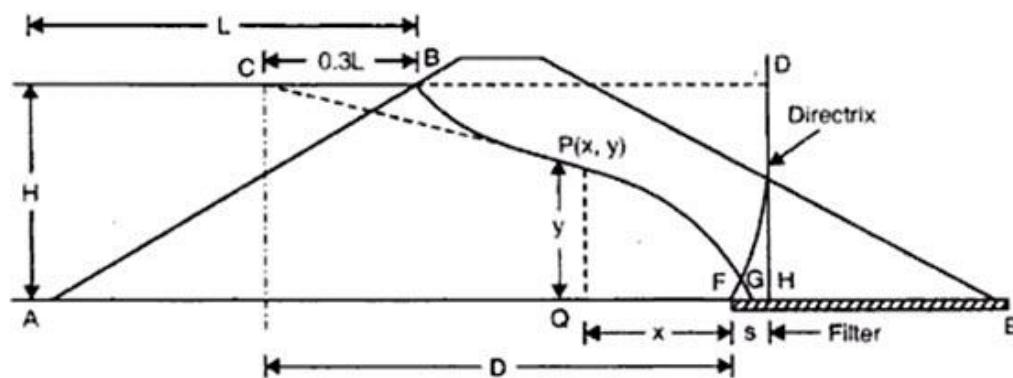

Fig. 26.2. Phreatic line with horizontal drainage filter.

(Source: Punmia, 1994)

1. *AB* is the upstream face. Let its horizontal projection be *L*. On the water surface, measure a distance *BC = 0.3 L*. then the point *C* is the starting point of base parabola (Fig. 26.2).

2. To locate the directrix of the parabola, use the principle that any point on the parabola is equidistant from the focus as well as from the directrix. Hence with point *C* as the centre and *CF* as the radius, draw an arc to cut the horizontal line through *CB* in *D*. Draw a vertical tangent to the curve *FD* at *D*. Evidently, *CD = CF*. hence the vertical line *DH* is the directrix.

3. The last point *G* on the parabola will lie midway between *F* and *H*.

4. To locate the intermediate points on the parabola use the principle that its distances from the focus and directrix must be equal. For example, to locate any point *P*, draw vertical line *QP* at any distance *x* from *F*. Measure *QH*. With *F* as the centre and *QH* as the radius, draw an arc to cut vertical line through *Q* in point *P*

5. Join all these points to get the base parabola. However, some correction is to be made at the entry point. The phreatic line must start from *B* and not from *C*. Hence the portion of the phreatic line at *B* is sketched free hand in such a way that it starts perpendicularly to *AB*, and meets the rest of the parabola tangentially without any kink. The base parabola should also meet the downstream filter perpendicularly at *G*.

In order to find the equation of this parabola, consider any point P on it, with co-ordinates (x, y) with respect to the focus F as origin (Fig.. 26.2).

From property of parabola we have

$$PF = QH$$

or $$\sqrt{x^2 + y^2} = QF + FH = x + s \qquad (26.1)$$

Where, s = FH = focal distance

From (26.1), $$x^2 + y^2 = x^2 + s^2 + 2xs$$

$\therefore$ $$x = \frac{y^2 - s^2}{2s}$$

And $$y^2 = 2xs + s^2,\text{ this is the equation of parabola}$$

$$s = \sqrt{x^2 + y^2} - x \qquad (26.2)$$

At C, $x = D$ and $y = H$,

$\therefore$ $$s = \sqrt{D^2 + H^2} - D \qquad (26.$$

Discharge through the body of dam: If q = discharge per unit length of dam

So we observe that through the vertical section PQ

$$q = kiA = k\frac{dy}{dx}(y \times 1) \qquad (26.4)$$

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

$$\therefore \frac{dy}{dx} = \frac{s}{(2xs + s^2)^{1/2}} \qquad (26$$

Substituting in (26.4), we get

$$q = k\frac{s}{(2xs+s^2)^{1/2}} \times (2xs+s^2)^{1/2} \qquad (26.6)$$

$$q = ks \qquad (26.7)$$

This is the simple expression for discharge q in terms of focal distances.

26.1.2 Casagrande Method for Embankment with Inclined Discharge Face (Without Filter)

In this case the base parabola *BLG* starts at *C*, which as before is at *0.3L*, cuts drawdown slope at *L* and goes beyond the limits of the dam to meet the base line extended at *G*. The focus of the parabola is at *F*(Fig. 26.3), the lowest point of the slope. However, in practice the phreatic line must emerge to meet the downstream slope tangentially at *N*. The portion *NF* is called discharge face which always remains wet. The portion *LN* is the correction *a* by which the parabola is to be shifted downwards.
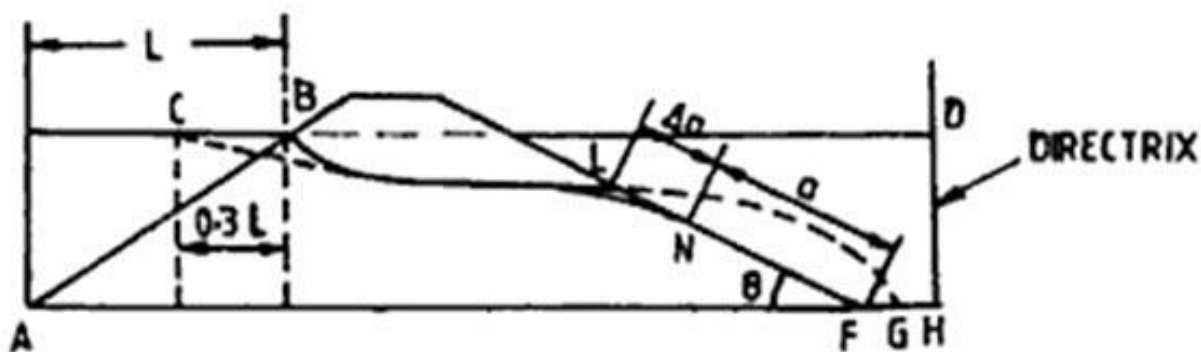
Fig. 26.3. Location of phreatic line (without filter).

(Source: Punmia, 1994)

The values of also varies in different exit situations like homogeneous sections, position of rock toe vertical or inclined and horizontal filter, which is shown in Fig.26.4.
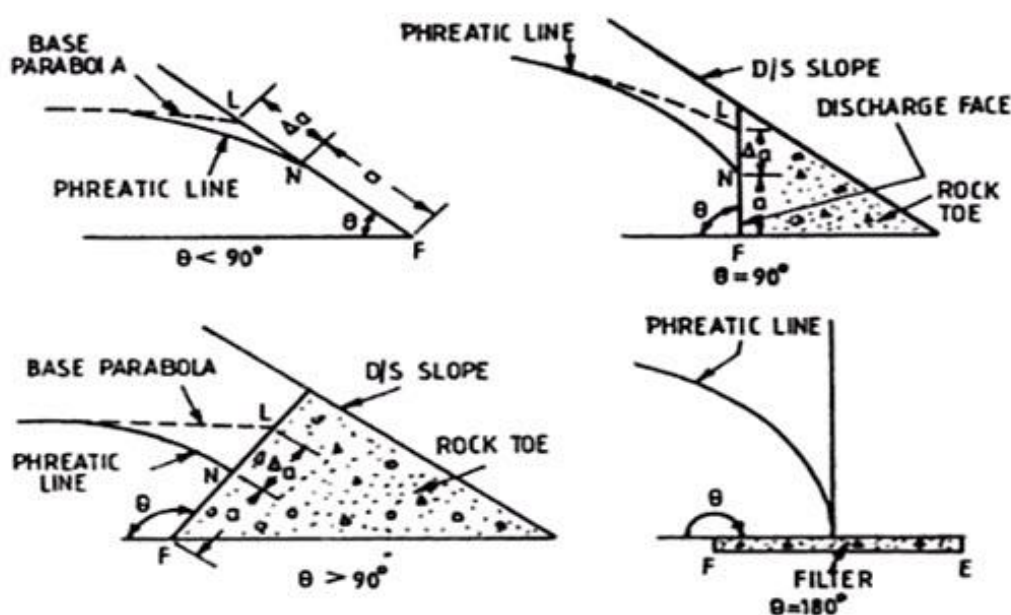
Fig. 26.4.Conditions for various slopes of the discharge face.

(Source- Punmia, 1994)

Casagrande developed the following relationship for the values of slope to determine a, from the known values of a which is shown in Fig.26.5.
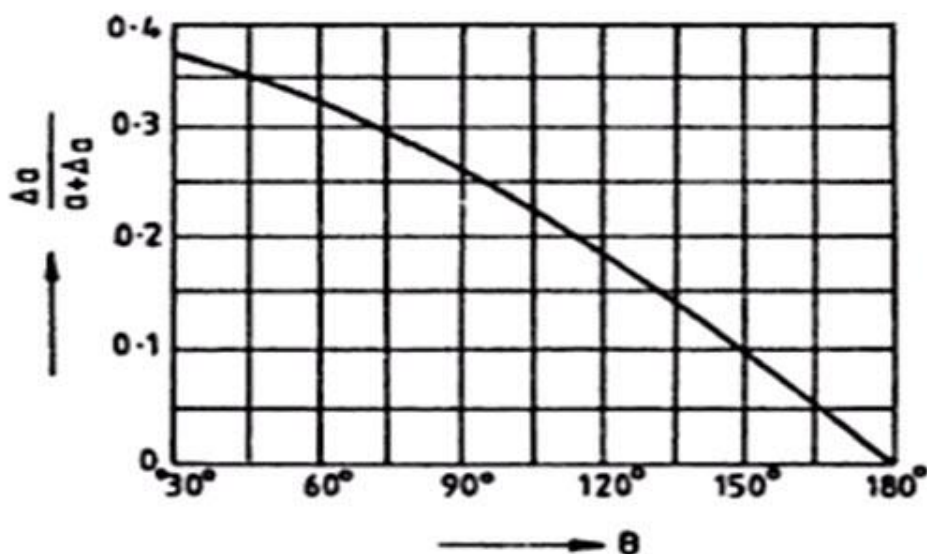
Fig. 26.5. Relationship between slope and $\frac{\Delta a}{a+\Delta a}$. (Source- Punmia, 1994)

Example 26.1:

For the earth dam of homogeneous section with a horinzontal filter as shown in Fig. 26.6, if the coefficient of permeability of the soil material used in the dam is 5 *10^{-4} cm/sec., find the seepage flow per unit length of the dam.

Solution:
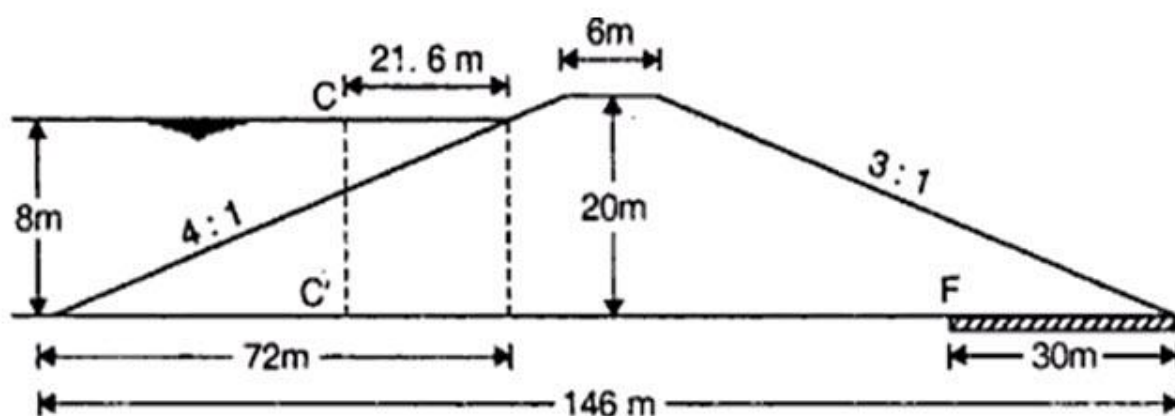
Fig. 26.6. Dimensions of earthen dam. (Source- Punmia, 1994)

Taking the focus as the origin, the equation of parabola is

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

The parabola cuts the reservoir water surface at C, such that

$$FC' = (140\text{-}\ 30\ \text{-}\ 72) + 21.6 = 65.6\ \text{m}$$

And $\qquad CC' = 18\ \text{m}$

$$\therefore s = \sqrt{65.6^2 + 18^2}\ \text{-}\ 65.6 = 2.42\ \text{m}$$

The equation of the parabola is, therfore,

$$y = \sqrt{2sx + s^2} = \sqrt{4.84x + 5.85}$$

the coordinate of the parabola are tabulated below:

x	0	10	20	30	40	50	60	65.6

y	2.42	7.36	10.11	12.30	14.1	15.7	17.25	18

Keywords: Seepage, Phreatic line, Earthen dam

Lesson 27. Farm Ponds

Farm ponds have a significantly role in rainfed farming system where annual rainfall is more than 500 mm. It helps in mitigating the ill effect of rainfall variability as it stores water from rainfall excess and provides for utilization during prolonged dry spells by means of supplemental/protective irrigation. It also helps in pre-sowing irrigation of *rabi* crop. In high rainfall semi-arid regions of India, farm pond can be used for multiple uses such as protective/supplemental irrigation, fish culture, duck farming integrated with poultry.

27.1 Types of Farm Ponds

Broadly farm ponds can be categorized into two type i.e. embankment type and excavated or dugout type.

27.2 Embankment Type

These type farm ponds are constructed across the stream or water course and consist of an earthen dam. Dimension of embankment are determined based on the required storage. These ponds are suitable for areas having gentle to moderately steep slope and also where stream valleys are sufficiently depressed to permit a maximum storage volume with least earth work. Given the Indian farming system, this type of pond is constructed largely at common land resources as it requires substantial land under submergence.

27.2.1 Capacity of the Ponds

The capacity of the pond is calculated using trapezoidal or Simpson's rule. In trapezoidal rule, the volume of storage (δV) between two successive contours is given as: $\delta V = \frac{h}{2}(A_u + A_l)$, in which his contour interval and and are area bounded by upper and lower contour respectively.

In Simpson rule which is more accurate than trapezoidal method, the volume is estimated as:

$$V = \frac{h}{3}\left[\begin{array}{l} Twice\ the\ area\ of\ Odd\ contours\ + \\ 4\ times\ the\ area\ of\ even\ contours\ + \\ Area\ of\ the\ first\ and\ last\ contours \end{array}\right] \qquad (27.1)$$

The design aspect of earthen embankment has been discussed in lessons 23 and 24. Like any hydraulic structures, the embankment design also includes hydrologic design, hydraulic design and structural design.

Example 27.1: Design of Embankment Type Farm Pond
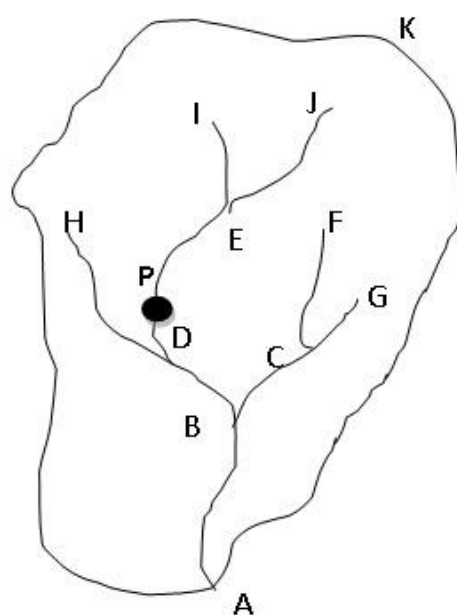
Let's assume to construct an embankment type pond in the watershed shown in Fig. 27.1 at location P. The detailed contours are presented in fig. 27.2. The embankment proposed to have maximum water level at 216 m. The dotted line represents the natural stream IE and JEP of this figure. Pond receives runoff during monsoon rain only. The catchment area is 65 ha. Following data are available.

1. Average annual monsoon rainfall: 750 mm
2. The site condition is such that the 20% of average annual monsoon rainfall is received by the pond as runoff.
3. Rainfall intensity for 25 years return period is 90 mm/hr and daily rainfall is 300 mm.
4. The average sedimentation @ 40t/ha/annum during first 5 years followed by 20, 10, 5 and 2 t/ha/annum in every succeeding 5 years.
5. The soil contains silt and clay with low compressibility.
6. Channel length is 1.2 km, and slope is 2%.

Fig. 27.1. Watershed Map.
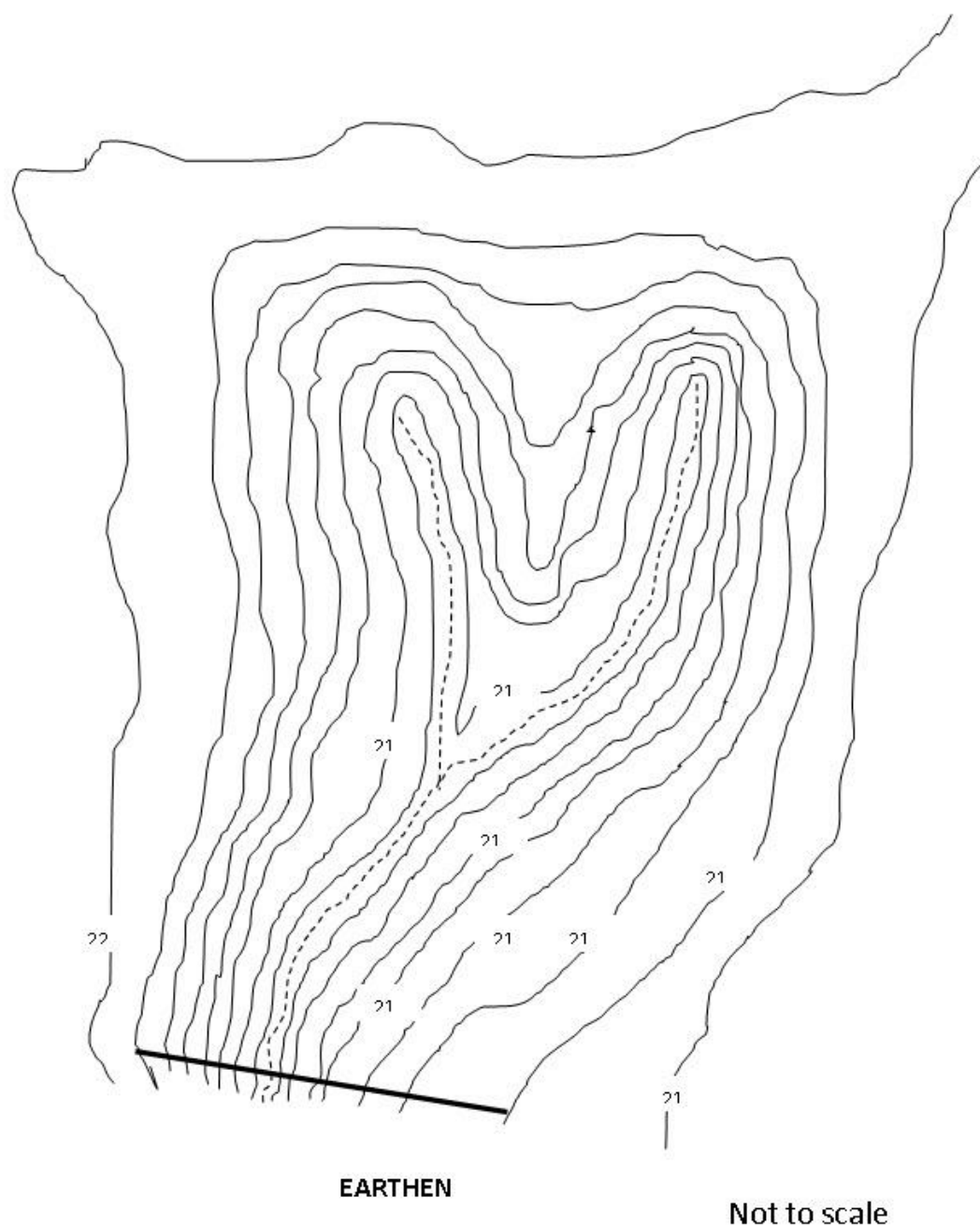
Fig. 27.2. Contour map of the area.

The storage capacity at different heights of the embankment is presented in Table 27.1.

Table 27.1 Storage capacity at different height of embankment

Contour level	Contour Interval	Maximum Water Depth	Area Bounded by (Ha)		Incremental Volume, Using Trapezoidal Method (ha-m)	Cumulative volume (ha-m)
			Upper contour, A_u	Lower Contour, A_l		
211	1	0	0.4	0	0.20	0.20
212	1	1	1.5	0.4	0.95	1.15
213	1	2	2.6	1.5	2.05	3.20
214	1	3	3.7	2.6	3.15	6.35
215	1	4	4.3	3.7	4.00	10.35
216	1	5	5.6	4.3	4.95	15.30
217	1	6	6.5	5.6	6.05	21.35

Like drop structures, here also the pond is designed under heading of hydrologic design, hydraulic design and structural design.

A. Hydrologic design

(i) Water Yield Estimation

Average annual monsoon rainfall =750 mm. Expected runoff volume or water yield from 65 ha catchment considering 20% of the rainfall as the runoff

$$= \frac{65 \times 750 \times 20}{1000 \times 100} = 9.75 \text{ ha-m}$$

(ii)   Estimation of Peak Discharge

Rational formula, $Q = \dfrac{CIA}{360}$

Time of concentration to determine, I

$$Tc = 0.01947 K^{0.77} ; \quad K = \sqrt{\dfrac{L^3}{H}} = \sqrt{\dfrac{1200^3}{24}} = 8485$$

And $Tc = 0.01947(8485)^{0.77} = 20.62$ min say 21 min.

From chart (relation of one hour rainfall intensity to intensities at oth duration), I for 21 minutes = 160mm/hr

Thus the peak discharge (assuming runoff coefficient, C, as 0.3)

$$Q = \dfrac{0.2 \times 160 \times 65}{360} = 5.77 \text{cumec.}$$

(B) Hydraulic Design

(i) Storage Capacity

Total storage capacity = live storage+dead storage

Live storage: 9.75 ha-m as determined from hydrologic design

Dead storage:

First 5 years @40 t/ha/annum = 200 t/ha

5-10 years @ 20 t/ha/annum = 100 t/ha

10-15 years @10 t/ha/annum = 50 t/ha

15-20 years @5 t/ha/annum = 25 t/ha

20-25 years @ 210 t/ha/annum = 1050 t/ha

Total = 385 t/ha

Total sediment yield during 25 years from 65 ha catchment

= 25 X 385 = 9625 ton

Assuming the bulk density of the sediment as 1.5 g/cc,

1 ha-m of sediment = 15000 tonnes

Thus dead storage = 9625/15000 = 0.64 ha-m.

Thus total storage capacity required = 9.75+0.64 = 10.39 ha-m.

From storage capacity table, the dead storage of 0.64 corresponds to 212 m contour level which will the top level of dead storage. The total capacity of 10.91 ha-m corresponds to 215.2 m contour. Thus the net height of the embankment at maximum water level will be 215.2 - 211 = 4.2 m. in which maximum water depth would be 3.2 meter.

(iii) Height of the Embankment

Height of water upto full level = 215.2 – 211.0 = 4.2 m

Height of free board (15%) = 15% of 4.2 m = 0.63 m.

Wave height for a fetch of 1.5 km,

$$h_w = 0.014(D_m)^{0.5}$$

= 0.542m

Since free board is more than wave height, free board will be considered.

Height of embankment will be 4.2+0.63 = 4.83. Add 5% settlement allowance thus height will be 4.83+5% = 5.07 m.

Location of principal spillway = at maximum water level = 215.2 m level.

Location of emergency spillway = 211 + 5.07 = 216.07 m level.

Thus the temporary storage between emergency spillway level and principal storage level will be

16.5 ha-m (at 216.07 m level) – 10.91(at 215.2 m level) = 5.59 ha-m

(ii) Design of Principal Spillway

24-hour maximum rainfall at 5 percent probability level is 300 mm with 30%

resulted into runoff. Thus expected storm runoff = $\frac{65 \times 300 \times 30}{1000 \times 100}$ = 5.85 ha-m. This principal spillway are mostly in the form of drop inlet spillway. Thus standard RCC pipes are selected from the manufacturer and laid at proper grade such that the condition of pipe running full could be obtained.

Designing drop inlet spillway (principal spillway)

The design discharge for drop inlet spillway is determined by the following relationship (Singh et al. 1990). Procedure is already discussed in case of drop inlet spillway design earlier. However, for the sake of completion of design it is also given here.

$$\frac{V_s}{V_r} = 1 - 2\frac{Q_o}{Q_i} + 1.8\frac{Q_o^2}{Q_i} - 0.8\frac{Q_o^3}{Q_i}$$

Where, V_s = volume of temporary storage (ha-m)

V_r = volume of runoff (ha-m)

Q_o = Required principal wpillway discharge (cumec) and

Q_i = Peak flow from design storm (cumec)

The above polynomial equation are solved for different $\frac{V_s}{V_r}$ and $\frac{Q_o}{Q_i}$ and given in Table 27.2 for quick solution. For intermittent values of $\frac{V_s}{V_r}$ and $\frac{Q_o}{Q_i}$, do the interpolation.

The above polynomial equation are solved for different $;\frac{V_s}{V_r}$ and $\frac{Q_o}{Q_i}$ given in Table 27.2 for quick solution. For intermittent values of $;\frac{V_s}{V_r}$ and $\frac{Q_o}{Q_i}$, do the interpolation.

Table 27.2. Estimating principal spillway discharge allowing for temporary storage (for watershed of less than 100 ha)

$;\dfrac{V_s}{V_r}$	$\dfrac{Q_o}{Q_i}$,									
	0.00	**0.01**	**0.02**	**0.03**	**0.04**	**0.05**	**0.06**	**0.07**	**0.08**	**0.09**
0.0	1.00	0.99	0.98	0.96	0.95	0.94	0.92	0.91	0.90	0.88
0.1	0.87	0.85	0.84	0.82	0.81	0.79	0.78	0.76	0.74	0.73
0.2	0.72	0.70	0.68	0.67	0.65	0.64	0.62	0.61	0.60	0.58
0.3	0.57	0.55	0.54	052	0.51	0.50	0.49	0.47	0.46	0.45
0.4	0.44	0.43	0.42	0.41	0.40	0.39	0.38	0.37	0.36	0.35
0.5	0.34	0.33	0.32	0.31	0.30	0.29	0.28	0.27	0.27	026
0.6	0.25	0.24	0.23	0.23	0.22	0.21	0.20	0.20	0.19	0.18
0.7	0.18	0.17	0.16	0.15	0.15	0.14	0.14	0.13	0.12	0.12
0.8	0.11	0.11	0.10	0.09	0.09	0.08	0.08	0.07	0.07	0.06
0.9	0.05	0.05	0.04	0.04	0.04	0.03	0.02	0.02	0.01	0.01

Here V_s = 5.59 ha-m, V_r = 5.85 ha-m; Q_i = 6 cumec.

Thus $;\dfrac{V_s}{V_r}$ = 0.96, for this ratio, $\dfrac{Q_o}{Q_i}$ = 0.02 (from above table)

And so Q_o = 0.02 X 6 = 0.12 cumec

Thus select the suitable RCC pipe which is capable to handle 120lps discharge.

(iii) Design of Emergency Spillway

Expected peak discharge = 6.0 cumec

Discharge through principal spillway = 0.12cumec

So, design discharge for emergency spillway = 6.0 – 0.12 = 5.88cumec.

So at the level 216.07 m, chute spillway should be constructed for the capacity of 5.88cumec (see design of chute spillway).

(c)Structural Design

Top width = H/5+1.5 = 5.07/5+1.5 = 2.01 m say 2 m.

U/s and D/s slope = 3:1 and 2.5:1

Provide berm of 2.0 meter at the level of 215.2 m i.e. at the level of principal spillway to install pipe inlet.

27.3 Excavated or Dugout Ponds

These types of farm ponds are small dug out structures with well-defined shape and size. These structures have provision for inlet and outlet. Farm ponds are constructed at lower portion of the farm and generally stored water is used for irrigation. In some places farm ponds are used for recharging groundwater. However, for recharging groundwater, high capacity structures located in the highly permeable soil are more suitable. These structures are also called percolation tank (Reddy et al. 2012).

Dugout ponds are constructed by excavating the soil from the ground and the excavated soil is used to make embankment around the pond. The pond could either be fed by surface runoff or groundwater wherever aquifers are available. The depth and size of pond depend upon the volume of water to be stored. This type of pond is more featured in individual farm. Dug-out ponds can be grouped into the following four categories:

1. Excavated or dug out ponds

2. Surface ponds

3. Spring or creek fed ponds and

4. Off stream storage ponds

27.3.1 Excavated or Dugout Ponds

Excavated pond site should be chosen based on general slope of the field. Various locations of dug-pond are illustrated in Fig. 27.1 based on the prevailing land slope. If slope is towards left bottom corner of the field, a form pond must be constructed in the left corner of the plot and similarly for slope towards right bottom corner. If the slope is towards the bottom of the field, pond can be constructed at either side corner with proper field channel at the bottom of the field connecting to the inlet of the structure. If the farm area has multiple slopes in different direction, pond should be located in a portion of the area where maximum water can be drained into the structure.
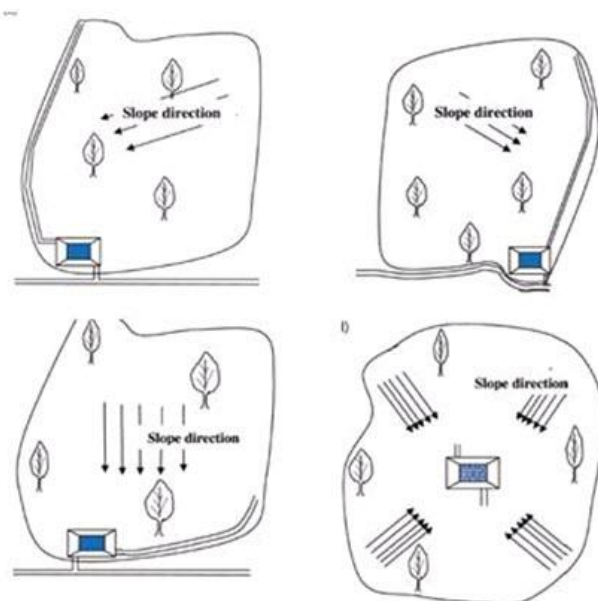
Fig. 27.1. Planning and selection of site for dugout farm pond.

(Source: Reddy et al. 2012)

27.3.2 Surface Pond

When the surface runoff from a farm area is collected into a local depression or the lowest portion of the farm such that the excavation is minimum, this type of pond is called surface pond (Fig. 27.2). Surface pond is possible in the farm area with undulating topography. This type of pond does not require a formal inlet provision but it should have formal outlet provision.
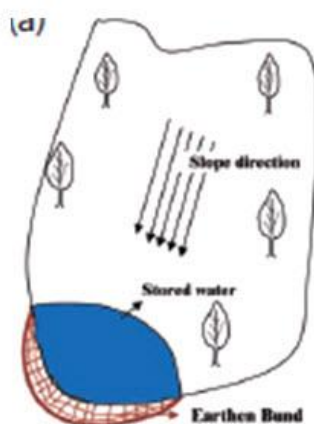
Fig. 27.4. Planning and selection of site for surface pond.

(Source: Reddy et al., 2012)

27.3.3 Spring or Creek Fed Ponds

This type of pond is generally constructed at the foothills of the hilly catchments. After the soil saturation occurred due to excess rainfall, the subsurface flow of the catchment oozes up to the surface at the foothills as base flow (Fig. 27.3).

Fig. 27.5. Planning and selection of site for spring or creek fed pond.
(Source: Reddy et al. 2012)

27.3.4 Off Stream Storage Ponds

This type of pond should be adopted where construction of embankment across the natural channel is not feasible or economically viable. Off-stream storage ponds collect water from the stream using diversion (Fig. 27.4). In hilly catchment

where, storage volume upstream of embankment of dam is not sufficient and unable to sustain the high flow velocity, these types of structure can be adopted.
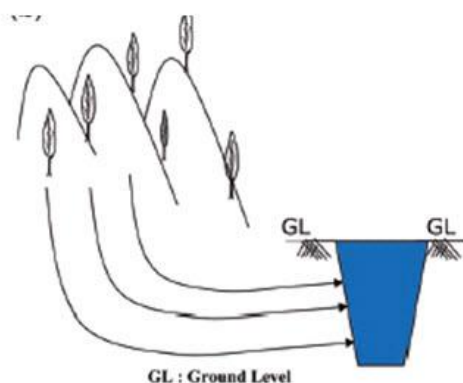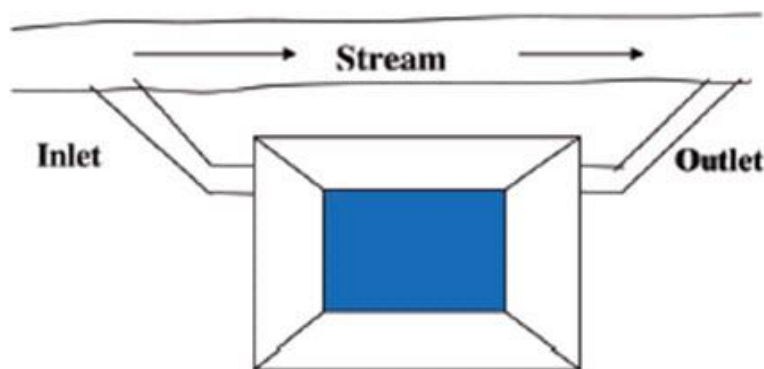

Fig. 27.6. Planning and selection of site for off-stream storage ponds.

(Source: Reddy et al. 2012)

27.4 Site Selection for Dugout Type Farm Pond

The site selection for a single or multiple pond system requires careful planning considering several variables. These include local soil condition, topography, drainage capacity, infiltration and rainfall pattern and its distribution. A suitable pond site should be selected to ensure long-term success. Generally such area should be selected where a limited amount of excavation is required to contain, or hold back, a large volume of water. However, the specific selection criteria for different type of ponds are explained in detail by Reddy et al.(2012).

Soil type

Soil type is also playing a major role in deciding the site for farm ponds. The soils having stones and boulders should be avoided for digging farm ponds particularly when pond is to be remained unlined. The soil profile must be investigated and location with good soil depth, free of stones, low Ph, EC and ground water level can be selected for farm ponds.

Topography

The proposed pond construction should be based on the topographic features of the site such that higher excavation to storage ratio could be achieved to attain the economy. Depending upon the capacity of the farm pond, contour survey should be conducted. However for smaller catchment (less than 5 ha), a reconnaissance survey is sufficient. Fig. 27.5 presents a sample contour map and respective location of ponds (Reddy et al. 2012).
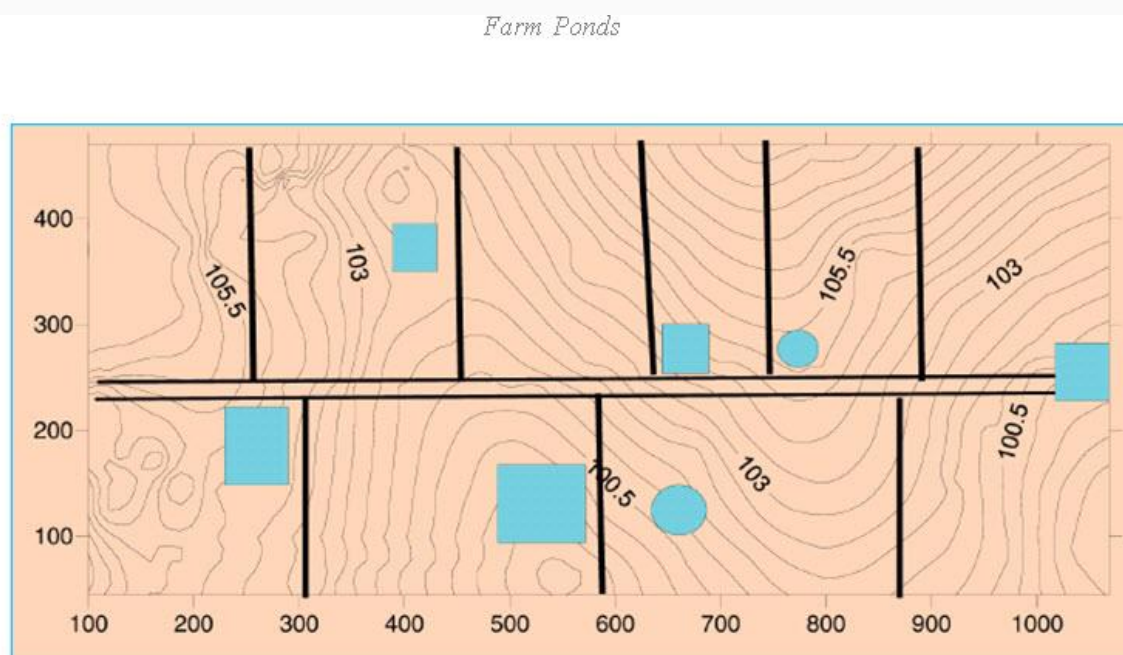
Fig. 27.7. Contour map to select site for farm ponds.

(Source: Reddy et al.2012)

27.5 Design Approach for Dugout Farm Ponds

Rainfall is the most important hydrological parameters for design of farm ponds. The design approach includes determination of design rainfall, probability analysis, surface runoff or water yield, rainfall-runoff relationship of the catchment, shape of the pond and utilization.

Design rainfall

The design rainfall refers to the quantum of rain required during the cropping season in catchment area to provide sufficient runoff to meet the crop water requirement. The design rainfall is very delicate preposition: if actual rainfall is less than design rainfall in cropping season, moisture stress will occur but when actual rainfall is more than design rainfall, surplus runoff may cause damage to the structure. Thus proper probability analysis of the historical rainfall event of the area should be carried out.

Probability analysis

Long term rainfall data, as long as possible but not less than 20 years, is required for carrying out probability analysis. Plotting position method is used for carrying probability analysis. Steps for probability analysis are given below:

1. Arrange the annual/season rainfall data in descending order and rank them such that maximum value gets rank 1 and minimum value with maximum rank (rank should be given in perfect digits such 1,2,3,... etc).

2. If two or more rainfall events are equal, then assign same rank to all such events.

3. Calculate the probability according to any plotting position methods such as Weibul.

4. Plot the probability vs rainfall on normal probability paper.

5. Determine the rainfall for 67 % and 75 % from the plotting curves.

Table 27.3 presents average seasonal rainfall and their expected design rainfall at different probabilities.

Table 27.3. Average seasonal rainfall and design rainfall at 67 and 75% probabilities for different locations of India

S. No	Centres	Average seasonal rainfall, mm	Probabilities	
			67%	75%
Low rainfall area: (average annual rainfall : 0-500mm)				
1	Anantapur	353.6	263	224.8
2	Hisar	345.4	299.1	238.2
Medium rainfall area ((average annual rainfall : 500-1000mm)				
1	Akola	671.7	598.8	534.3
2	Anand	815.5	643.1	549.5
3	Bangaluru	523.5	408.7	388.3
4	Bijapur	403.9	336.6	293.3
5	Faizabad	791.6	667.1	597.7
6	Kanpur	714.5	639.4	611.1

7	Kovilpatti	145.8	109.8	86.2
8	Ludhiana	593.1	409.5	349.0
9	Parbhani	834.1	618.0	545.0
10	Rakhdiansar	878.4	786.4	753.0
11	Solapur	533.0	451.8	408.5
12	Udaipur	604.5	381.0	350.0
High rainfall areas (average annual rainfall : >1000 mm)				
1	Bhubaneshwar	1116.1	1042.1	959.2
2	Dapoli	3259.5	2962.4	2894.3
3	Jabalpur	1174.5	1050.0	793.8
4	Jorhat	1173.3	1065.4	1034.0
5	Mohanpur	1050.2	881.4	865.0
6	Palampur	1521.3	1310.7	1222.5
7	Ranchi	1260.7	1040.7	990.0
8	Ranichauri	771.8	698.3	673.1
9	Raipur	951.1	795.4	734.9
10	Samastipur	993.6	830.0	798.0

Using the design rainfall from Table 27.1, runoff volume for the selected catchment area can be determined using curve number method, which is described earlier.

The further details on dugout farm pond design and construction are provided in the next lesson.

Keywords: Farm pond, design rainfall, probability analysis

Exercise:

1. Prepare detailed drawing of the embankment using the parameters obtained from solved designed example in this chapter.

2. Estimate the capacity of farm pond for the following information:

Sr No.	Contour Value	Area Enclosed (m²)
1	150	200
2	151	250
3	152	300
4	153	307
5	154	405

Lesson 28. Farm Pond Design and Construction

In the previous lectures, general description on farm pond is covered along with detailed design of embankment type of farm pond. In this lecture, design and construction details of dugout farm pond is discussed. A well design serves as a valuable asset for integrated farming and should be properly constructed after thorough design and planning.

28.1 Designing Dugout Farm Ponds

The design of farm ponds includes determination of capacity, its location, utilization plan and shape and size.

28.1.1 Catchment and Cultivable Area Ratio

This ratio is particularly important to determine the optimal size of pond based on the available catchment and the utilization of stored water in the cultivable area. The ratio between catchment (A_{ca}) and cultivable area (A_{cu}) also influenced by runoff coefficient and runoff efficiency factor. This ratio indicates the amount of cultivated area in the total catchment area. This ratio is given as (Critchley et al., 1991):

$$\frac{CWR - DR}{DR \times RC \times RE} = \frac{A_{ca}}{A_{ca}} \qquad (28.1)$$

Where CWR is the crop water requirement, DR is the design rainfall, RC is the runoff coefficient, RE is runoff efficiency factor, A_{ca} is the catchment area and Acu is the cultivable area.

Crop water requirement depend upon the type of crop and location specific climate and can be calculated from standard equations and models.

Runoff coefficient is the runoff to rainfall ratio. It is amount of surface flowover the ground. The coefficient is governed by slope, soil type, vegetation, AMC, and rainfall characteristics (intensity, frequency and duration). Typically this coefficient ranges between 10 to 50%. Larger catchment has lower runoff coefficient than smaller catchment (1-5 ha). The runoff coefficients are site specific and should be determined locally.

Efficiency factors address the issue of uneven distribution of the runoff water within the field. The uneven distribution occurs due to differential infiltration and percolation and surface depression. When the cultivated area is leveled and smooth, the efficiency is higher. Generally micro- catchment systems have higher

efficiency. The efficiency can be determined using the graphical relationship between runoff efficiency and catchment area (Fig. 28.1).
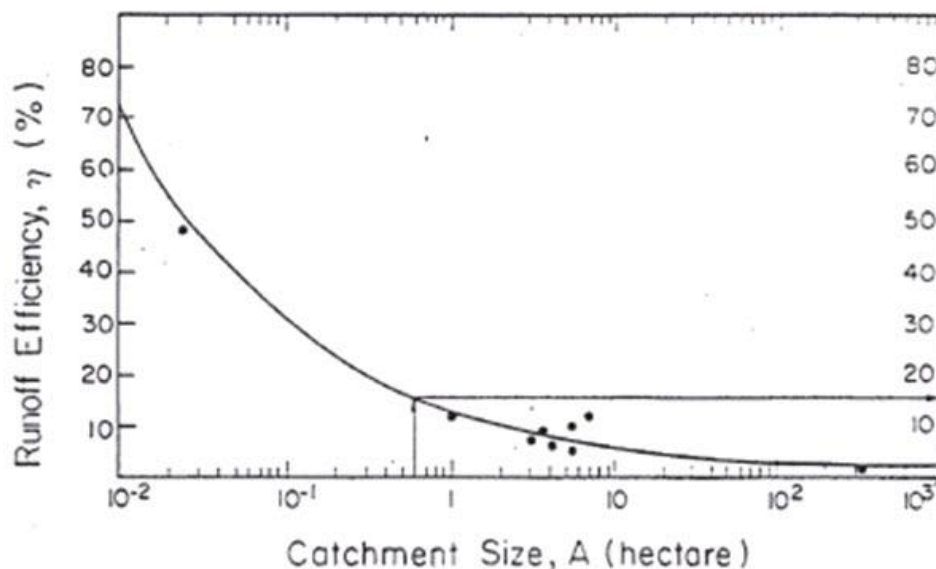
Fig. 28.1. Relationship between runoff efficiency and catchment area.

(Source: Ben Asher, 1998)

The basic principle of design is to produce a satisfactory functional structure at a minimum cost. To minimize the cost, the pond should be designed for its maximum utilization considering the three strategies for irrigation particularly in rainfed agriculture (Reddy et al., 2012).

1. Meeting the crop water requirement of growing season.

2. Meeting water requirement of critical irrigation (CRI) during the critical stages of crop growth.

3. Meeting water requirement in cropping system approach e.g. irrigation during critical stages of *kharif* crop plus water requirement of *Rabi* crop such as vegetables.

The graph provided in Fig. 22a and 2b can be used to determine the expected runoff volume to determine the net capacity of the of the proposed farm pond. This graph is however developed for seasonal rainfall of 375 mm at 75 % probability and 425 mm at 67 % probability. Similar graphs can be generated for different rainfall as well. Depending on the efficiency factor, runoff coefficient and catchment area, runoff volume is estimated and accordingly shape, size and dimension of the ponds are designed.

28.1.2 Shape of the Pond

Generally, the farm ponds are excavated in the shape of rectangular, square or inverted cone with circular cross section. Though, inverted cone and other curved shape ponds are difficult to construct but many times it provides higher ratio of volume to wetted surface that increases the net storage after moderating for seepage loss.

28.1.3 Dimension of Ponds

The optimum dimension of farm ponds should be based on the hydrological consideration and catchment area. The farm pond with respect to small catchment may not have sufficient runoff to be filled and risk of being dried during dry spell. Similarly pond with large catchment would require large water control structure and would be difficult to manage. Generally the area under dug out farm pond should not be more than 10% of the farm catchment.
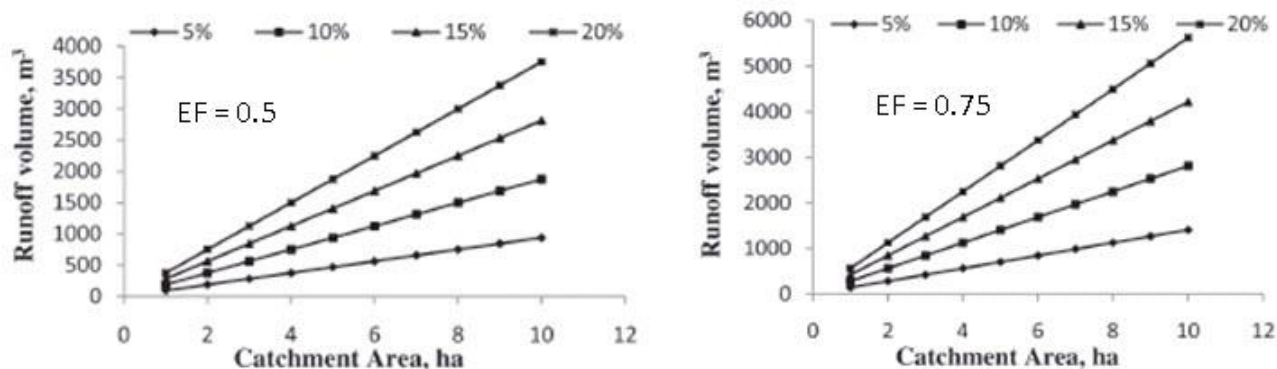
Fig. 28.2a. Expected runoff volume at design seasonal rainfall of 375mm (75 % probability) for different runoff coefficient, efficiency factor (EF) and catchment area. (Source: Reddy et al., 2012)
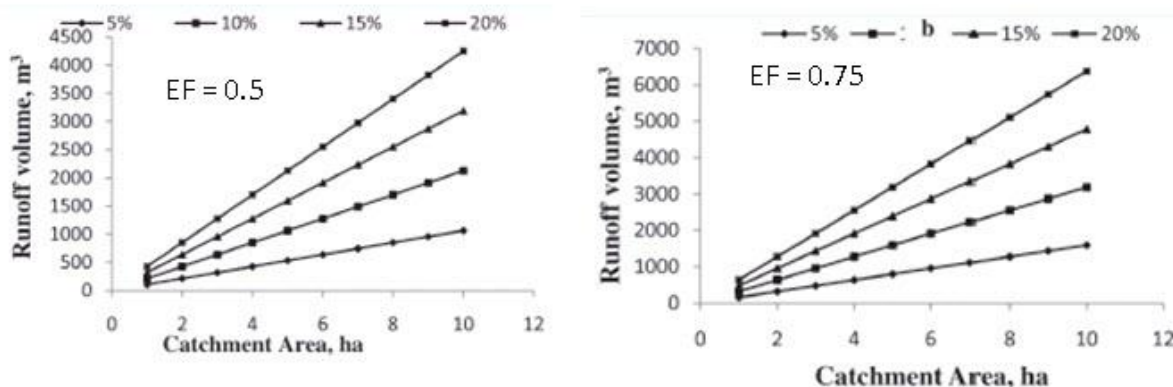
Fig. 28.2b. Expected runoff volume at design seasonal rainfall of 425mm (67 % probability) for different runoff coefficient, efficiency factor (EF) and catchment area. (Source: Reddy et al., 2012)

28.1.4 Depth and Side Slope

The depth of the farm pond is decided by considering soil depth, soil type and equipment used in excavation. Though, the evaporation loss component can be minimized by increasing the depth but, from practical point of view the ideal depth is limited to 3 to 3.5 meter. Any depth beyond 4.0 meter will be uneconomical if human labour is employed in excavation.

Based on experience, it is observed that the side slope of the pond should not be steeper than the natural angle of repose of the excavated material. Table 28.1 presents the recommended side slope of farm pond for different soil as suggested by Critchley et al. (2011). Another factor while considering the side slope is duration of standing of water in the farm pond. For higher duration, flatter side slope is recommended to avoid the slippage due to saturation particularly for unlined pond.

Table 28.1. Recommended side slope

Soil type	Slope (horizontal:vertical)
Clay	1:1 to 2:1
Clay loam	1.5:1 to 2:1
Sandy loam	2:1 to 2.5:1

Sandy	3:1

The volume of the farm pond can be determined using eq. (28.2) as follows:

$$V = \frac{A + 4B + C}{6} \times D \tag{28.2}$$

Where, is volume of exacavation (m³); is the area of excavation at ground surface (m²), is the area of exacavation at mid-depth point (D/2) (m²); C is the area at the bottom of the pond (m²) and D is the design depth (m).

28.1.5 Rectangular/Square Section Farm Pond
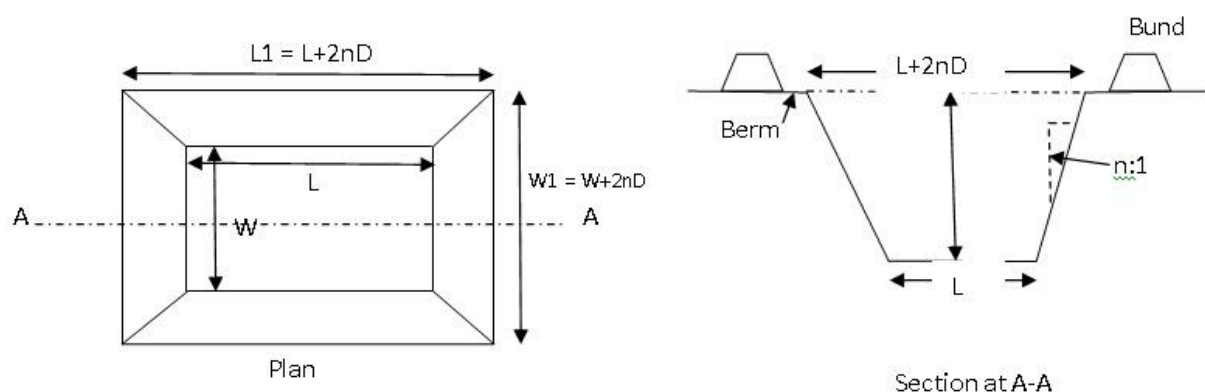
Fig. 28.3. Cross-section of dugout farm pond.

Using eq.28.2, the bottom dimension for rectangular pond is derived as

$$L = \left(\frac{0.5}{C} \right) \times \left(\sqrt{n^2 D^2 (1 + C)^2 - 4C \left\{ (4/3) n^2 D^2 - \frac{V}{D} \right\}} - nD(1 + C) \right) \tag{28.}$$

Where, $C = \frac{L}{W}$, the ratio between length and width at the bottom. For square section, C will be equal to 1, Eq (28.3) would be simplified as

$$L = W = \sqrt{n^2 D^2 - 4\left\{\left(\frac{4}{3}\right)n^2 D^2 - \frac{V}{D}\right\}} - nD \qquad (28.4)$$

Example 28.1: A dugout farm pond of capacity 500 m³ is to be constructed in clay loam soil. The soil profile is such that the depth is limited to 3 m and site permits length width ratio as 1. Determine the dimension of the dugout farm ponds.

Solution:

V = 500 m³, Side slope for clay loam (n) = 2:1 (Table 28.1), D = 3 m

UseEq (28.4)to determine bottom length and width as

$$L = W = \sqrt{2^2 3^2 - 4\left\{\left(\frac{4}{3}\right)2^2 3^2 - \frac{500}{3}\right\}} - 2 \times 3$$

$$L = W = \sqrt{36 - 4\{1.33 \times 36 - 166.67\}} - 6 = 16.6 \text{ m say } 17\text{m}$$

The dimension of top width would be equal to base width plus nD i.e. 17+23 = 23 m.

Similarly from Eq. (28.2), the dimension for inverted cone can be derived as

$$d_1 = \sqrt{\left\{\left(\frac{4V}{\pi D}\right) - \left(\frac{1}{3}\right)n^2 D^2\right\}} - nD \qquad (28.5)$$

Where, is the diameter at the bottom of the pond and top diameter, can be computed as

$$d_2 = (d_1 + 2nD) \qquad (28.6)$$

Example 28.2: From above exercise, design an inverted cone type farm pond.

Solution:

Use Eq. (28.5)to determine the diameter of bottom, d_1 as

$$d_1 = \sqrt{\left\{\left(4 \times 500 \Big/ \pi \times 3\right) - \left(\frac{1}{3}\right)2^2 3^2\right\} - 2 \times 3}$$

$$d_1 = \sqrt{\{(212.31) - 12\}} - 6 = 8.15\text{m, say } 8.5 \text{ m.}$$

Using Eq. (28.6), the top diameter will be 14.5 m.

28.2 Economical Cross Section of Farm Ponds

Water lost from dugout ponds are mainly due to evaporation and seepage. The evaporation loss could be minimized by minimizing the surface area and volume ratio, whereas seepage losses could be minimized by reducing the ratio of wetted area and volume. Theoretically the volume of stored water per unit wetted area increases with the increase in depth of the water in ponds but at the same time increases the cost of pond lining. Thus optimization of various pond dimensions must be done for seepage reduction and cost of pond lining.

28.2.1 Optimization of Various Pond Dimensions

Mishra and Sharma (1994) suggested a design methodology to optimize the different pond dimensions based on designed storage volume and prevailing soil type. The optimization include following computational steps.

(a) Average cross-sectional area (

$$A_{av} = \frac{LW + (L + 2nD)(W + 2nD)}{2} \qquad (28.7)$$

(b) Wetted area

$$A_w = LW + 2D\sqrt{1 + n^2} \, (L + 2nD + W) \qquad (28.8)$$

(c) Storage volume, V

$$V = \frac{LW + (L + 2nD)(W + 2nD)}{2} \times D \qquad (28.9)$$

Where, L and W are the two sides of the pond bottom, D is depth of the pond and n is the side slope (horizontal: vertical::n:1)

Now expressing C=L/W i.e. W=CL, Eqs. 28.7, 28.8 and 28.9 transform as

$$A_{av} = \frac{CL^2 + (L+2nD)(CL+2nD)}{2} \tag{28.10}$$

$$A_w = CL^2 + 2D\sqrt{1+n^2}\,(L + 2nD + CL) \tag{28.11}$$

$$V = \frac{CL^2 + (L+2nD)(CL+2nD)}{2} \times D \tag{28.12}$$

Now converting Eq.(28.12) to quadratic form and solving for L,

$$L = \frac{-Dn(1+C) + \sqrt{[Dn(1+C)^2 - 4C(2D^2n^2 - V/D)]}}{2C} \tag{28.13}$$

The above Eqs 28.10 to 28.13 become simple for a pond with square bottom, i.e. C=1 means L=W

$$A_{av} = \frac{L^2 + (L+2nD)^2}{2} \tag{28.14}$$

$$A_w = L^2 + 4D\sqrt{1+n^2}\,(L + nD) \tag{28.15}$$

$$V = \frac{L^2 + (L+2nD)^2}{2} \times D \tag{28.16}$$

Theoretically, for a particular storage volume, the surface area of the pond is the lowest when the depth is increased to a point where the bottom surface area becomes zero. For this conditions,

$$D = \sqrt[3]{V/2n^2} \qquad\qquad (28.17)$$

$$A_{av} = V/D = 2D^2n^2 \qquad\qquad (28.18)$$

$$A_w = 4D^2n\sqrt{1+n^2} \qquad\qquad (28.19)$$

Example 28.3:

A dugout farm pond of capacity 500 m³ is to be constructed in clay loam soil. Determine the optimal dimension of a farm pond.

Solution:

V = 500 m³, n = 2

Using(28.17), the optimal depth, D will be

$$D = \sqrt[3]{500/8} = 3.96 \text{ m say, 4 m}$$

$$A_{av} = 2 \times 4^2 \times 2^2 = 128 \text{ m}^2$$

$$A_w = 4 \times 4^2 \times 2\sqrt{1+2^2} = 286.22 \text{ m}^2$$

Compare this result with the exercise 28.1. In earlier exercise, the average cross sectional area was computed as 409 m² for 3 m depth whereas in this case the average cross sectional area is only 128 m² for 4 m depth. Just by increasing depth by one meter (if site conditions allow), one can significantly reduce evaporation loss (because surface area reduced) and seepage loss (because wetted area reduced). Moreover, the cost of lining could be minimized significantly by just increasing the depth by 1 m.

28.2.1.1 Inlet and Outlet Channel

The inlet channel to the farm pond are constructed in such a way that, all the surface runoff from the catchment area to concentrate at the channel and drain into the pond. The inlet is designed as chute spillway for conveying the runoff in to the pond. While laying the inlet channel, the safe velocity of the water must be ensured to avoid channel erosion. Grasses can be grown in the channel for further protection.

Stone pitching is widely adopted to reduce the erosion from the inlet channel. The entry section is designed as a rectangular broad crested weir to allow 0.3 to 0.5 m depth of flow over 1 to 1.5 m width (Fig. 28.3).
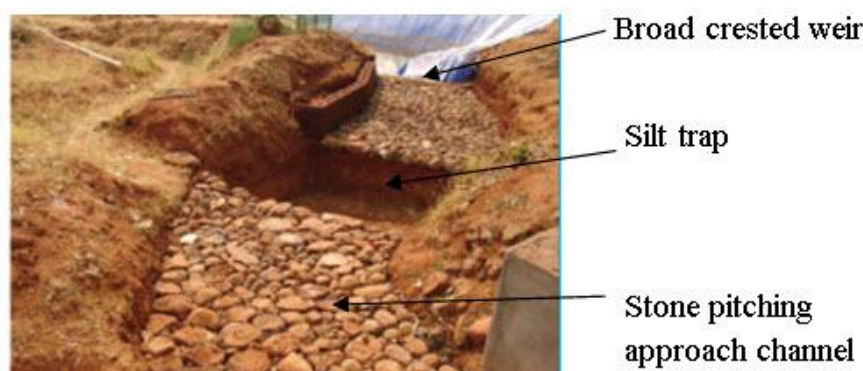
Fig. 28.3. A view of inlet channel and stone pitching.

(Source: Reddy et al. 2012)

28.2.1.2 Outlet/Waste Weir

The out let or waste weir is designed to remove the surplus runoff over and above the designed capacity of the pond. The outlet is generally located at the lower end of the pond. It should be thickly vegetated with grass to reduce the erosion. The flow through outlet should be maintained at non-erosive velocity to minimize the outlet erosion. The outlet is kept at slightly lower elevation (up to 30 cm) as compare to the inlet elevation to avoid the back water.The discharge capacity of the outlet can be assumed to be half as that of the inlet capacity as peak rate of runoff

28.2.1.3 Lining of Farm Pond

The various losses including seepage, percolation and evaporation account for as high as 60% of the gross storage. Seepage and percolation losses can be controlled using pond lining techniques. The seepage losse under different soil class is presented in Table 28.2.

Table 28.2. Seepage losses in different soil class.

Sl. No	Type of soil	Water loss through seepage (cumec/million m² of wetted surface)	Drop in depth per day (cm)
1	Heavy clay loam	1.21	10.36
2	Medium clay loam	1.96	16.84
3	Sandy clay loam	2.86	24.61
4	Sandy loam	5.12	44.03
5	Loose sandy soil	6.03	51.80
6	Porous gravelly soil	10.54	90.65

Several lining techniques are available that uses the material like bricks and stone with cement and mortar. Various other techniques are also used for seepage control. Some of these are Asphalic material, paddy husk with cow dung, cement with soil mixture, fly ash mixture and bentonite treatment of pond surface. Performance of these methods varies but none of them can control seepage losses completely. The LDPE (low density poly-ethylene) film lined pond with brick overlaying as anchorage and safety from external damage to the plastic sheet can last up to 15 to 20 years and is considered as one of the permanent solution.

The advantages of LDPE film lined ponds are listed as

1. Reduction in water losses through seepage and percolation to the extent of 95%

2. Availability of water for a longer period can be ensured.

3. This type of lining provides scope to construct farm pond in porous soil as well where water retention is low

Method for LDPE lining of farm pond

For farm pond lining, the quality and strength of LDPE film depends on the capacity of pond and depth. For higher capacity say 200-1000 m^3 with 3 to 3.5 m depth, minimum 500 micron thickness and 300-350 gsm (gram/m^2) film is required. However, for smaller pond (capacity less than 200 m^3 and depth is limited to 2.0 m) adopted in the prevailing conditions of hill and mountain agro-ecosystem of Himalayan region, 200-250 micron with 200 gsm LDPE film is sufficient for lining. The plastic film other than LDPE namely HDPE (High density polyethylene) and geo-membrane (reinforced HDPE) are also used for pond lining. The HDPE and geo-membrane film are used for lining bigger ponds where capacity is more than 1000m^3. The properties of different polyethylene (PE) films are presented in Table 28.3.

Table 28.3. Properties of different PF films

Property	Test method (ASTM coded)	Unit	Values		
			LDPE film 500µ	HDPE film 500µ	Geo-membrane 500µ
Material density	1505	g/cc	0.92	0.94	0.94
Breaking strength	6093, 638 type IV	N/mm	12	14	28
Puncture resistance	4833	N	120	176	491
Tear resistance	1004	N	50	73	120
Bursting strength	751	Kg/cm^2	4	4.3	8.5
Hydro static registance	751	Kg/cm^2	2	3	6
Impact failure load		Gram force, Gf	555	585	>2000

Key words: dug out farm pond, seepage loss

Module 5. Trenching and Diversion Structures

Lesson 29. Trenching

29.1 Trenching

Trenching is one of the major engineering measures for erosion control in non-arable lands and is mainly aimed to slope stabilization and drainage line treatment. The area with steep slope e.g. hilly region, are prone to soil erosion due to lack of vegetative cover and accelerated transportation of soil. The hilly area exhibits the characteristics of "high rainfall, quick drainage" that provides little retention time to the runoff to infiltrate into the soil profile. The overland flow velocity is often surpassing the safe limit to cause soil erosion from surface. The trenches constructed in these regions of address the problem of soil conservation to act as flow barrier (restricting the flow velocity within the safe limit from soil erosion point of view) and facilitating in-situ water conservation for establishment of vegetation.

29.2 Types of Trenching

The trenches are constructed in different geometrical configurations namely contour trenching, continuous trenching, staggered trenching and in line trenching. The selection of trenches depends on the site characteristics and rainfall intensity. The schematic diagram of different types of trenches are shown below.

29.2.1 Continuous Trenches

These are series of broad channel or embankments constructed at suitable spacing along the graded contours of gentle slopes. They are suitable for the areas having high annual rainfall. This type of trenches are long trenches (as long as 50 m) and have fixed interval (15 -30 meter). In this type of trenches, horizontal intervals are fixed but vertical intervals are varied. These types of trench are easy to construct but there is a problem of inconsistent deposition of soil in the trenches due to varied vertical interval and hence it is difficult to maintain. The cross-sectional area of the trench is usually kept as 0.25 m^2 and depth should not be higher than 0.5 m. The cross section of trenches are kept as square. The life of trench is considered as 5 years. Desiltation is required in every alternate year under normal rainfall condition and every year under excess rainfall conditions.
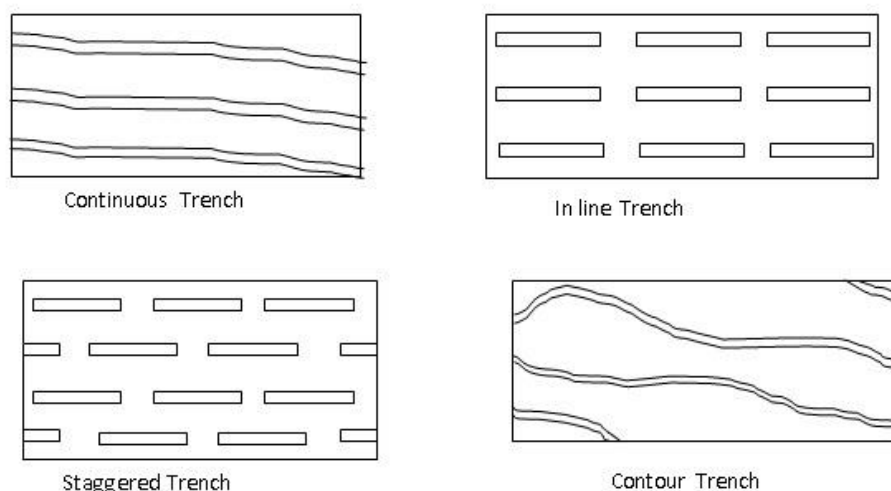
Fig. 29.1. Types of trenches. (Source: Murthy and Jha, 2009)

29.2.2 In-line Trench

This type of trench addresses the problem of inconsistent deposition of soil. These trenches are maximum 5 meter long and cross section is similar to continuous trenches. The gap between two in-line trenches should not be more than 2 meters as shown below. This type of trenches has the limitation that it fails to collect runoff flowing between the gaps of two trenches.
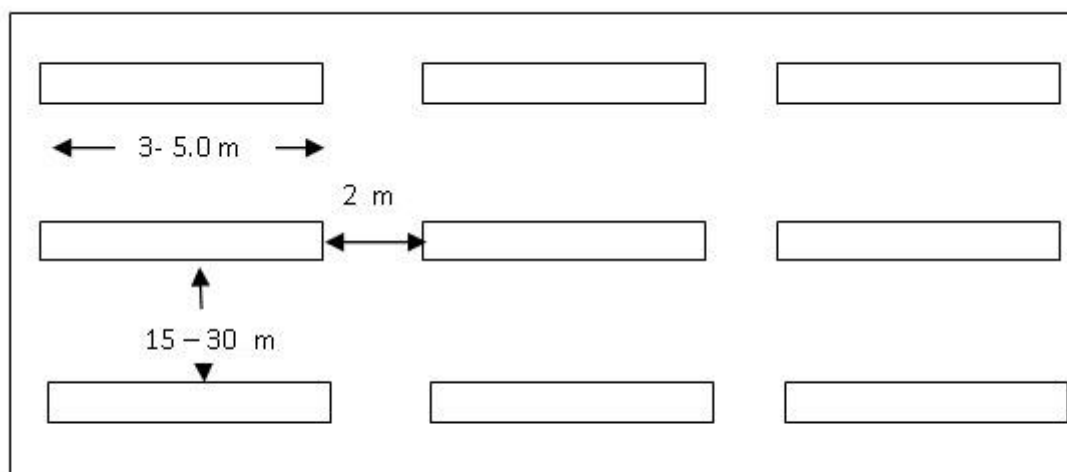
Fig. 2. Schematic diagram of In-line trenches.

29.2.3 Staggered Trenches

The staggered trenching involves the excavation of trenches of shorter length in a row along the contour with interspace between them. These trenches are arranged in straight line (staggered form). Suitable vertical intervals between the rows are

restricted to impound the runoff without overflow. In the alternate row, the trenches are located directly below one another. The trenches in successive rows are thus staggered, with the trenches in the upper row and the interspace in the lower row being directly below each other. The length of the trench and the interspace between the trenches in the same row should be suitably designed such that no long unprotected or uninterrupted slope to cause unexpected runoff or erosion. As the trenches are not continuous, no vertical disposal drain is excavated. The cross sectional area of these trenches should be designed to collect the runoff expected from intense storms at recurrence intervals of 5-10 years.
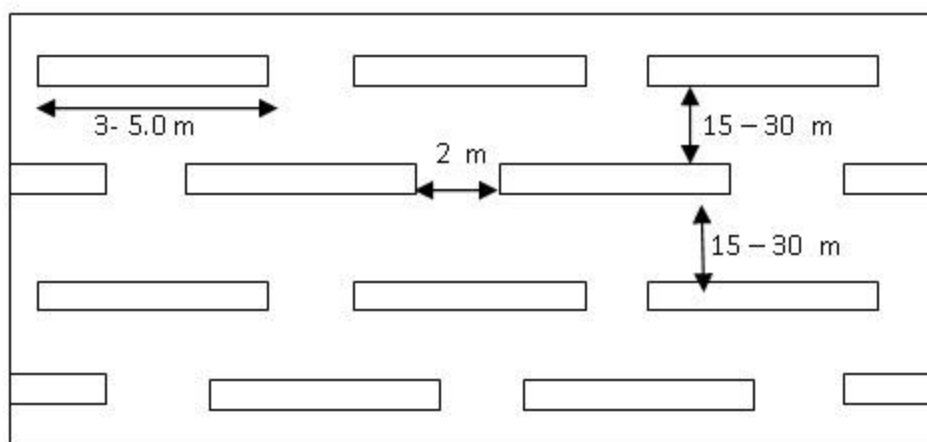
Fig. 29.3. Schematic diagram of staggered trenches.

29.2.3.1 Key Features of Staggered Trenches are:

- These trenches are short (3-5 m long) and arranged in row along the contour with inter-space between them.

- The vertical interval between two successive trenches is decided on the basis of expected runoff from the area, above without overflow.

- In staggered sequenced, the alternate rows of trench are located directly below one another.

- The length of row and slope between them fixed based on the concept that, there should not be greater length of unprotected or uninterrupted slope to cause unexpected runoff and erosion.

- The cross sectional area of these trenches is designed to collect the runoff from an intense storm during 5 to 10 year's return period.

29.2.4 Contour Trenching

The contour trenches are similar to continuous trench except it follows the prevailing contour of the area. In contour trenches, the horizontal interval are

varying unlike continuous trenches. However, vertical interval is fixed usually kept at 3-5 meters. This type of trenches are little difficult to construct but has the advantages over the continuous trenches in terms of consistent soil deposition and hence easy maintenance and less risk of failure.

Contour Trenches are a simple, low cost method of checking the velocity of runoff in ridge area of any watershed. A contour trenching is excavated trench along a uniform level across the slope of land in the top portion of catchment. Trench along contour line increases retention of runoff for a longer period within the trench and significant reduction in soil erosion. Contour trenching should be carried out perpendicular to the slope. Contour trenches also can be laid as staggered. Contour trenches are widely adopted in soil conservation works.

29.3 Site Specification for Trenching

Following points must be considered while selecting site for trenching.

- If the slope of the ridge area is 25% or more, the best form of treatment is the planting of grasses, shrubs and trees. This is because for contour trenches to be effective on such high slopes, they will have to be constructed at very close intervals, which could end up causing more soil erosion due to excessive digging.

- On the other hand, if the slopes are less than 10%, then contour trenches are not considered as the best measure. This is because in such a situation, in comparison to contour trenches, contour bunds are a more effective means of checking runoff and soil erosion. In a contour bund, water not only stops in the excavated portion but also against the bund. However, on very high slopes, it is not possible to make contour bunds since there is a great danger of the bunds breaking.

- Thus, given the above considerations, contour trenches are the most appropriate where the slope of the ridge area lies between 10-25%. It is an ideal treatment for the non-agricultural land.

- If the depth of soil is less than 20cm as well black soil, trenching should be avoided.

29.4 Design of Trenches

The design of trench system involves the trench cross-section, spacing of trenches so as to collect the runoff generated from catchment area. The runoff majorly depends on the hydrologic conditions of the catchment including vegetation, soil and land slope. Following steps are described in designing trenches (Sharda et al 2007).

(a) **Determination of Direct Runoff Volume:** The trenches are designed such that it should hold the runoff generated from storm of 6-hr rainfall intensity with 5 years recurrence interval. The dimension of trenches are influenced by soil type as well. In cases of coarse textured and fine textured soils, the trenches may be designed to hold 60-70% of the runoff and 50% respectively.

(b) Determining Cross-Sectional Area and Volume: The cross section of the trench can be square, rectangle, trapezoidal or triangle in shape. The choice of shape depends on the soil type and prevailing land slope. Square and rectangular shaped trenches are constructed in relatively flat land with nominal slope whereas in sloppy lands, trapezoidal or triangular trenches are constructed. The cut and fill should be kept equal in trench construction and while computing volume, only cut portion is accounted for.

Trenches are excavated at suitable vertical intervals (depending upon slope of land and soil type). The distance between the two successive trenches depends on the volume and the velocity of runoff they are expected to handle. This in turn depends on:

- The quantum of rainfall: The greater the rainfall, the lesser the interval.

- The permeability of the soil: The more permeable the soil, the greater shall be the interval

- The slope of the land: The greater the slope, the lesser the interval

As the volume and velocity of runoff increase due to any reasons given above, the trenches should become more closely spaced. In practice, one must fix the maximum and minimum horizontal interval between the two successive trenches in a way, which is depicted as follows:

- On high slopes, the trenches should be close to each other but should not closer than 10 m.

- On low slopes, the trenches should be far from each other but should not more than 30 m.

(c) Determination of Spacing: Spacing, in trenching point of view, are expressed as horizontal and vertical interval (HI and VI). The definition sketch describing various terminology used in trenching is presented below.
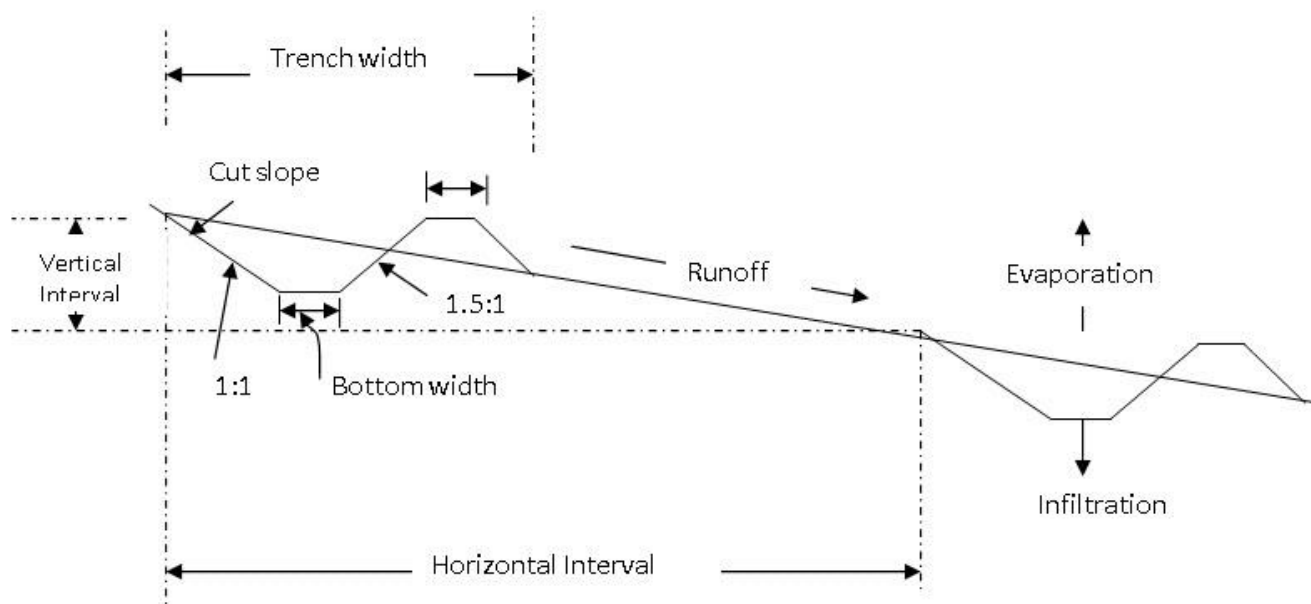
Fig. 29.4. Definition sketch of contour trenches.

(Source: Training manual on Soil Conservation and Watershed management (Vol II), CSWCRTI, Dehradun)

Theoretically, the relationship between horizontal spacing of trenches, runoff depth and trench dimension are described as following:

$$HI = \frac{Cross - sectional\ Area\ of\ Trench}{Expected\ runoff} = \frac{A}{Q}$$

Assuming the trench of rectangular cross section,

In practice, the vertical interval between two successive trenching rows can be measured by:

i) For the vertical interval between two rows.

$$V.I = \frac{Slope\ (\%)}{C}$$

Where,

V.I. = Vertical interval (VI), which is defined as the difference in elevation between the two similar points on two consecutive bunds.

C = 12 for the medium rainfall.

ii)
$$V.I = 0.305 \times \left[\left(\frac{s}{a} \right) + b \right]$$

Where VI = Vertical Interval in meter, S = Slope in percent, a and b = constants specific to particular region. For soil with good infiltration rates values of 'a' and 'b' are respectively taken as 3 and 2 whereas for soils of low infiltration rates the corresponding value are 4 and 2, respectively.

- *Size And Depth of Trench*: Size of trench depends on the depth of soil and also on some other factors of watershed. Based on the above parameters different sizes of trenches have been planned. The sizes of trenches are as follows:

Sr. No	Width (m)	Depth (m)
1	0.50	0.50
2	0.45	0.45
3	0.60	0.45
4	0.60	0.30
5	0.30	0.30

In general, the most popular size has been used in the many watersheds is as follows (Fig. 5):

Depth: 50 cm

Width: 50 cm

Berm: The mud excavated is piled up 20 cm away, downstream of the trench.

This gap between the trench and mud is called the berm. This distance is essential so that this mud does not fill up the trench again.

Plantation: If grass has to be planted along the trenches, then the excavated mud should be piled up in a 10cm. high rectangular layer. If trees have to be planted, they should be planted either in the space after the trench or on either side of the trench.

The general cross section of the trench is given below.
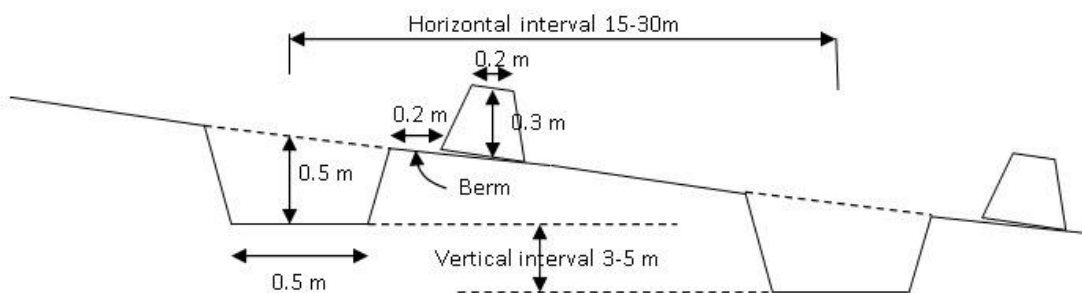
Fig. 29.5. Trenching with berm.

29.5. Planning Contour Trenches

The planning and designing of contour trenches involves 2 factors,

1. Amount of runoff water to be stored
2. Sufficient length required to store design runoff

Amount of runoff to be stored

It is estimated as the portion of rainfall that results into the runoff and given by the following equation.

$$Q = C \times R \times A$$

Where,

Q is runoff (m³), C is runoff coefficient, R rainfall (m) and A is the area (m²) of the ridge from which the runoff is received

Thus for collecting 70% of runoff in the field

$$Q_1 = Q \times \frac{70}{100}$$

Length required to store design runoff

Cross sectional area of contour trenches is always 0.5 m x 0.5 m = 0.25 m². The trenches are filled more than once during the rainy period. However, this will depend upon soil permeability and rainfall intensity as well. Usually trenches are designed for at least three filling.

Therefore to store 70% of runoff in an area of one hectare, we divided Q_1 by cross-section area (At) of the contour trench multiplied by the number of refills (f).

$$l = \frac{Q_1}{A_t \times f}$$

Laying of trenches

This involves finding out the distance between successive rows of contour trenches and the number of such rows, we have values of area (A) and length of contour trenches (L).

The distance between two successive rows of contour trenches will be,

$$d = \frac{A}{L} \; metres$$

We have to start layout of trenches from the longest section of the ridge area. In order to decide number of rows of contour trenches (N) required, we have to divide the longest section of ridge area (L_1) by the distance between successive rows of contour trenches (d).

$$N = \frac{L_1}{d}$$

Example 1: Design a continuous contour trench in area of 25 hectares in Bhubaneswar. The runoff coefficient of this area is 0.4. The daily rainfall is 100 mm and only 75% of the runoff has to be stored in the contour trench. Assume the trench gets 2 refills in a day. The longest section of the ridge area is 2500 m. Make a plan for continuous contour trenching .

Solution:

A= 25 ha = 250000 sq.m

R = 100 mm = 0.1 m

Therefore, **Q** = 0.4 x 0.1 x 250000 = **10000 cum**

Only 75% of run of we want to catch,

Q_1= 10000 x 75/100 **= 7500 cum**

Length of contour trench required,

We knew that usually contour trenches are 0.5 m x 0.5 m, so the cross sectional area is 0.25 sq.m and its given that refills 2 per day.

L = 7500/0.25 x 2 = **15000 m**

Distance between successive rows of contour trenches (d),

d = A/L = 250000/15000 = **16.67 m**

Number of rows of contour trenches (N),

N= L_t/d = 2500/ 16.67 = **150**

Practice Adapted in Indian State Tamilnadu:

Contour and staggered trenches are adapted in high rainfall hilly areas of lands with slopes steeper than 33% or any slope with badly eroded soil. Instead of trenching, graded trenching (modified American) have been suggested. The trenches are limited in length to about 450 m starting from the end farthest from the outlet, trenches run level for 90 -120 m, than on a gradient increasing from 1 in 500 to 1 in 300 at the outlet. The bund or equalizers in the trenches are left closer at about 3 to 4.5 m apart. The trenches are located as below according to slope:

Slope Type	Slope %	V.I. (m)
Gentle slope	5- 10	13.5 – 19.5
Medium slope	10 – 25	6 – 13.5
Steep slope	More than 25	1.25

Keywords: Trenching, in-situ water conservation

Lesson 30. Diversions and Ditches

30.1 Diversion

Diversions are the water conveyance structures that are constructed to intercept the surface runoff and transport to the main drain. The major purpose of the diversions and ditches is to convey runoff to a suitable disposal point at non-erosive velocity. The complete system to dispose excess water from surface includes: Diversion channels, ditches and grassed waterways. Diversions are constructed at the point where the soil is susceptible to be eroded due to the surface runoff attaining erosive velocity. The diversions are constructed across the prevailing slope and divert it across the slope of grassed waterways. Both diversion and ditches are in form of channel or trench with gentle slope. However, in case of ditch, the channels are deeper as compare to diversion. Grassed waterways designed to transport excess runoff down the slope to the natural drainage system.

Diversion – The design of field channel, diversion channels and grassed water ways includes the determination of cross-section for design discharge and velocity of flow. The cross section and channel slope should be sufficient to handle the design flow and maintain the flow velocity within permissible limit.

The average velocity of flow is estimated using Manning's equation.

$$V = \frac{R^{2/3} \times S^{1/2}}{n} \tag{30.1}$$

Where, is average velocity of flow (m/s), is hydraulic radius (ratio of cross sectional area of flow, A and the wetted perimeter, P). Wetted perimeter, P defined as the length of cross-section, which is in contact with the flowing water and so it depends on depth of flow, D. S is hydraulic slope (m/m) and n is the Manning's roughness coefficient.

Discharge capacity of the of the channel is estimated using the continuity equation

$$Q = A \times V \tag{30.2}$$

Where, Q is discharge (m^3/s), A is cross sectional area (m^2) and V is average velocity (m/s).

30.1.1 Diversion – Criteria for Selection

The slope is the main criteria for selection of diversion. The dimensions of diversion depend on the quantity of runoff to be intercepted and transported to the main drain. Fig.30.1 presents the concept of the diversion. The configuration of diversions is like dendrogram. The diversion are provided to prevent the advance of the gully erosion. The considerations for selection of location are:

1. where above slope runoff endangers the low-lands;

2. where gully is progressing and its head cutting has reached to advanced stage;

3. Part of the farm where runoff is concentrated and

4. Where there is a need to collect additional water particularly to divert runoff to the farm pond or natural drainage system.
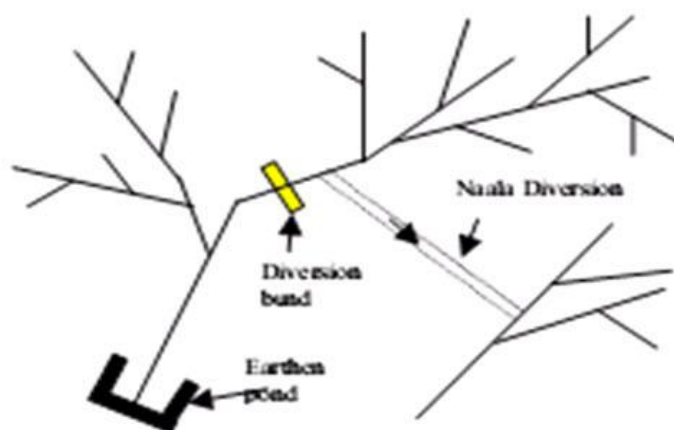
Fig. 30.1. Configuration of diversions. (Source: FAO conservation guide

13/3 Watershed management and field manual)

30.1.2 Most Economical Channel Section

The most economical channel section provides maximum flow rate at minimum excavation or amount of construction material and labor. Theoretically, highest possible hydraulic radius for given cross-sectional area would provide most economical cross-section. For these conditions, semicircular cross-section is most economical since it has minimum wetted perimeter. Generally, semi-circular cross section is difficult to construct except smaller lined channel. Trapezoidal section is most adopted because it is easy to construct. The discharge in trapezoidal cross-section is maximum when the bottom width equals to (), in which, d is the depth

of flow and is the angle of inclination of the side to the bed. For rectangular section, remains zero and hence in this case the bottom width would be equal to 2d.

30.1.3 Design Specification for Diversion

The design specifications involve the determination of runoff volume to be conveyed and suitable dimension of the diversion channel. Following points should be considered while designing the diversions.

a) The length should not exceed 350 m.

b) Quantity of runoff should be estimated for 10 years return period for agricultural land.

c) The maximum velocity in the channel should be limited as per Table 30.1

Table 30.1. Safe limit of flow velocity under different soil covered channel

Soil type	Safe limit (m/s)
Sandy soil	0.4
sandy loam and silt loam	0.6
Clay loam	0.65
Clay	0.70
Gravelly soil	1.0

A free board of 10-30 cm may be added to the depth of flow as a safeguard. Usually, the carrying capacity of the channel is higher than designed runoff to provide extra protection. The cross section is determined by the amount of runoff, designed velocity of flow and shape and side slope of the diversion and ditches. Fig. 30.2 shows typical cross-section of diversion ditch. The design of ditch can be adjusted by increasing the cross-section and gradient to accommodate larger amounts of run-off. However, the velocity of the flow should not exceed the safe limits explained earlier. Another design modification is to make the ditch's side slope and gradients less steep on sites where the soils are prone to erosion.
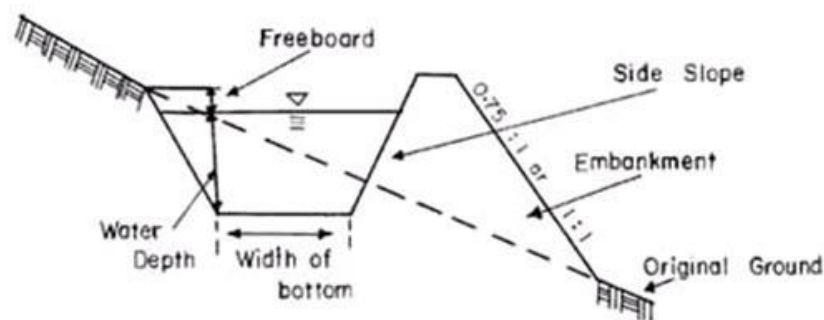
Fig. 30.2. Cross section of diversion and ditch. (Source: FAO conservation guide 13/3 Watershed management and field manual)

30.1.4 Design Procedure

The design of flow channel includes determination of flow rate to be carried out by the channel; selection of suitable channel bed slope based on topography and soil type; Selection of suitable channel shape and side slope; finding cross sectional area and wetted perimeter; estimating velocity of flow using manning's formula; Validating flow velocity to be within permissible limit and finally providing free board (about 15-20% of the depth of flow) to determine total depth of channel.

Example 30.1: Design a trapezoidal diversion channel to carry design discharge of 2 cumec to the reservoir through earthen channel. The prevailing slope is 0.001 m/m and soil type is clay loam. Take Manning's roughness coefficient as 0.03.

Solution:

The permissible velocity for this type of soil is 0.65 m/s. Using Manning's equation, determine hydraulic radius, R

$$V = \frac{R^{2/3} \times S^{1/2}}{n};$$

$$0.65 = \frac{R^{2/3} \times (0.001)^{1/2}}{0.03}$$

Thus hydraulic radius, R = 0.49 m

The hydraulic radius for trapezoidal section, $R = \dfrac{b \times d + Z d^2}{b + 2 \times d \times \sqrt{1 + z^2}}$, where b,d and z are bottom width, depth of flow and side slope, respectively. For clay loam soil, the side slope should be 2:1 i.e. Z =2, Thus

$$0.49 = \dfrac{b \times d + 2d^2}{b + 2 \times d \times \sqrt{1 + 2^2}} = \dfrac{b \times d + 2d^2}{b + 4.48 \times d} \text{ and so}$$

$$2d^2 + bd - 0.49b - 2.2d = 0 \qquad (30.3)$$

Now discharge is given as

$$Q = A \times V \text{ thus } A = \dfrac{Q}{V} = \dfrac{2}{0.65} = 3.08 \, m^2 \text{ or } 2d^2 + bd = 3.08$$

Thus equation 3 modifies to

$$0.49b + 2.2d = 3.08 \qquad (30.4)$$

For most economical cross section, $= 2d \tan{}^{\varnothing}\!/_2$, since side slope, Z is 2, Ø will be 60° thus $\tan{}^{\varnothing}\!/_2$ would be equal to 0.5. so for most economical cross-section, b=d , substituting this to equation 30.4

$$b = d = 1.15 \, m$$

Adding 15% free board, total depth of channel will be 1.32 m say 1.35 m

Hence top width, T = b+2Zd = 1.15+2x2x1.35 = 6.55 m.

Example 30.2: Design a grassed waterway which is to be constructed as an outlet for flow from a graded bud system. The expected runoff is 2.9cumec. The type of

grass to be used is dub and needs to be maintained in excellent way. The slope of the channel is to be kept at 3.5%.

Solution:

The problem (waterway designed) can be solved by trial and error method,

Assume the top width (t) of the water way = 4.75 m, Depth of flow (d) = 0.45 m

For parabolic channel, cross section area (a) = 2/3 td = 1.425 m²

Wetted perimeter (p) of the waterway = t + 8d²/3t = 4.86 m

Hydraulic radius (R) of the waterway = a/p = 0.293

Mean velocity by using the Manning's formula

$$v = \frac{R^{2/3} S^{1/2}}{n}$$

Where s = channel slope = 3.5/100

    n = 0.04 for natural channels

Therefore, v = 2.07 m/s

Flow capacity **Q** of the water way = a x v = **2.95 cumec**

The discharge capacity is slightly more than 2.95cumec and thus it's acceptable. The velocity is also within the permissible limits, Hence the design is acceptable where t = 4.75 and d = 0.45 m.

30.2 Ditches

Ditches are primarily components of surface drainage system that used to remove excess water in soil having slow infiltration, low permeability and no quick percolation. The ditch system is designed to remove surplus water within 12-24 h. Depending upon the alignment, different categories of ditch systems are described below (Sharda et al. 2007).

(i) Random field ditch system

(ii) Bedding system

(iii) Parallel field ditch system

30.2.1 Random Field Ditch System

Random field ditch system, as shown in Fig. 30.3, is best suited in the area which have randomly scattered depression or potholes and depth of cut is limited to 1 m. In this system, the depth of ditch is low to allow movement of farm equipment across. The cross section can be rectangular, triangular or parabolic and can be constructed using tractor mounted field equipments. While aligning the ditches, it should follow such a route that provides minimum cut and interference with other agricultural operations. For effective removal of excess water, the ditches may be supplemented with land grading. The outlet of this system is natural stream.

30.2.2 Bedding System

In this system, the ditches comprises of dead furrows that run parallel to the prevailing slope. The area between two adjacent dead furrow is known as bed (Fig. 30.4). These beds are aligned such that the farming operations are parallel to the dead furrows. The design and layout involves proper spacing of dead furrow to remove the excess water within 24 hours.
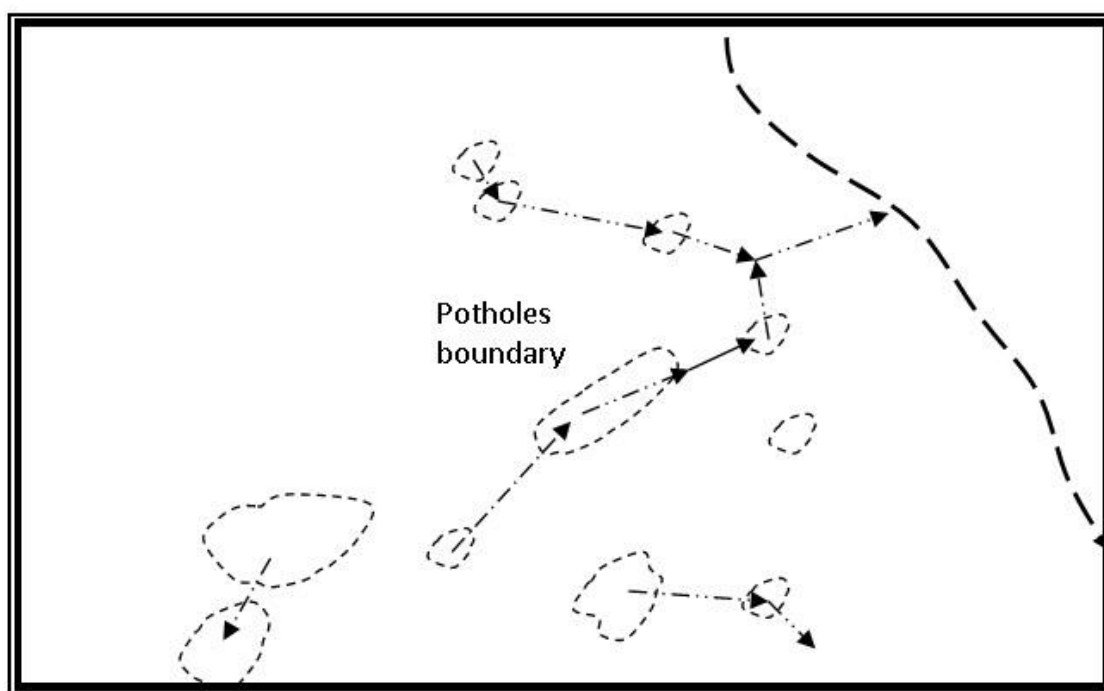
Fig. 30.3. Random field ditch system. (Source: Training manual on Soil Conservation and Watershed management (Vol II), CSWCRTI, Dehradun)
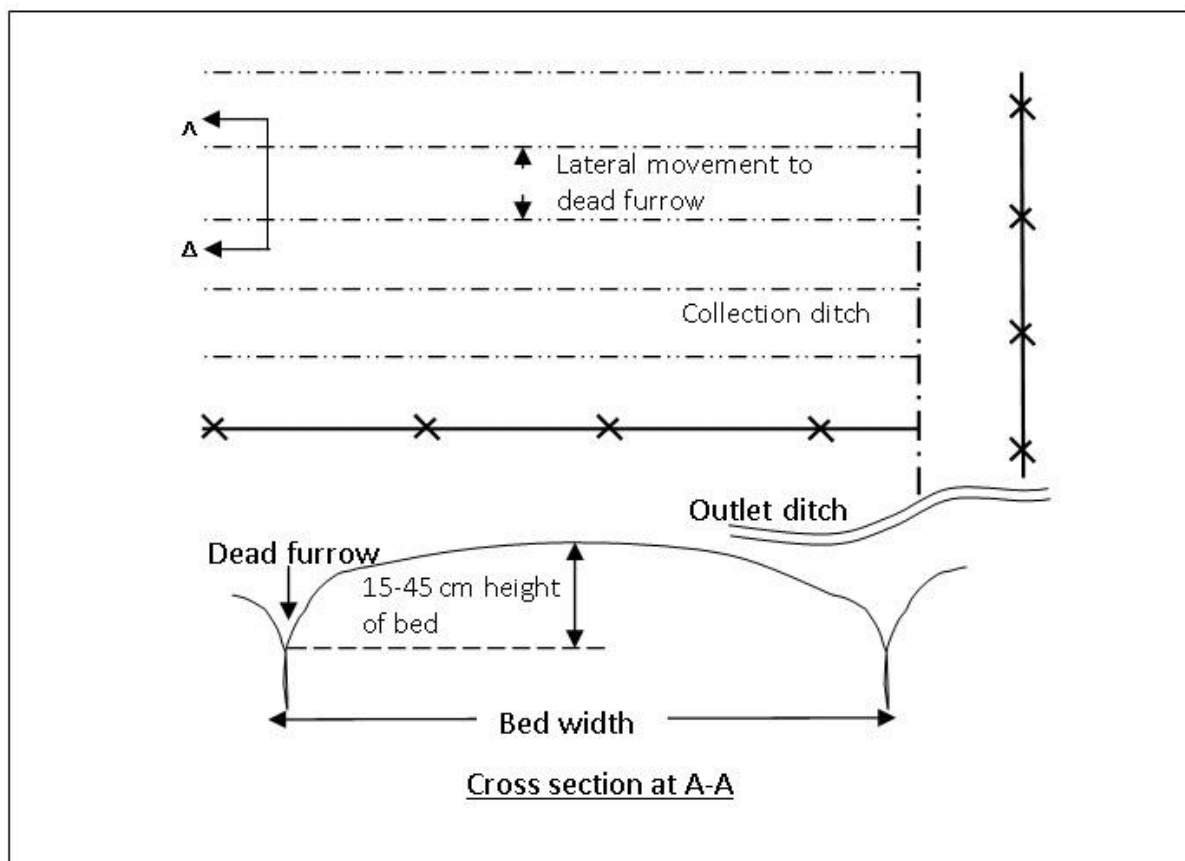
Fig. 30.4. Bedding system. (Source: Training manual on Soil Conservation and Watershed management (Vol II), CSWCRTI, Dehradun)

30.2.3 Parallel Field Ditch System

In this system, the channels are spaced widely with higher flow capacity and cultivation done across the field slope unlike bedding system. This system is most suited for flat and poorly drained soil conditions. The ditches are not necessarily spaced equally and size of ditches can be varied as per the land grading, and size of drainage area. In this system ploughing are done parallel to the system (30.5).
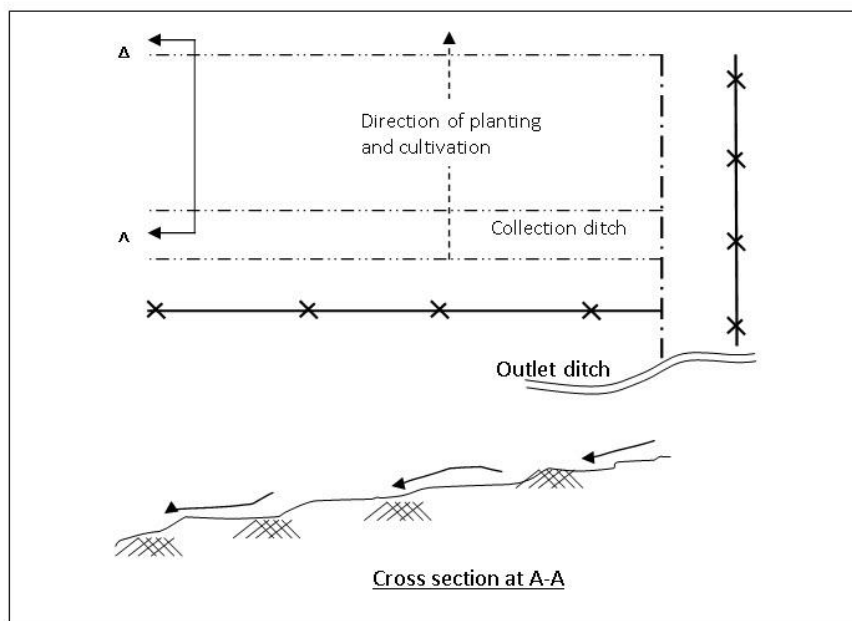
Fig. 30.5. Parallel field ditch system. (Source: Training manual on Soil Conservation and Watershed management (Vol II), CSWCRTI, Dehradun)

30.3 Design Consideration of Ditch System

The ditch system includes determination of cross sectional area and discharge is same as design of diversion considering the same side slope and permissible velocity criteria. However, the design of ditch system further influenced by factors such as maximum 1 day rainfall occurance with a frequency of 5 years; infiltration rate of soil; evaporation rate; crop grown and stages; land slope and gradient to the drainage channel. The gradient should normally follow the natural slope of the land.

Example 30.3: The drainage coefficient of an area of 40 ha is 0.4 m. Estimate the capacity of the ditch system draining the area.

 Solution:

The drainage coefficient = 0.4

Total quantity of water to be drained within 24 hours = 0.4(40X100X100)

$$= 160000 \text{ m}^3$$

Thus flow rate = 160000/(24*60*60) = 1.85 cumec.

Module 6. Cost Estimation

Lesson 31. Cost Estimation

Estimating and costing of different soil and water conservation structures are important component of any watershed management plan. The estimation includes calculation and computations of quantities of material likely to be consumed for particular work and also expenses likely to incur for execution of the work. The cost estimation has two components namely material coat and labour cost. The cost estimation forms an important basis for project approval and sanction from competent authority. Moreover, it also helps in economical evaluation of the project.

31.1 Type of Estimate

31.1.1 Preliminary Estimate

This type of estimate is based on *reconnaissance* survey and practical knowledge and past experience of similar works undertaken. This rough estimate helps in deciding the financial involvement and policy for approval in principle by the competent authority. This estimate briefly presents the cost of major components of work and cost of land, if any separately. A brief report explaining the necessity and urgency of work are also accompanied along with the site plan and preliminary estimates.

31.1.2 Detailed Estimate

This estimate is computed from plan and sectional drawing of the proposed work and is accurate. The quantities of each item for each component of work are calculated using the dimensions taken from the drawing/details. The cost involved for this estimate is known as abstract of cost. The rates of different items of work are taken as per schedule of rates (SOR) available with the department. This schedule of rate is based on prevailing market rate of labour and material for finished item. The most labour involved in soil and water conservation structures are dealing with earthwork. The prescribed earthwork volume that can be assigned to a labour are depends upon the soil type and depth. The details are published by the respective government from time to time. The labour wages are incorporated in the detailed estimate in consultation of prevailing wages in vogue. Detailed estimate consists of working out the quantities of different items of work and their cost as explained below:

Cost Index

Cost index is the value (given in percent over and above) that serves as a factor to take into the account inflation from the period when schedule of rates is published. The usual practices followed is to estimate overall cost including inflation involve the following steps.

1. Detailed estimation based on scheduled of rates is carried out. This value represents the estimated cost for the time and place for which these rates were published.

2. This estimated cost is further multiplied by the cost index (The cost index may be negative as well) which vary from time to time and place to place as follow

$$OverallCostEstimate = CostEstimate_{(ScheduledRate)}\left(1 + \frac{CostIndex}{100}\right)$$

For example, the detailed estimated cost for a drop structures is computed as Rs. 5.0 lacs on the basis of Delhi Schedule of Rates, 2012 (DSR 2012). The cost index for Hyderabad for year 2013 is 9.39. Thus, if the same structure is to be constructed at Hyderabad in the year of 2013, the overall cost estimate would be = 5X1.0939 = 5.47 lacs.

Details of Measurement and Calculation of Quantities

The proposed work is divided into different sub-heads, e.g., earthwork, concrete work, brick work, rubble work etc. Computation is carried out for each work using the methodology described below. It is important to remember that the dimensions of different components are measured from center to center.

Details of Measurement Form

Item No.	Description or particulars of item	No.	Length (unit)	Breadth (unit)	Height or Depth(unit)	Quantity (unit)

Abstract of Estimated Cost

The cost of each item of work is calculated from the quantities already computed at a workable rate and the total cost is worked out in a prescribed form (Abstract of Estimate Form) as shown below:

Item No.	Description or particulars of work	Quantity	Unit	Rate	Amount

In the above forms, the description of each item should be such that in can be expressed explicitly in terms of work, material, proportions of mortar etc. A contingency (@ 3-5 %) is added towards unforeseen expenditure, changes in design and rates etc., which may occur during the execution of work. An expenditure towards work-charged establishment (@ 1.5-2%) may also be added, if required. The grand total thus obtained is the estimated cost of the work. The following format may be utilized for preparation of estimated cost of work.

i) Name of work   ii) Location of the work   iii) Cost estimate

SI. No.	Description	Number	Measurement			Unit	Quantity	Unit rate (Rs.)	Total cost (Rs.)
			L (unit)	B (unit)	W (unit)				

Total

Contingencies   (%)
Grand Total

Prepared   by
Checked   by
Approved by

31.1.3 Revised Estimate

This estimate is also a detailed estimate when the original sanctioned estimate is likely to exceed during the execution of work. The revised estimate is then prepared incorporating the component of work/rate which is responsible for the escalation of the cost. This is necessary for obtaining the additional sanction from the authority.

31.1.4 Supplementary Estimate

This is the first detailed estimate of the work in addition to the original one and is prepared when the additional work is required to supplement the original work as a result of further development during the execution of work. The abstract of cost

should show the amount of original estimate and the amount to be incurred in supplementary estimate for which sanction is to be required.

Schedule of Rates

Estimation of the financial cost of works undertaken is usually done on the basis of a Schedule of Rates (SOR) prepared by concerned departments of each state government. The SOR breaks up all work done to build any structure into a series of tasks and provides government approved rates for costing of each task. These are called Task Rates. These task rates are for different types of work (buildings, bridges, roads, dams, irrigation canals, lift irrigation, water supply, sanitation and sewerage). The rates include labour rates (based on minimum wages), rates for materials used, rates for transportation of materials, hiring charges of machines and vehicles, royalty payments for "freely available natural resources" and rates for maintenance of structures. The SOR is used both for:

1. Preparing cost estimates for proposed works; and

2. Valuation of work already done

To use the SOR, the proposed work is needed to break up into a detailed list of all the tasks involved in it. For instance, to construct earthen contour bunds, the work can be break up into its various component tasks, as given below:

1. Excavation of earth

2. Construction of contour bunds with excavated material and

3. Providing stone exits, this in turn involves,

- collection of stones (including transportation where stones are not locally available); and

- Construction of exits with stones at appropriate places.

Similarly, construction of loose boulder checks involves the following tasks:

- Site clearance

- excavation of foundation

- collection of stones (including transportation where stones are not locally available); and

- stacking of boulders to form the boulder check

Construction of more complicated structures like core wall type earthen dams involves tasks:

1. Site clearance

2. Survey work

3. Digging cutoff trench;

4. Puddle-filling of trench;

5. Construction of core wall;

6. Raising outer embankment by laying earth, watering and compaction;

7. Stone pitching/Sodding;

8. Construction of rock toe and sand filters;

9. Excavation for exit weir;

10. Stone pitching of exit weir; and

11. Transportation of construction materials including water, clay and stone

31.2 Analysis of Rates

The rates for different items of work comprise of following components.

1. Material cost: The material cost while estimation is taken as the cost of material at the site that includes material cost, transportation and taxes if any.

2. Cost of labour: The labour requirement for different items depends on the nature of work. Thus wages as per man-hour or man-day required for particular item of work is estimated and then accounted for total cost of labour.

3. Tools and plants and sundries: A lump sum provisions are made for tools and plants (T&P) and other small items (sundries) such as water charges, which are not possible to be accounted in detailed estimate. Usually, 3-5% of labour and material costs are considered for this account.

4. Carriage: The carriage charges to transport material to the remote construction site should be added. This cost components is required where the construction site is not connected with the motorable roads etc.

5. Contractor's profit: A provision of 10-15% of total cost including material cost, labour cost, T&P may be added to get the rate per unit of item of work. However, in case where the material is supplied departmentally, contractor's profit is not applicable to the material cost.

Material required for different items of work

Table 31.1 describes the material required for different items of work specific to soil and water conservation structures.

Table 31.1. Material required for different items of works.

Sl. No.	Particulars of items	In MKS unit	
		for	quantity
1.	No. of bricks in masonry work	1 m^3	500 nos
2	Dry mortar for cement concrete	1 m^3	1.54 m^3
3	Dry mortar for lime concrete	1 m^3	0.40 m^3
4	Dry mortar for brick masonry	1 m^3	0.3 m^3
5	Dry mortar for coursed rubble stone masonry	1 m^3	0.4 m^3
6	Dry mortar for random rubble stone masonry	1 m^3	0.4 m^3
7	Stone for rubble masonry	1 m^3	1.25 m^3
8	Brick for brick ballast	1 m^3	370 nos
9	Brick ballast for lime concrete	1 m^3	1 m^3
10	Brick for R.B. lintels	1 m^3	450 nos
11	Dry mortar for RB works	1 m^3	0.45 m^3
12	Dry mortar for ashlar stone masonry	1 m^3	0.30 m^3
13	Brick bats for brick ballast	1 m^3	1.10 m^3
14	Dry mortar for pointing in brick work	100m^2	0.60 m^3
15	Dry mortar for C pointing	100m^2	0.80 m^3

16	Dry mortar for 12 mm thick cement plaster	100m²	1.44 m³
17	Cement for brick flooring	100m²	3.80 bags
18	Dry mortar for half brick wall	100m²	3.20 m³
19	Bricks required for 10 cm thick wall	100m²	50 nos
20	One bag of cement (50 kg)	-	0.035 m³
21	One m³ of cement	-	28.8 bags

Labour required for different works

Table 31.2 describes the requirement of labour for different items of work.

Table 31.2. Manpower required for different work

Sl. No	Particulars of work	Quantity of work per day	Kind of manpower per day
1.	Earthwork excavation in ordinary soil including disposal upto 30m and lift upto 1.5 m	3 m³	Mazdoor 1
2	Earth filling under floors including consolidation in 15 cm	6 m³	Mazdoor 1, Bhisti
3	Earth filling over roofing including ramming	3 m³	Mazdoor 1, mason
4	Cement concrete in foundation with brick or stone ballast; lime concrete in foundation	8 m³	Mazdoor 12, mason 1, Bhisti
5	Plain cement concrete (1:2:4)	6 m³	Mazdoor 9, mason

			1, Bhisti
6	RCC 1:2:4 in slabs, beams, lintels and column including bending and binding, reinforcement, centering and shuttering	4 m^3	Mazdoor 16, mason 1, Bhisti 2, Blacksmith , carpenter,
7	Damp proof course 2.5 cm thick 1:2:4 with 2 coats of bitumen laid hot	12 m^2	Mazdoor 2, mason , Bhisti
8	I class brick work in cement/lime/mud mortar in super structure	1.25 m^3	Mazdoor 2, mason 1, Bhisti
9	I class brick work in lime/mud mortar in foundation and plinth	1.4 m^3	Mazdoor 2, mason 1, Bhisti
10	I class brick work in cement/lime mortar in arches	0.7 m^3	Mazdoor 2, mason 1, Bhisti
11	Course/random rubble stone masonry in mud/cement/lime mortar in super structure	1.0 m^3	Mazdoor, mason 1, Bhisti
12	Course/random rubble stone masonry in mud/cement/lime mortar in super structure	0.7 m^3	Mazdoor 2, mason 1, Bhisti
13	Plastering with any mortar (12 mm thick)	12 m^2	Mazdoor 1, mason 1, Bhisti
14	Cement pointing or lime pointing	10 m^2	Mazdoor 1, mason 1
15	Conglomerate flooring (CC), 4 cm thick cement concrete 1:2:4 over 10 cm thick and cement concrete 1:6:12 over 10 cm thick	7 m^2	Mazdoor 2, mason , Bhisti
16	Brick flooring 10 cm 1:6:12 CC with sand filling and cement pointed flush	11 m^2	Mazdoor 2, mason , Bhisti

31.3 Precautions while Using SOR

1. The strata on which the rate should conform to the classification in SOR

2. Avoid duplication in the use of rates.

3. The SOR provides no rates for certain items like placing plastic sheet in an underground dyke or chain link wire in gabion structures. Such items should be costed on the basis of actual market prices.

4. In all estimates and valuations, the name of the SOR, place and year of publication, item number of the rate used and detailed description of the task should always be mentioned. This ensures cross-checking and transparency.

5. No extra rates should be applied for providing settlement allowance. The rate given for construction of earthen bunds and dams is inclusive of cost of raising the bund or dam to extra height on account of settlement allowance.

6. No extra rate is allowed for removing "dead-men" or "tell-tales". While excavating earth, certain portions are left unexcavated as evidence of the situation before work started. For instance, when constructing a dugout pond, the unexcavated portions reveal the lay of the ground before work started and hence the depth to which excavation has been done. Such evidence is necessary for conducting valuation of the structure. After fio e measurements, these portions are removed. The rate for this work includes this cost also and should not be separately charged.

7. Rates related to lift only apply after crossing the threshold of the initial free lift (usually 1.5 m). Similarly, rates for lead apply after crossing the threshold of the initial free lead (usually 50 m). The quantities of free lift and lead should be deducted from the total lift and lead while applying the rate.

31.4 Estimation of Quantity of Work

For estimating the quantity of work, structures are classified into four groups on the basis of their cross-sectional area and type of material used. Table 31.3 presents classification of various soil conservation structures.

Table 31.3. Classification of various structures for quantity estimation

Cross-sectional area	Type of Material	
	Homogeneous	**Heterogeneous**
	Group I (Contour Trenches & Contour Bunds in One Type	Group II (Contour Trenches & Contour Bunds in Different Types of

Uniform	of Strata)	Strata)
Variable	Group III (Boulder Check, Gabion Structure, Homogeneous Earthen Dam)	Group IV (Core wall Type Earthen Dam, Hearting/Casing type Earthen Dam)

31.4.1 Group I Structures

Fig. 31.1 show dimension of group I structure. When the cross-section is similar throughout the length and material is also same, the quantities can be calculated as:

Volume (V) = Length (L) x Width (W) x Height (or Depth) (H)

$$V = L \times W \times H \qquad\qquad (31.1)$$

and

Cost of Work (C) = Volume (V) x Unit Rate (R) per volume of work

$$C = V \times R \qquad\qquad (31.2)$$

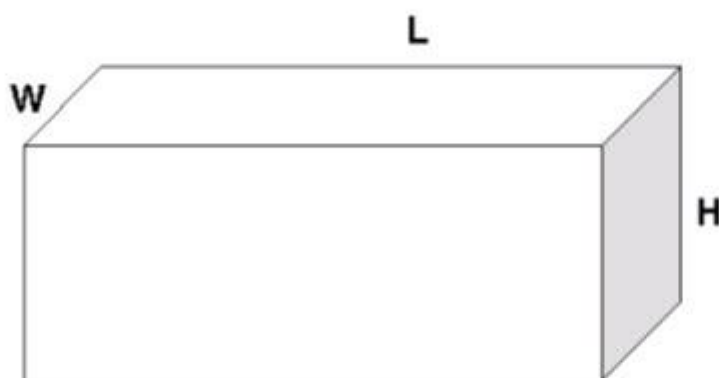Fig. 31.1. Typical dimension of Group I structures.

31.4.2 Group II Structures

In these types of structures, the cross-section area is uniform throughout the length of the structure, but material varies. The quantities of various types of materials is calculated separately, as the unit rates for different strata are different. For example, a contour trench with two layers, the upper one and the lower one.

Volume of Upper Layer V_1= Length x Width x Depth of Soft Soil Strata

V_1= L x W x H1   (31.3)

Volume of Lower Layer V_2 = Length x Width x Depth of Hard Soil

V_2 = L xW x H2   (31.4)

Total Volume (V) =V_1 + V_2 (31.5)

Assuming that the rate for digging in soft soil = $R_1$and the rate for digging in hard soil = R_2, then the total cost C will be sum of cost of work for upper layer (C_1) and cost of work for lower layer (C_2)

C = C_1 + C_2 = V_1 x R_1 + V_2 x R_2 (31.6)

31.4.3 GROUP III Structures

This group of structures has homogeneous construction material but their cross-section area varies across the length of the structure. Here, the structure is divided into several sections and volume of each section is worked out. These volumes are then summed to get the total volume of the structure. The total volume is multiplied by the rate for that particular stratum to derive the total cost. Fig. 31.2 shows a variation in cross-section of the structure at different sections.

Cross-section at A Cross-section at B Cross-section at C Cross-section at D Cross-section at E
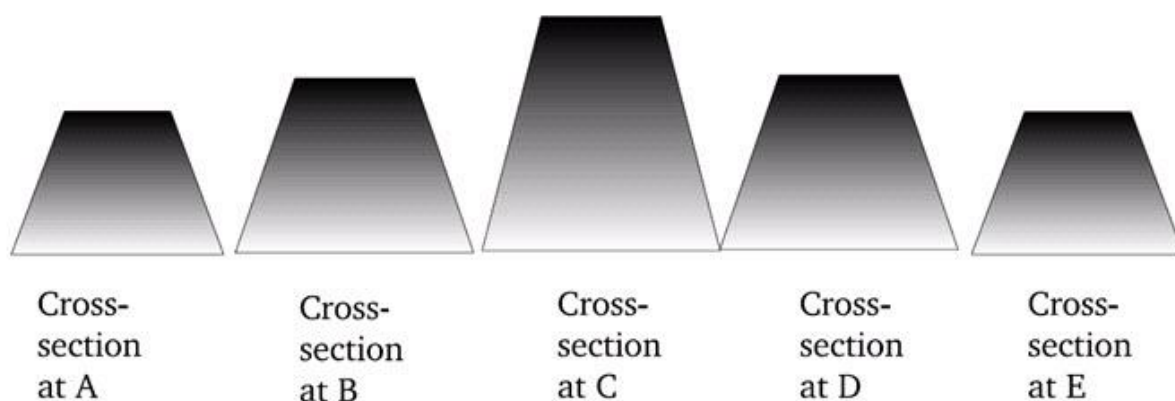
Fig. 31.2. Variation in cross section at different sections.

Total volume of work = Volume of work of Section A to B

+ Volume of work of Section B to C + Volume of work of Section C to D+...

The volume of work of a section is arrived at by averaging the cross section areas of two ends of the section and then multiplying it with the length of that section:

Volume A to B, V_1

$$= \frac{(\text{Cross section area at A} + \text{Cross section area at B})}{2} \times \text{Length AB}$$

Volume B to C, V_2

$$= \frac{(\text{Cross section area at B} + \text{Cross section area at C})}{2} \times \text{Length BC}$$

Volume C to D, V_3

$$= \frac{(\text{Cross section area at C} + \text{Cross section area at D})}{2} \times \text{Length CD}$$

Volume D to E, V_4

$$= \frac{(\text{Cross section area at D} + \text{Cross section area at E})}{2} \times \text{Length DE}$$

$$\text{Total Volume of Work, V} = V1 + V2 + V3 + V4 + \cdots \qquad (31.7)$$

As the material used is the same throughout,

Total Cost of Work = V x R   (31.8)

31.4.4 Group IV Structures

In this group, structures have variable cross section area along their length and are also made of heterogeneous construction material. Hence, cost is calculated as:

1. Calculate cross section area (C_1, C_2 etc.) for each material (for instance, top soil) at each point;

2. Calculate average cross section areas of each material;

3. Multiply this average area with the length between two points to get the volume of the material in that section;

4. Repeat this for all materials and all sections;

5. Sum up all volumes of a particular material;

6. Repeat this for all materials; and

7. Multiply volume for each material with its unit rate to get the total cost of work.

If the unit-rates for work with different types of material are R_1, R_2, R_3 ..., then:

Total Cost of Work, $C = V_1 \times R_1 + V_2 \times R_2 + V_3 \times R_3 + ...$ etc.   (31.9)

Key words: Estimation and costing, schedule of rates, cost index

Lesson 32. Cost Estimation Examples

Computation for Material Requirement – Some Examples

Example 1: Calculate dry material required for 1 m³ cement concrete (CC) having density of 2.3 ton/m³ with 1:3:6 ratio.

 Solution:

Ratio: 1:3:6 (cement: sand: 20 mm aggregate)

Sum = 1+3+6 = 10

From Table 31.1 (lesson 31), dry matter required for 1.0 m³ CC mix is 1.54 m³

Cement requirement = $\frac{1}{10} \times 1.54$ = 0.15 m³= 216 kg (using Table 32.1 below)= 4.3 bags of 50 kg each

Sand requirement = = 0.46 m³ = 1.22 ton

Aggregates = = 0.92 m³ = 1.56 ton (20 mm aggregates)

Table 32.1. Bulk density of different construction material

Material	Bulk Density (kg/m³)
1. Cement	1440
1. Aggregates (10 mm)	1600
1. Aggregates (20 mm)	1700
1. Aggregates (40 mm)	1950
1. Solid rock	2700
1. Sand	2650
1. Cement mortar	2160

Example 2: Find dry matter for 1.0 m^3 brick masonry with cement and sand mortar of 1:6 ratio.

 Solution:

For 1.0 m^3 brick work, 500 bricks are needed (from Table 31.1 described in lesson 31).

Cement: sand ratio = 1(cement):5(sand)

Sum = 1+5 = 6

 For 1 m^3 brick masonry, 0.3m^3 cement mortar is required (see material requirement Table in lesson 31.1).Cement requirement = 0.05 m^3 and

Sand requirement = $\frac{5}{6} \times 0.3$ 0.25 m^3

 Some Example of Rate Analysis

 Example 3: Determine unit cost for Gabion works

 Solution:

The gabion work involves collection of stones (225 mm) and their arrangement in wire mesh accordingly. The material requires are GI wire mesh and stones. Labours (unskilled and semi-skilled) worker are required for netting the wire mesh. The unit cost is computed as per the following analysis.

Particulars	Quantity	Rate (Rs/unit)	Amount (Rs)
Material			
1. GI wire 10 gauge with 10cm X 10cm spaced in 3mX1m size (standard size available in the market). For 3 m^3 gabion, 14 m^2 surface area of GI wire is needed. Thus weight of the GI wire @1.3 kg/m^2 including wastage	18.2 kg	60	1092
1. Stones of size > 225 mm including wastage (25%) and transportation at site	3.75 m^3	200	750

Total material cost			1842
Labour			
1. Skilled labour for wire netting	no	240	120
1. Semi skilledlabour for placing stones in the net	no	220	275
1. Unskilled labour		180	405
Total labour cost			800
Total material and labour cost			2642
Add 3% contingency			79.26(say 80)
Grand total for 3 m³			2722
Cost per m³			907.33(say 910)

Example 4: Determine unit cost for logwood crib structures filled with stones.

Solution:

The logwood crib structures involve placement of stone (250 mm or more) in between the wooden pole to provide a barrier to flowing water. The required materials are wooden logs, nails, oils for painting, stones whereas labour requirement include skilled labour such as Carpenter, Mason, Painter and unskilled labour. The unit cost is computed as per the following analysis of rate for 15 m³ crib for span of 10 m.

Particulars	Quantity	Rate (Rs./unit)	Amount (Rs.)

Material			
1. Wooden logs			
1. Vertical post in two rows 1 meter Centre to Centre and span of 10 meters; 2.15 m long and 100-120 mm in diameter	22 poles	120	2,640
1. Horizontal post of 3.0 meter long and 100-120 mm in diameter placed 50 cm apart	30 poles	140	4,200
1. 200 to 250 mm long iron nails for joining logs at 88 joints (say 100); weight of 100 nails @200 gm per nails	20 kg	60	1,200
1. Oil Painting	5 lit	40	200
1. Creosote oil			
1. Turpentine oil	5 lit	30	150
1. Stones at site (> 250mm)	15m^3	200	3,000
1. T&P (Drills, hacksaw, Hammer etc.)	Lumpsum		1,000
Total material cost			12,390
Labour			
1. Carpenter	10 nos.	240	2,400
1. Mason	5 nos.	240	1,200
1. Painter	2 nos.	240	480
1. Unskilled	20 nos.	180	3,600

Total labour cost	7.680
Total labour and material cost	20,070
Add 3% contingency	602
Grand total for 15 m³	20672
Cost per m³	1378 (say 1380)

Example 5: Determine unit cost for RCC work

Solution:

RCC work includes steel reinforcement of cement concrete. The most adopted cement concrete is of 1:2:4 mix of cement, sand and aggregates (volume basis). The density of concrete depends on the nature of aggregates used. Most frequently, 20mm stone ballast are used as aggregates. Reinforcement is done using steel bars. In watershed structures 12 mm steel bars are used. The unit cost can be computed as per the following analysis of rates.

Dry mortar required for 1m³ cement concrete = 1.54 m³.

Thus for 10 m³ cement concrete, dry mortar volume will be 15.4 m³

Cement requirement = 15.4/7 = 2.2 m³ = 64 bags of 50kg each

Sand requirement = 4.4 m³ and

Aggregates requirement = 8.8 m³

Assuming density of concrete as 2300 kg/m³, the weight of 10 m³ such concrete will be = 23 ton

Particulars	Quantity	Rate (Rs/unit)	Amount(Rs)
Material			

Cement grade 53	64 bags	280	17,920
Coarse sand (1-2 mm)	4.4 m³	2000	8,800
Stone ballast 20mm	8.8 m³	1200	10,560
Mild steel bar @1% reinforcement	0.23 ton	60000	13,800
Binding wires (1 mm)	2 kg	65	130
Total material cost			51,210
Labour			
Head mason (Raj mistri)	1	270	270
Mason (mistri)	3	240	720
Unskilled labour (beldar)	12	180	2160
Bhisti	6	220	1320
Sundries, T&P etc	Lumpsum		1000
Total cost of labour			5,470
Centering and shuttering			
Timber planks and post	On hire		1,000
Carpenter	6 nos	240	1,440
Unskilled labour (beldar)	1		

Nails	Lumpsum		500
T&P	Lumpsum		1,000
Sub-Total			3,940
Total of material and labour cost			60,620
Add 3% contingency			1820
Total cost for 10 m³ RCC			62,440
Cost of per m³ RCC			6,244 (say 6,250)

Example 6: Determine unit cost for I-class brick work with 1:6 cement mortar

Solution:

First estimate cost for 10 m³ brick work in order to rationalize the labour requirement and mortar mix.

Dry matter requirement for mortar in brick work = $0.3 m^3$ per m³ of brickwork. Thus $3.0 m^3$ dry matter will be required for $10 m^3$ brick work.

Cement requirement = $(1/7)*3$ = 0.43 m³ or 12.47 (say 13) bags of 50 kg each.

Sand requirement = $(6/7)*3$ = 2.58 m³

The unit cost can be computed as per the following analysis of rates.

Particulars	Quantity	Rate (Rs/unit)	Amount(Rs)
Material			
Cement grade 53	13 bags	280	3,640
Coarse sand (1-2 mm)	2.58 m³	2000	5,160

Class-I brick @500 per m³	5000	4.50	22,500
Total material cost			31,300
Labour			
Head mason (Raj mistri)	1	270	270
Mason (mistri)	10	240	2,400
Unskilled labour (beldar)	7	180	1,260
Bhisti	2	220	440
Sundries, T&P etc.	Lumpsum		1,000
Total cost of labour			5,370
Total of material and labour cost			36,670
Add 3% contingency			1,100
Total cost for 10 m³ RCC			37,770
Cost of per m³ RCC			3,777 (say 3,800)

Estimating and costing of some watershed management work - Examples

Example 7:

Estimate the cost to construct CCT in hard soil in 20 ha area. The distance between two row of CCT is kept as 25 m and width and depth of CCT is 50 cm each.

Solution:

The construction of CCT involves excavation only and so only labour is required with some T&P.

Length of CCT per ha can be calculated using following equation

$$LengthofCCTperha = \frac{Area}{Distancebetweentworowsoftrench}$$

$$= \frac{10,000 \text{ sqm}}{25 \text{ m}} = 400 \text{ m}$$

Earthwork volume = 0.5*0.5*400 = 100 m^3

Particulars	Quantity	Rate (Rs/unit)	Amount(Rs)
Labour			
Unskilled labour @1.5 m³ per manday	67 mandays	180	12,060
Skilled labour	2 mandays	240	480
T&P	Lumpsum	-	500
Total labour cost			13,040
Per m³ earth work			130

Example 8:

Gabion structures are proposed to stabilized 240 m long and 8 m wide gully. The average slope of the gully bed is 10% and the vertical interval may be taken as 2 meter. The proposed design is presented in Fig. 32.1. Estimate the project cost.

Example 2: Continuous Contour Trenches (CCT) in Heterogeneous Material

Example 2: Continuous Contour Trenches (CCT) in Heterogeneous Material
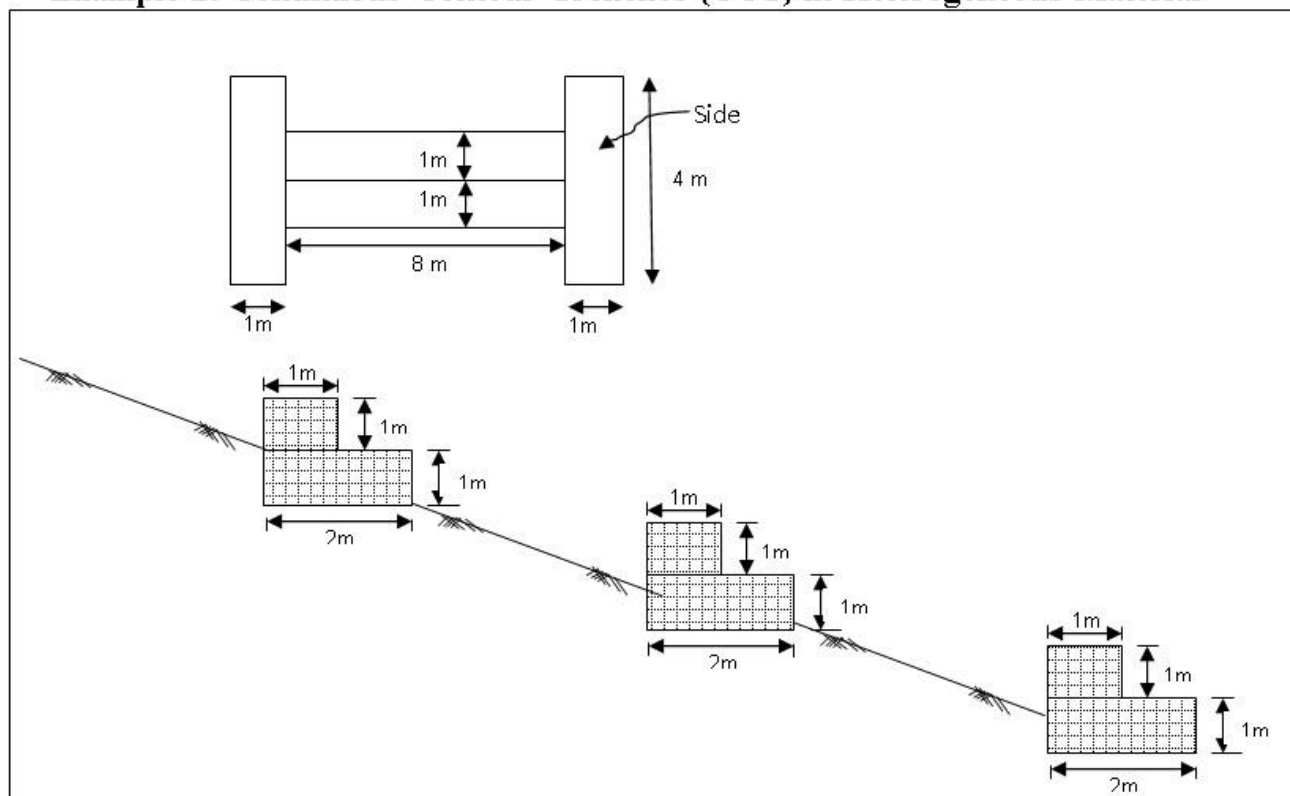
Department of Agriculture in Gaya District, Bihar proposes to make CCT in a catchment area of 40 hectares. The top soft soil stratum has a depth of 30 cm followed by hard soil for about 1 metre. The horizontal spacing between two successive rows CCT is 25 m. The width and depth of the CCT is 50 cm x 50 cm. Calculate the cost of construction of CCT.

Solution:

Fig. 32.1 Proposed design of gabion

Solution: Since the vertical interval is 2 meter and 10% slope, means gabion should be 20 m apart horizontally. Thus total number of gabion for the gully stabilization would be 240/20 = 12. The average gully width is 8 meter and 1 meter wide side wall is to be provided, the total width for earth work would be 8+2 =10m. The estimate involves computation of material and abstract of cost.

- Details of measurement and quantities

Sl. No.	Particulars of work/items	No	Length (m)	Width (m)	Height (m)	Quantity (m^3)

1	Excavation in foundation	12	10	2	0.6	144
2	Gabion box filled with boulders of >200mm					
	1. Bottom	12	10	2	1	240
	1. top	12	10	1	1	120
	1. side	12	4x2 (two sides per gabion) = 8	1	1	96
Total gabion work						456

- Abstract of cost

Sl. No.	Particular of work/ items	Quantity	Unit	Rate (Rs/Unit)	Amount (Rs)
1	Excavation in foundation	144	m^3	130(see example 7)	18,720
2	Gabion work	456	m^3	910(see example 3)	414,960
Total cost					433,680

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