Addis Ababa University
Faculty of Technology
Department of Civil Engineering

HYDRAULLIC STRUCTURES I

LECTURE NOTE

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Addis Ababa

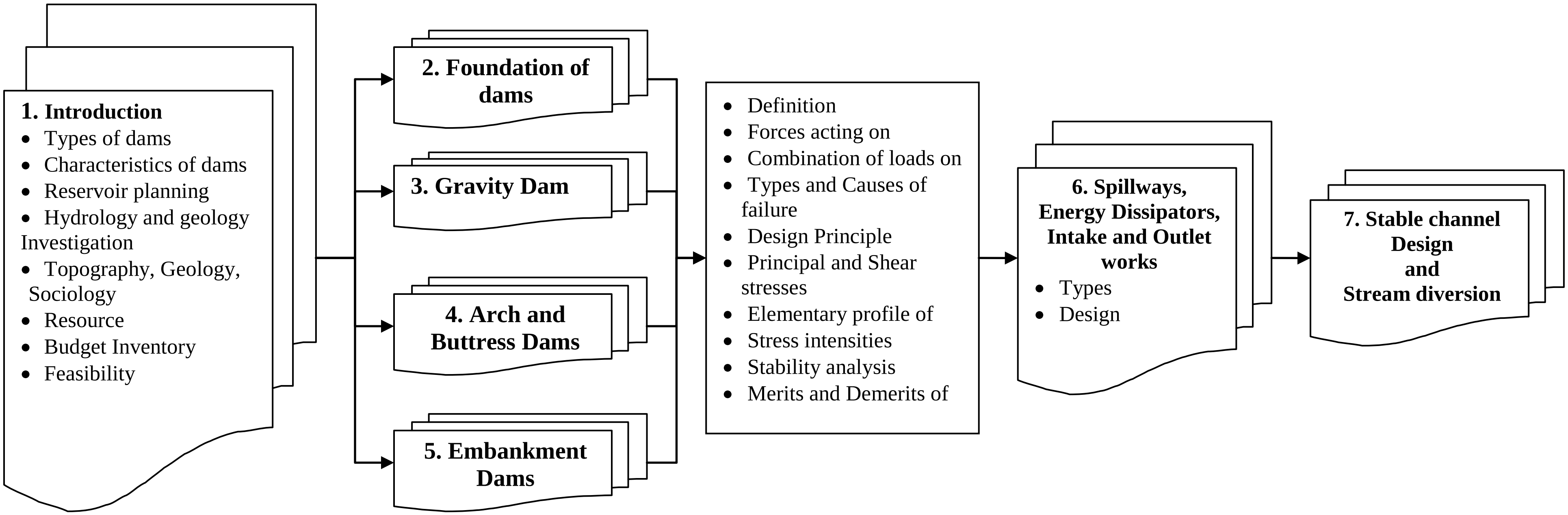
May 2006

Course Syllabus

Course Objective

This course provides a broad understanding of the basic principles of hydraulic structures. The emphasis is on design and analysis of different types of dams and spillways. Computer applications included.

Course Content



Reference Books:

1. P. Novak, Hydraulic Structures
2. S. R. Sahasrabudhe, Irrigation Engineeering and Hydraulic Structures
3. S. K. Garg Irrigation Engineering and Hydraulic Structures
4. V. T. Chow, Open Channel Hydraulics
5. USBR, Design of Small Dams

Learner Assessment 100%

Assignment 10 %
Project 10%
Mid Exam 30 %
Final Exam 50 %

Instructor

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Table of Contents

1 Introduction

1.1 General

Hydraulic Structures are engineering constructions designed and mechanically fit for managing and utilizing water resources to the best advantage of the human being and environment.

Dam is a barrier across flowing water that obstructs, directs or retards the flow, often creating a Reservoir.

Reservoir is an artificial lake created by flooding land behind a dam. Some of the world's largest lakes are reservoirs.

Spillway is a section of a dam designed to pass water from the upstream side of a dam to the downstream side. Many spillways have gates designed to control the flow through the spillway.

Flood is an overflow or an expanse of water submerging land.

Dams differ from all other major civil engineering structures in a number of important regards:

- Every dam, large or small, is quite unique; foundation geology, material characteristics, catchment flood /hydrology etc. are each site-specific.
- Dams are required to function at or close to their design loading for extended periods.
- Dams do not have a structural lifespan; they may, however, have a notional life for accounting purposes, or a functional lifespan dictated by reservoir sedimentation.
- The overwhelming majority of dams are of earth fill, constructed from a range of natural soils; these are the least consistent of construction materials.
- Dam engineering draws together a range of disciplines, e.g. Structural and fluid mechanics, geology and geotechnics, flood hydrology and hydraulics, to a quite unique degree.
- The engineering of dams is critically dependent upon the application of informed engineering judgment.

Hence the dam engineer is required to synthesize design solutions which, without compromise on safety, represent the optimal balance between technical, economic and environmental considerations.

1.2 Types of dams

Dike is a stone or earthen wall constructed as a defense or as a boundary. The best known form of dyke is a construction built along the edge of a body of water to prevent it from flooding onto adjacent lowland.

Levee is a natural or artificial structure, usually earthen, which parallels the course of a river. It functions to prevent flooding of the adjoining countryside. However it also confines the flow of the river resulting in higher and faster water flow.

Weir is a small overflow type (designed to be overtopped) dam commonly used to raise the level of a small river or stream. Water flows over the top of a weir, although some weirs have sluice gates which release water at a level below the top of the weir.

Check dam is a small dam designed to reduce flow velocity and control soil erosion

Diversion dam is a type of dam that diverts all or a portion of the flow of a river from its natural course

Masonry dam is a type of dam constructed with masonry. It is made watertight by pointing the joints with cement. A plaster of cement is also applied. The interior could be either in coursed masonry or rubble masonry.

Dams may be classified according to

Material of construction,
structure,
intended purpose or
height;

Hence, no classification is exclusive.

Classification according to material of construction

- Timber dams
- Steel dams
- Concrete dams
- Earth dams
- Rockfill dams
- Combined dams

Classification according to design criteria

Hydraulic design	Stability consideration
Non-overflow dams	Gravity dams
Overflow dams	Non-gravity dams
Composite dams	

Classification according to Purpose

Storage dams	Stage control dams	Barrier dams
Flood control	Diversion	Levees and dykes
Water supply	Navigation	Coffer dams
Detention storage		

Classification according to height (H)

- $H \leq 30$m low dam
- $30 \leq H \leq 100$m medium
- $H \geq 30$m high dam

When the size of the dam has been determined, the type of dam envisaged requires certain geological and topographical conditions which, for the main types of dams, may be stated as follows.

Concrete Dams	**Embankment Dams**
Gravity dams	Rock fill dams
Buttress dams	Hydraulic fill dams
Multiple ach dams	Earthen embankments
Thick arch dams	Composite dams
Thin arch dams	

1.3 Characteristics of dams

Coffer dam is a temporary structure constructed of any material like timber, steel, concrete, rock or earth. It is built to enclose certain work site or to divert the flow to enable construction activity in the main river channel. After the main structure is built (bridge, barrage or dam) either the coffer dam is dismantled or it becomes part6 of the structure if the design so provides.

Gravity Dams

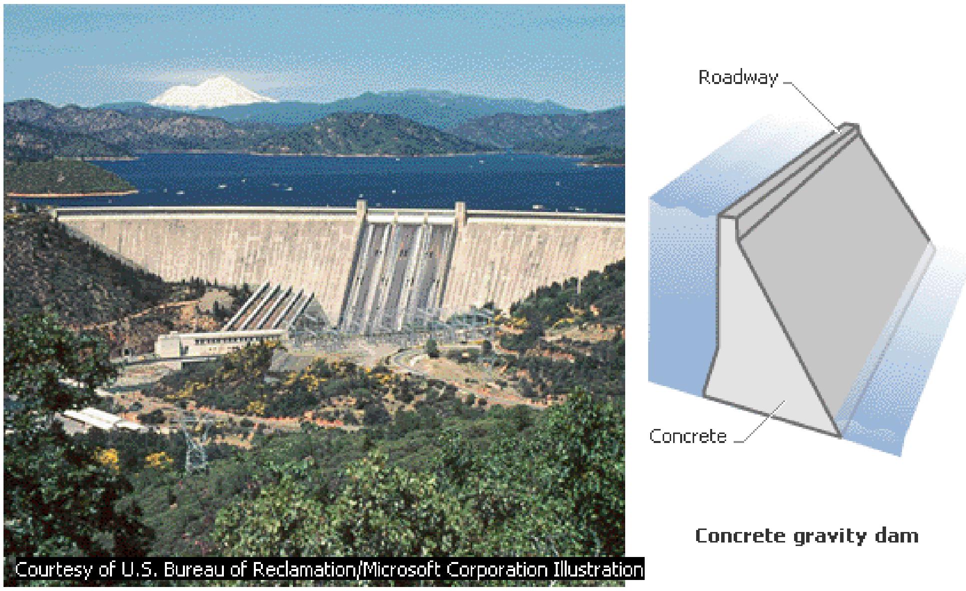
Stability is secured by making it of such a size and shape that it will resist overturning, sliding and crushing at the toe.

The dam will not overturn provided the resultant force falls within the base.

to prevent tension at the upstream face and excessive compression at the downstream face, the dam cross section is usually designed so that the resultant falls within the middle third at all elevations of the cross section

good impervious foundations are essential

inspires more confidence in the layman than any other type; it has mass that lends an atmosphere of permanence, stability, and safety



Shasta Dam impounds the Sacramento River in northern California. Like all concrete gravity dams, Shasta Dam holds back the water in its reservoir, Shasta Lake, by the sheer force of its weight. Built of solid concrete, the massive structure rises 183 m (602 ft). It measures 165 m (542 ft) at the base and just 9 m (30 ft) at the crest. This shape, typical of concrete gravity dams, counteracts the force of the water pressing against the dam at the bottom of the reservoir, where the pressure is most intense.
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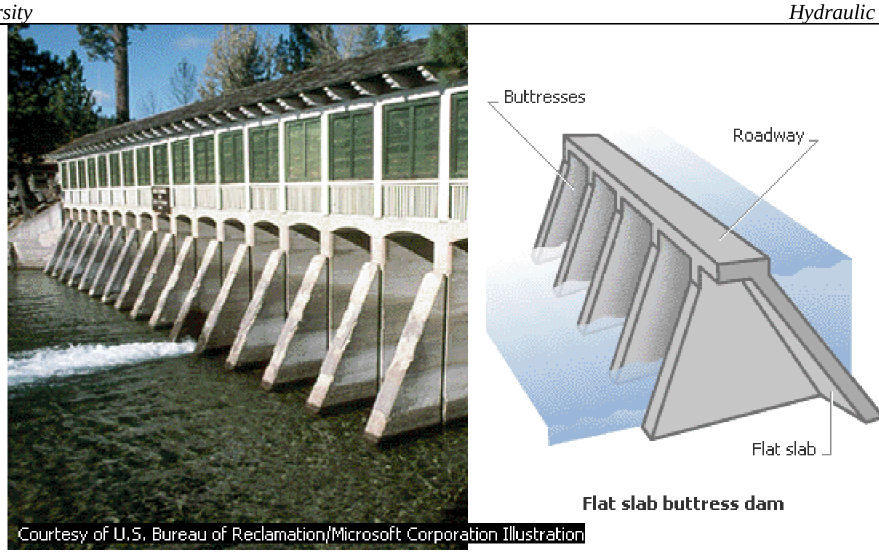
Figure 1-1 Example of concrete Gravity dam

Gravity dams are classified as "solid" or "hollow." The solid form is the more widely used of the two, though the hollow dam is frequently more economical to construct. Gravity dams can also be classified as "overflow" (spillway) and "non-overflow."

Buttress Dams

- The buttress dam is suitable where the rock is capable of bearing pressures of 2 - 3 MPa.
- Buttress dams require between one thirds and half of the concrete required for a gravity section, hence making it more economical for dams over 14m.
- Additional skilled labor is required to create the formwork.
- Threat of deterioration of concrete from the impounded water is more likely than from a thick gravity section.
- There is also an elimination of a good deal of uplift pressure, the pressure resulting from the water in the reservoir and possibly of water from the hillside rocks gaining access through or under any grout curtain and exerting upwards underneath the mass concrete dam.

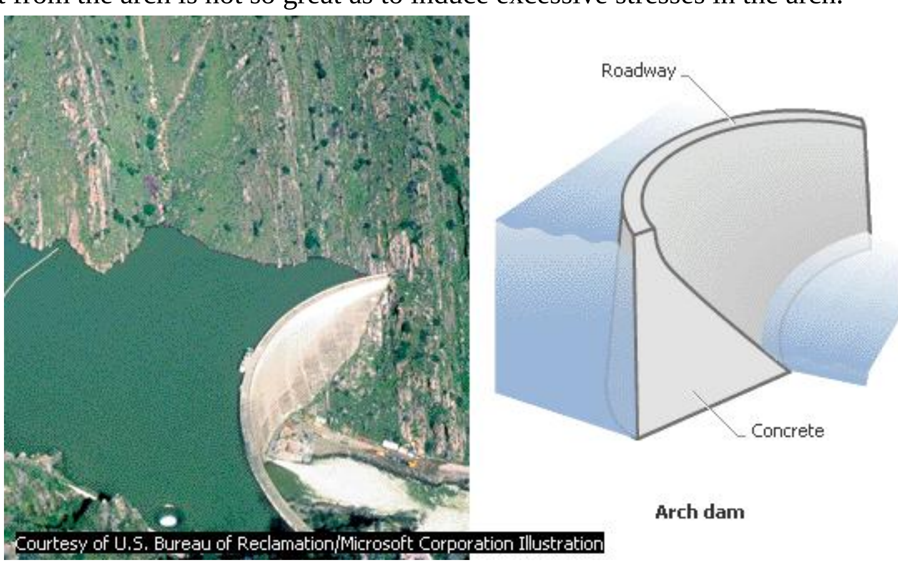


Lake Tahoe Dam impounds the Truckee River in northern California. Like all flat slab buttress dams, it has a flat slab upstream face supported by a series of buttresses on the downstream side. Lake Tahoe Dam measures 5.5 m tall and 33 m long. It was completed in 1913 to raise the water level in Lake Tahoe, a natural lake, to provide additional water for crop irrigation.

Figure 1-2 Profile of buttress dam

Arch dam

- Stability is obtained by a combination of arch and gravity action
- Utilizes the strength of an arch to resist loads placed upon it by 'arch action'
- The foundations and abutments must be competent not only to support the dead weight of the dam on the foundation but also the forces that are directed into the abutments because of arch action in response to the forces acting on the dam.
- The strength of the rock mass at the abutments and immediately down valley of the dam must be unquestionable and its modulus of elasticity must be high enough to ensure deformation under thrust from the arch is not so great as to induce excessive stresses in the arch.



Monticello Dam impounds Putah Creek west of Sacramento, California. The solid concrete structure stands 93 m tall. The dam's arched upstream face transfers some of the pressure from its reservoir, Lake Berryessa, onto the walls of the canyon. This design enables an arch dam to be much less massive than an equivalent gravity dam, which relies solely on the force of its weight to hold back the water in a reservoir. While Monticello Dam measures 30 m at its base, an equivalent gravity dam might be more than five times as thick at the base.

Figure 1-3 Sample of Arch dams

Rockfill dams

- Can be built where the following conditions exist -
- Uncertain or variable foundation which is unreliable for sustaining the pressure necessary for any form of concrete dam.
- Suitable rock in the vicinity which is hard and will stand up to variations of weather.
- An adequate amount of clay in the region which may be inserted in the dam either as a vertical core or as a sloping core.
- Accessibility of the site and the width of the valley is suitable for the manipulation of heavy earth-moving machinery, caterpillar scrapers, sheepfoot rollers and large bulldozers.

Hydraulic Fill Dams

Hydraulic fill dams are suitable in valleys of soft material and are constructed by pumping soft material duly consolidated up to moderated heights up to 30m.

A hydraulic fill is an embankment or other fill in which the materials are deposited in place by a flowing stream of water, with the deposition being selective. Gravity, coupled with velocity control, is used to effect the selected deposition of the material.

Earthen Embankment dams

Near the site there must be clay to fill the trench and embanking material capable of standing safely, without slipping, to hold up a clay core.

An advantage of earthen embankments is that troubles due to the deterioration of the structure by peaty waters of low pH do not arise.

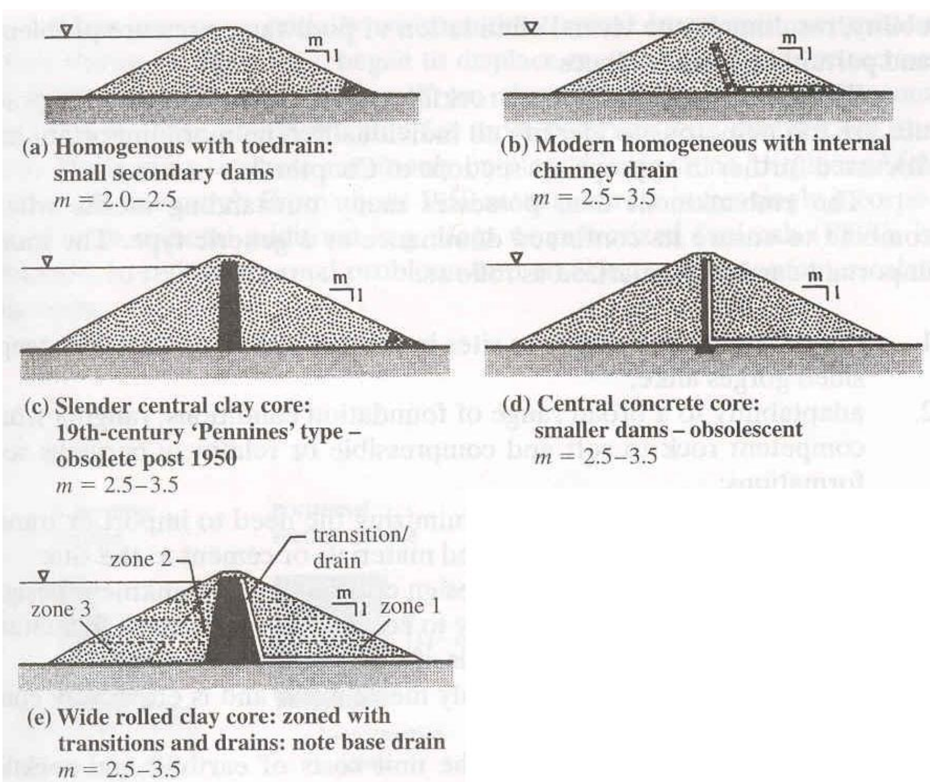


Figure 1-4 Typical profiles of Earth dam

Rock-fill dams

are embankments of loose rock with either a watertight upstream face of concrete slabs or timber or a watertight core

Where suitable rock is at hand, a minimum of transportation of materials can be realized with this type of

Resist damage from earthquakes quite well.

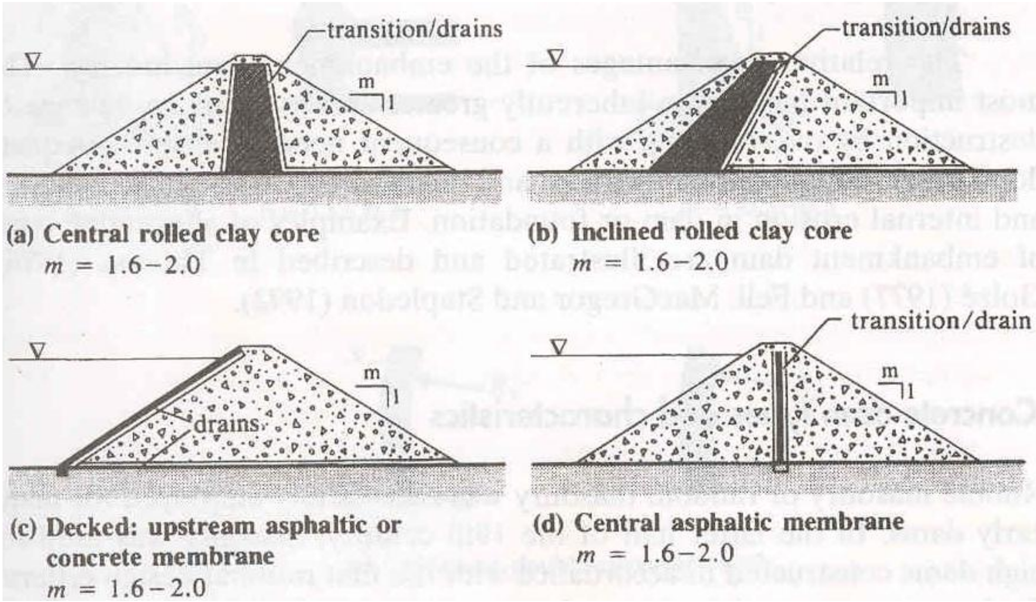


Figure 1-5 Typical profile of Rock fill dams

Composite Dams

Not only can different types of dam can be built in the same valley, but the same dam can be of different types owing to the varying geological and topographical features of the dam site.

Many buttress dams also join up with gravity mass concrete dams at their haunches at the sides of the valley, and again at the centre have a mass concrete gravity dam to form a suitable overflow or spillway.

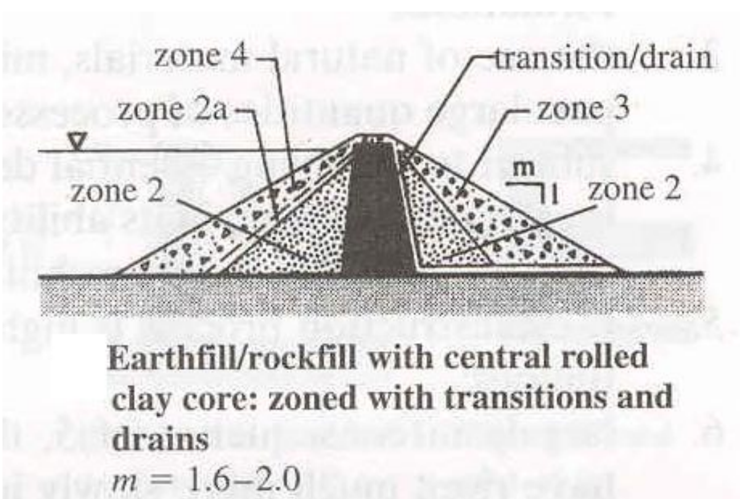


Figure 1-6 Profile of Composite dam

1.4 Reservoir Planning

The absence of natural storage of adequate capacities necessitates construction of some artificial storage works. Development of natural storages may also be included in this category sometimes (Cherecherea weir at Lake Tana). In rainy season there is excess flow down the valley in a river. An impounding reservoir can be constructed in the valley to store this excess water which will meet the demand in dry periods.

Storage works may be designed and constructed to serve single or multipurpose. The various purposes for which storage works are required are mentioned below

1. Irrigation
2. Hydro-electric power generation,
3. control of destructive floods
4. Low water regulation for navigation
5. Domestic and industrial water supply
6. Recreation
7. Preservation and breeding of useful aquatic life, etc.

Before any dam is built, certain hydrological information is necessary regarding river discharge, rate and character of siltation, and the location and duration of flooding. A critical concern in rivers is the magnitude and duration of discharge with respect to time. Feasibility studies are necessary in assessing the water budget for future industrial operations. Relevant studies involve meteorological monitoring, hydrological measurements, reservoir capacity, safe yield, and flood frequency. Questions that need to be confronted during dam site investigation include the depth at which adequate foundation materials exist, the strength of the rocks and soils, and the likelihood of water leakage.

By analysis of storage data, availability of water is ascertained before any project is contemplated. The next step in reservoir planning is to fix the reservoir capacity. The reservoir has to provide sufficient storage for various purposes, namely

1. Dead storage to contain silt deposition,
2. Storage to account for evaporation loss
3. Live storage to meet the downstream demands for irrigation domestic or industrial supply, power generation, etc.
4. Storage to act as flood protection.

The basis of fixing storage capacity for dead storage and evaporation loss depends upon the amount of incoming sediment and the annual evaporation loss respectively. Requirement for flood protection depends on the intensity and volume of flood flow. The live storage capacity of a reservoir depends on the demand for various purposes. It can be arrived at by plotting the mass curves of demand and inflow of accumulated flow or rainfall plotted against time. The capacity of the reservoir is fixed in such a way as to take care of the demands during the minimum flow period in the driest year on record. In some cases it is necessary to cover a period of successive dry years to consider storing of sufficient water to meet the demand during periods of prolonged drought.

1.5 Hydrology and geology

The hydrological and geological or geotechnical characteristics of catchment and site are the principal determinants establishing the technical suitability of a reservoir site. The hydrology of the catchment indicates the available quantity and quality of water to be stored in the reservoir. The geology of the site is one of the important aspects to be investigated for a dam to take decision about selection and location of the reservoir and the type, and size the dam. More discussion will be made in the following sections about the hydrology and geology considerations for dam design.

Assignment. #1

It is proposed to construct a reservoir in a river basin for which 34 years run off data is available as shown in Table 1-1. The pattern of releases required from the storage to meet irrigation and hydro-power generation requirements are 1300Mm^3 per annum. Take average annual evaporation loss to be 120Mm^3. Assuming the rate of demand distribution to be uniform over the year calculate the capacity of the reservoir that needs to be created.

Table 1-1 Monthly flow data of a nearby hydrologic station in m^3/s

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	AnnuaL Q
1965	6.04	4.22	10.54	11.49	10.57	10.21	125.57	414.22	276.45	30.16	17.25	16.51	933.23
1966	11.85	15.14	10.26	18.59	6.1	17.56	162.21	651.63	426.9	43.23	11.78	7.88	1383.13
1967	6.29	8.16	6.39	7.68	34.24	32.79	213.26	405.98	296.78	85.18	43.48	13.17	1153.4
1968	14.44	25.05	20.1	91.7	25.72	25.25	261.3	464	262.9	57.12	21.45	20.79	1289.82
1969	23.4	31.51	66.51	36.47	43.21	75.73	431.3	897.4	336.7	28.72	16.48	14.61	2002.04
1970	17.92	14.56	64.89	20.34	11.47	16.73	481.8	842.2	384.2	45.62	19.38	17.27	1936.38
1971	18.42	8.91	5.28	12.43	15.13	106.21	455.13	977.54	709.57	47.7	15	12.34	2383.66
1972	13.1	23.01	27.13	33.69	22.68	29.01	218.93	397.65	146.19	20.01	7.83	6.39	945.62
1973	8.92	6.9	6.32	5.68	11.15	16.83	133.04	540.64	429.09	87.16	11.38	7.99	1265.1
1974	7.85	6.24	15.9	18.2	13.78	22.48	296.45	538.89	314.51	35.53	15.17	11.38	1296.38
1975	6.35	5.78	5.86	9.35	9.25	29.37	349.02	473.14	358.47	38.48	11.85	9.51	1306.43
1976	8.91	7.85	13.6	13.68	18.81	28.68	184.22	440.84	216.59	21.52	16.78	10.8	982.28
1977	17.88	12.7	10.7	16.4	19.3	41.42	320.93	558.26	270.18	69.22	210.81	16.74	1564.54
1978	13.42	14.78	19.58	11.8	13.4	50.5	241.47	527.92	325.97	83.79	16.9	14.68	1334.21
1979	18.74	14.06	21.25	17.15	27.12	31.73	220.68	533.13	177.24	47.79	17.33	14.08	1140.3
1980	13.41	13.1	9.97	8.1	17.47	30.24	260.32	597.44	226.63	31.94	12.48	11.51	1232.61
1981	10.91	9.5	41.83	59.18	20.9	16.88	248.7	510.7	492.9	51.57	16.39	14.24	1493.7
1982	11.99	11.09	10.03	15.32	14.43	16.99	120.08	469.42	199.56	71.05	15.65	13.14	968.75
1983	10.49	10.22	14	30.07	62.84	60.47	161.26	633.54	379.24	42.86	14.73	11.06	1430.78
1984	10.11	7.37	7.53	5.68	10.96	66.13	325.27	372.87	277.83	18.13	8.34	7.51	1117.73
1985	7.1	5.52	4.43	7.07	26.5	19.33	231.92	750.94	322.14	29.45	9.58	9.92	1423.9
1986	5.45	14.98	15.2	26.14	21.54	59.13	187.81	417.06	266.62	22.43	9.27	8.77	1054.4
1987	7.9	8.89	34.26	64.72	58.11	75.84	131.6	192	46.53	17.34	9.65	7.98	654.82
1988	8.59	7.97	8.16	11.11	8.52	24.9	138.66	736.75	548.97	52.54	11.83	9.86	1567.85
1989	10.78	13.49	16.59	28.75	12.77	21.17	286.32	540.02	416.9	31.18	10.15	10.62	1398.72
1990	8.99	28.64	47.48	68.29	12.13	25.72	235.25	676.85	306.22	41.47	9.92	7.82	1468.79
1991	7.44	11.49	22.43	7.3	6.85	24.46	256.25	773.08	429.32	23.07	9.35	8.61	1579.65
1992	8.94	14.88	8.8	10.8	10.55	25.65	188.19	588.96	394.35	39.42	10.33	8.48	1309.33
1993	7.78	11.07	6.83	24.14	32.58	65.16	352.1	795.05	538.46	79.39	18.68	10.34	1941.57
1994	7.54	5.79	6.68	11.54	10.8	23.76	208.31	440.09	400.07	40.39	12.1	12.1	1179.17
1995	10.79	10.28	8.15	35.05	13.6	25	188.44	544.38	192.78	19.06	8.3	7.62	1063.45
1996	13.42	9.88	7.46	29.15	53.09	207.3	574.8	1108.4	351.2	32.07	14.18	3.81	2404.76
1997	10.33	7.74	7.88	13.2	10.52	37.03	147.5	326.4	76.99	24.09	24.68	14.97	701.32
1998	12.57	8.58	27.19	20.06	28	62.27	386.51	1621.8	442.63	105.66	21.11	13.49	2749.87
Mean	11.12	12.04	17.92	23.54	21.00	41.82	256.61	610.56	330.62	44.54	20.58	11.358	1401.70

Reservoir capacity determination partial procedure.

To select ten consecutive years which relatively show dry periods, plot the annual average flow for the entire data.

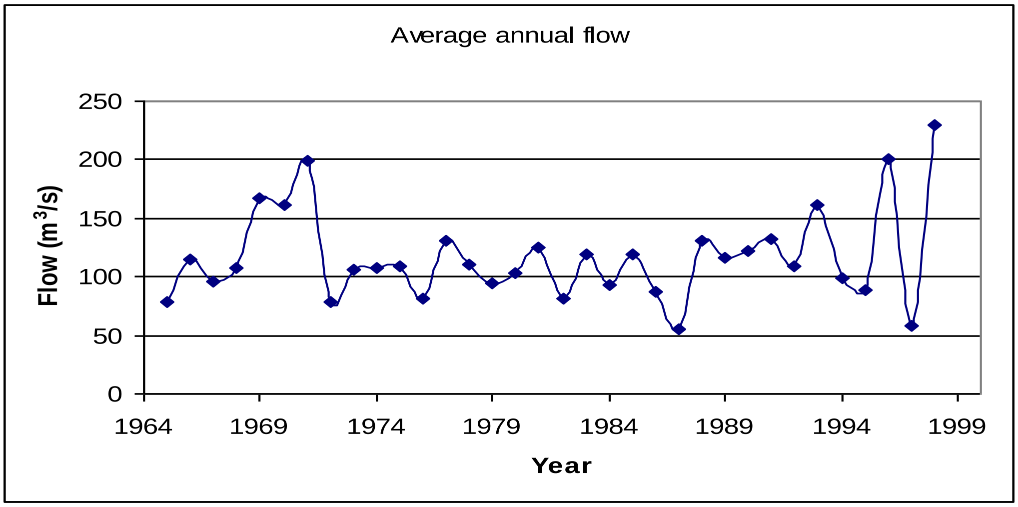


Figure 1-7 Annual average flow

From the plot it can be seen that the flow record from 1978-1988 can be taken as a critical period and be used for further analysis and determination of reservoir capacity.

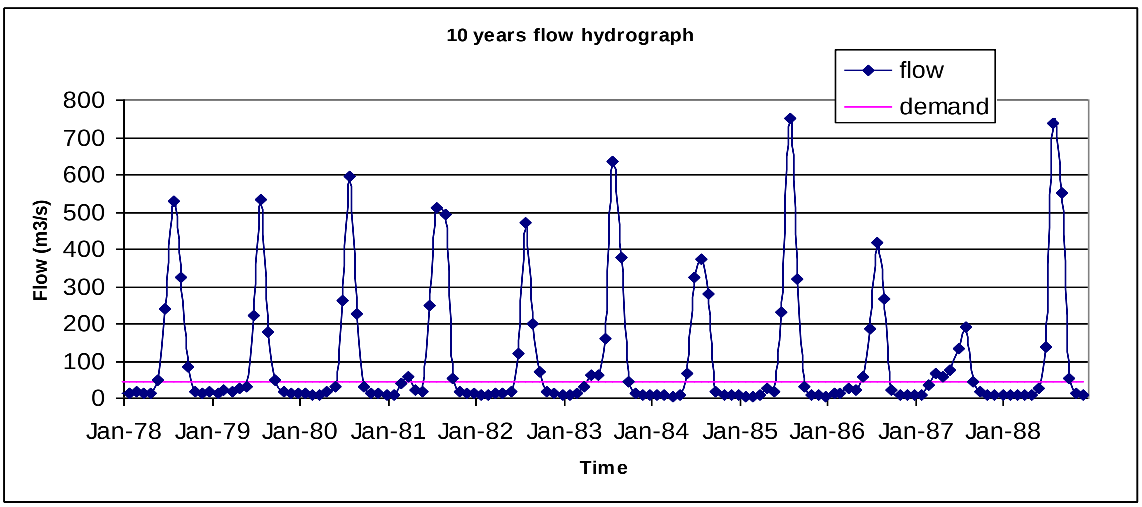


Figure 1-8 Critical dry period hydrograph from data series

The 10 year flow hydrograph indicates a dry period to be used in the mass curve analysis. This dry period is from January 1986 to January 1989. For this specific period the mass curve is plotted as shown below.

To know the capacity of the reservoir needed to meet the demand, the reservoir can be considered as full or empty at the beginning. Assuming that the reservoir is full at the beginning, move the demand line in a way that it forms tangent line that do not intersect the inflow mass curve of the previous period. The point at which the tangent line crosses the inflow mass curve is where the reservoir fills again. If the line never intersects the curve this means that the reservoir will never be full with in the time frame considered.

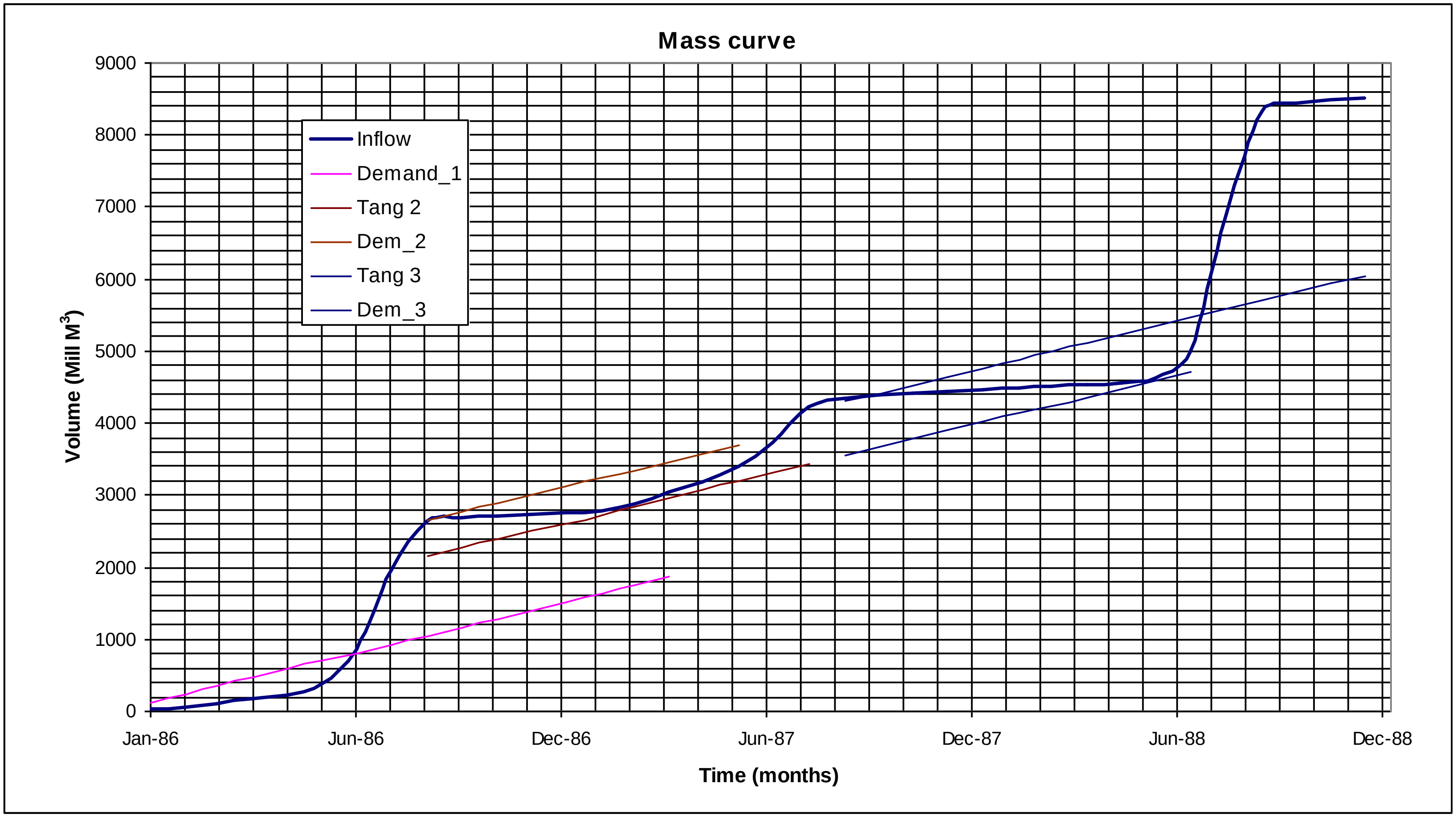


Figure 1-9 Mass Curve diagram for reservoir capacity

1.6 Environmental, Social, Economical and Political investigation

The environmental, economic and other socio-political issues associated with reservoir development must in all instances be acknowledged at the outset and fully addressed thereafter. This is especially important in the case of the larger high-profile projects and all other, large or lesser, sited in environmentally or politically sensitive locations.

Environmental impacts and other socio-political considerations can extend across a diverse spectrum of issues. Socio-political considerations may range from population displacement, with consequent economic impacts, to the preservation of cultural or heritage sites; from the consequences of sedimentation and/or of changing flood regimes to altered patterns of disease.

It is necessary to examine the complex relationships between human society and its surrounding environment, paying particular attention to issues relating to the local and regional environment, especially the use and misuse of water resources and the policies governing resource use.

Various types of surveys based on functional and technical requirements should be carried out for selecting a site for the dam and reservoir. Functional suitability of a site is governed by the balance between its natural physical characteristics and the purpose of the reservoir. Catchment hydrology, available head and storage volume etc. must be matched to operational parameters set by the nature and scale of the project served. Technical acceptability is dictated by the presence of a satisfactory site for a dam, the availability of materials suitable for dam construction, and by the integrity of the reservoir basin with respect to leakage. To these must be added an assessment of the anticipated environmental consequences of construction and operation of the dam.

1.7 Location criteria for dam and spillway site

While selecting a site for a dam the following points should be taken into consideration

i. The dam should be as near as possible to the area to be served, hence conveyance cost and water losses will be minimized.
ii. Foundation area should be impervious and should be able to support the weight of the dam.
iii. The topography of the dam and reservoir sites should permit maximum storage of water at minimum cost.
iv. Materials of construction should be available in sufficient quantity and good quality at a reasonable distance.
v. The value of property and land which will be submerged by the reservoir has to be as small as possible.
vi. The cost of relocating roads, buildings etc. should be as small as possible
vii. The cost of stream diversion and dewatering the site should be as small as possible
viii. Transportation facilities and accessibility of the site
ix. Availability of suitable sites for construction equipment and camps
x. The safety of the structure.

While selecting a site for spillway the following points should be taken into consideration

i. The spillway must be a part of the dam itself (not for embankment dams) or it may be located at a separate site,
ii. The location depends on the foundation and the topography of the area. Good rock foundation is always desirable and the topography should permit easy diversion of the flood waters passing over the spillway, back into the original stream channel.

1.8 Dam site investigation

The items of investigation required mainly for a dam structure are listed below:

1. General planning and preparation of location maps: before undertaking actual investigation it is necessary to prepare location maps indicating
   - likely dam and spillway site,
   - proposed relocation of approach roads,
   - quarry sites for construction material,
   - stream gauging stations,
   - proposed camp site,
   - existing utilities like lines of communication, transmission lines, rail/road communication,
   - Other important features.
2. Hydrologic investigation:
   - collection and analysis of stream flow and precipitation records,
   - assessment of available yield, estimation of flood peaks,
   - determination of spillway capacity and
   - Ground water studies.
3. Topographic survey:
   - Detailed survey for the dam site covering sufficient area on the u/s and d/s as well as above the likely height of the dam on both the banks,
   - Detailed survey for areas proposed for constructing spillway, diversion tunnels, outlets, power houses etc
   - Preparation of detailed maps to various scales based on the data collected.
4. Surface geologic investigation:
   - identification of boundary and nature of deposits and overburden;
   - the characteristic, structure, strike of rock beds;
   - Shape and magnitude of folds and fault zones.
5. Subsurface or foundation exploration:
   - sinking open pits,
   - drilling holes,
   - driving shafts and drifts,
   - Geophysical prospecting using latest techniques.
6. Seismic surveys.
7. Construction material survey:
   - location and estimate of quantities of available construction material,
   - estimates need to be supported by laboratory tests to determine suitability of various materials for construction of dam and other structures.

1.9 Data collection

The collection of relevant data is the first state in the formulation of a project.

i. Physical data

General plan

- obtain a general plan of the catchment and project area
- carryout limited surveys to include additional information in this plan
- the plan must include the dam site, spillway site, irrigable area, catchment area of the stream, locality to be supplied with potable water (if any) map scale may vary from 1:1000 – 1:10,1000
- features to be included in the map

   - contour at 0.5 – 1.5m interval
   - location of existing works, if any, affected by the proposed development,
   - Proposed relocation of roads, railways, transmission lines, etc.
   - Additional transportation facilities such as access roads, cableways, etc. required for the execution of the project.
   - Location of stream gauging station, water sampling and meteorological stations, if any, in the area.

Larger plans for dam and spillway sites

   - This should be in the scale of 1:500 – 1:1000 with contours as small as possible
   - The plans should show the following
      - Over-banks
      - Location of elevation of all features such as buildings, roads, etc.
      - Location and number of test pits and boreholes

ii. hydrologic data

- data needed:
- Monthly flow, momentary peak flow of a stream at or near the dam site
- Annual sediment load
- If available, the following information is necessary
   - Maximum observed flood level in the system
   - Report on damage caused by flooding
   - Data for estimating water demand
      - # of people to be served
      - approximate maximum and minimum daily water demand
      - irrigation water requirement
      - Other water requirements (industries, livestock, etc.)

iii. Meteorological Data

- Average monthly temperature
- Average monthly rainfall
- Maximum recorded storm intensities
- Annual rate of evaporation

iv. Geologic Data: geological map of the entire catchment area.

- Dam and Spillway sites
- Geologic map is essential
- Subsurface investigation should be carried out by an experienced geologist
   - geologic sections of selected dam site
   - quality of the overburden material if an earth dam is to be built
- shearing strength of the overburden material and the dam material
- permeability of the overburden material
- quantity and quality of the overburden material for construction purposes
- Presence, orientation and extent of joint planes, seams, caverns, and solution channel.
- Strength of the rock (Hardness, etc.) if a concrete dam is to be built.
- Reservoir site
- Check the existence of cracks
- Banks should be checked for possible zones of landslides

v. Earthquake information on past seismic activity in the area should be obtained

vi. Agricultural data

- For irrigation purposes, the following data are essential for determining of water requirement

- The size of the area to be irrigated
- Soil structure
- Possible types of crops
- Soil types

vii. Material type

- Soil, sand and stone (for aggregate and riprap) are needed in good quality and sufficient quantity for dam construction. Therefore, possible quarry sites for these materials should be identified with in a reasonable distance from the construction site.
- Selection of a suitable burrow area is influenced by
- Thickness of top organic soil which has to be discarded
- Content of organic matter in the rest of the soil
- Quantity of oversize cobbles which could have to be removed from the soil
- Rock for aggregate and riprap has to pass the standard tests of specific gravity absorption, abrasion soundness etc.

viii. Miscellaneous data

- Erosion in the catchment area
- Identify exception sources of erosion
- Transport
- Existing facilities and rates
- Local labor
- Availability and rates

1.10 Site Requirement

For the budget allocated and the data collected the site selected for the reservoir and dam need to be further evaluated for detail design. The detail design will of course necessitate additional data collection.

1.11 Stages in Project Planning and Implementation

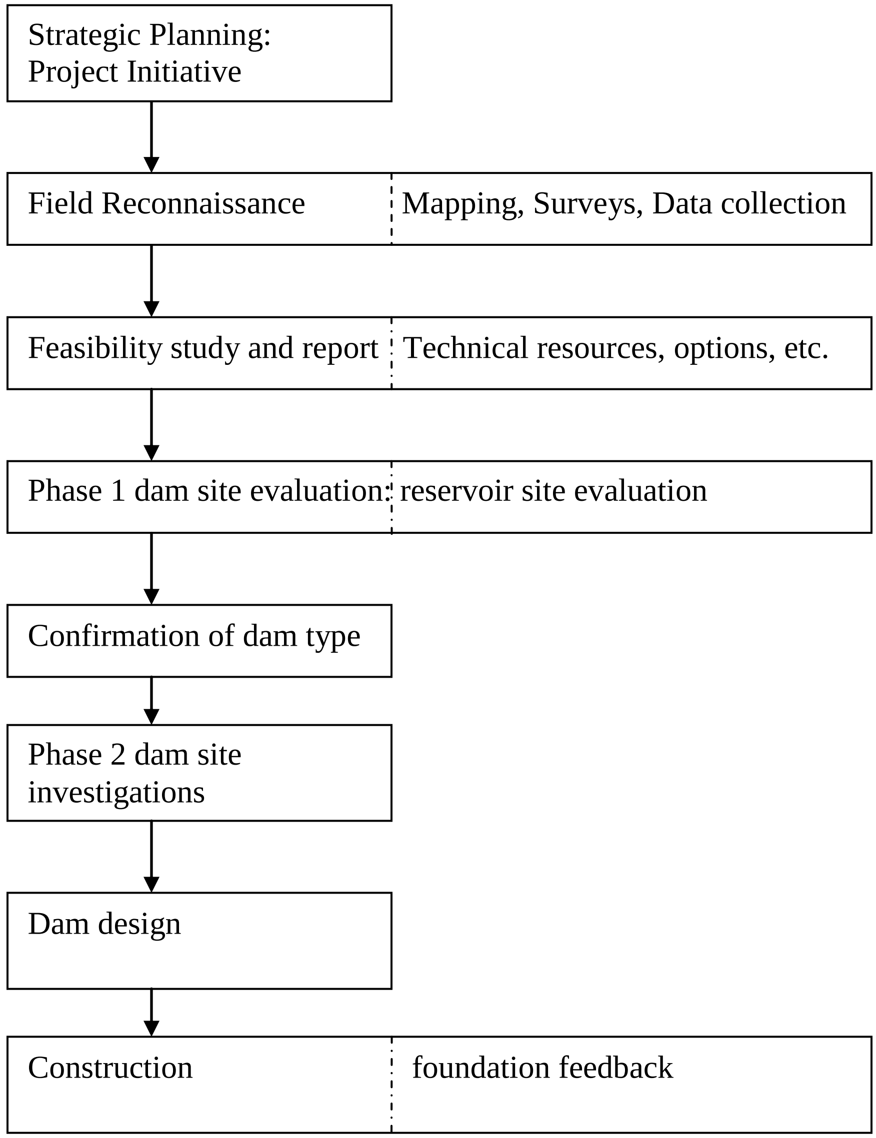


Figure 1-10 Stages in dam site appraisal and project development (P. Novak, 2001)

2 Foundations of Dams and their Treatment

2.1 General

Foundation is part of the area under and adjacent to a dam, i.e., bottom and abutments.

A sound foundation

- Must have sufficient strength to withstand crushing and to prevent sliding,
- Must be tight enough to prevent excessive leakage and to reduce uplift as much as possible.
- Must not be damaged by overflow discharge and discharge from outlet works.

Foundations may be classified as:

- Rock foundations
- Earth foundations
    - Foundations of coarse grained material (sand and gravel)
    - Foundations of fine grained materials (silt and clay)

2.2 Rock foundation

2.2.1 General

In the strength and stability calculations rock foundations are considered to be homogeneous, continuous and isotropic but actually the rock as well as earth foundations are heterogeneous, anisotropic, consisting of rocks of different properties and are divided by various cracks, i.e. foundations are never continuous.

In general, rock foundations present no problem of bearing capacity and settlement even though the foundation mass has smaller strength and large deformability than its composing rock.

Defects of rock foundations:

- Presence of seams, fissures, cracks or faults that have usually resulted in erosive leakage, excessive loss of water and sliding.
- Presence of weathered zone (surface rock) or crushed zone that have usually resulted in separate foundation.

2.2.2 Foundation treatment

Treatment of foundation, if it is necessary, consists of grouting cracks and tectonic zones and infilling of weak portions with concrete, in strengthening broken-up parts using different connecting arrangements and structures.

Weathered portion (surface rock) has to be excavated and removed. Excavation has to be deep enough to give a firm ‘toe hold’ to the dam.

Consolidation of foundation:

Grouting is carried out to consolidate fissured or cracked foundations (consolidation grouting) by a grout that is prepared properly as a mixture of water and cement with admixtures of rock flour, bentonite, etc. Grouting is usually started with a mixture of cement and water in the proportion 1:5 and gradually thickened to 1:1.

Grouting hole: Depth =15 m

Spacing= 3 to 6 m on centers

Grout pressure = 3.5 kg/cm^2 (=350 KPa)

Execution starts with holes drilled and grouted from 12 to 25 m apart; then, intermediate holes are drilled and grouted.

Allowable stress:	Granite	: 4.0 – 7.0 KPa
	Limestone	: 2.5 – 5.5 KPa
	Sandstone	: 2.5 – 4.0 KPa

2.2.3 Measures against leakage

Leakage through rock foundations can be prevented by making grout curtain or trench filled with concrete.

Grout curtain: High pressure holes drilled relatively deep and near the u/s foundation of dam at close intervals and grouted under pressure (depth to be determined by water pressure test).

Depth: In hard rock = 30 – 40 % of dam height

In poor rock = as much as 70 % of dam height.

Tentative spacing = 1.5m on centers

Grout Pressure = 0.25 Kg/cm^2 per meter depth below the surface

For small dams, one row of grouting holes may be sufficient. No grouting is required for detention dams. Hot asphalt is used for sealing openings of large size containing running water.

Trench filled with concrete: Preferable if it can be done economically.

Treatment of faults, shear joints, etc.:

Optimum depth of back filling = 20% of dam height

Estimation of optimum depth as per USBR recommendation,

$d = 0.0066\ bh - 1.5$ for $h \leq 46$ m

$d = 0.3\ bh + 1.5$ for $h \geq 46$ m

Where: d = depth of excavation of weak zone below ground surface at adjoining sound rock in m.

h = height of dam above average foundation level general

b = width of wet zone

Preparation of rock and dam interface

The rock and dam interface must be prepared to obtain reliable interlocking and long contact length in the flow direction.

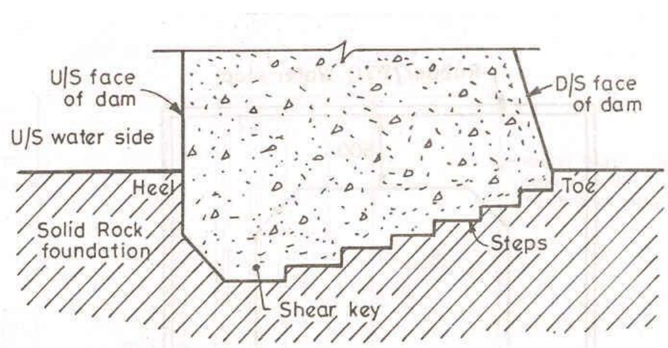


Figure 2-1 Profile of a typical interface

2.2.4 Drainage

Drainage is provided to relieve uplift pressure at base of dam. It is provided by a line of drilled holes d/s from the grout curtain. The holes are connected to drainage gallery to carry the seepage to the tail water.

The reduction in uplift pressure in a properly working drainage gallery can be estimated as,

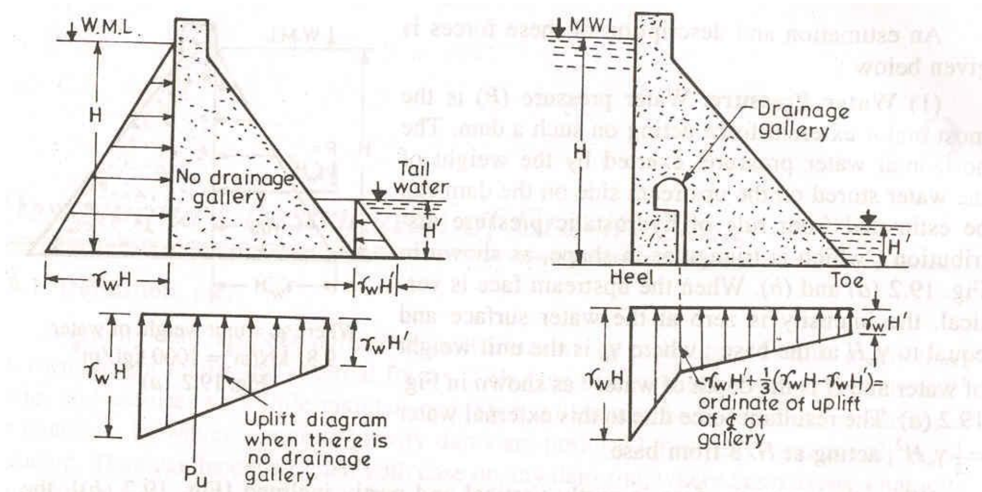
$$\Delta p_U = \frac{2}{3}\gamma_w (H - H')$$

Where ΔP_U = reduction of pressure at the drainage

γ_w = unit weight of water

H = u/s water head

H' = tail water head



a. No drainage b. Drainage gallery is provided

Figure 2-2 Uplift pressure distribution with and without drainage gallery

Net pressure available at the gallery becomes

$$\Delta p_{Ug} = \gamma_w H - \frac{2}{3}\gamma_w (H - H')$$

$$= \frac{1}{3}\gamma_w (H + 2H')$$

2.2.5 Stability of Dams and Strength of Rock Foundation

Loss of stability of dam and displacement thereof may occur due to:

i. sliding, when its contact with the foundation is disturbed or due to cracks in the foundation when inadmissible tensile and tangential stresses appear;
ii. overturning, when its contact with the foundation is disturbed;
iii. destruction of rock mass of foundation under the action of stresses appearing in it.

2.3 Earth foundation

2.3.1 General

Earth foundation may be classified as:

Foundations of coarse-grained material (gravel and sand)

Foundations of fine-grained material (silt and clay).

In preparation of earth foundations, the objectives are to prevent: crushing, sliding, excessive seepage under the dam, piping, and scouring by water flowing over the dam.

Because of the high cost of treatment of earth foundations, gravity dams on earth foundations are limited in height to 20m.

2.3.2 Gravel and sand foundation

Gravel and sand foundations are alluvial in origin. The following two basic seepage problems are encountered in using these foundations:

excessive loss of water

large seepage force

Extent of treatment to reduce the effect of the aforementioned problems depends on:

The purpose of the dam (seepage quantity is of little concern in a detention dam).

The necessity of the downstream release

Regardless of the quantity of seepage, adequate measures have to be provided to reduce the danger of piping.

2.3.3 Estimation of seepage amount

A rough estimation of the amount of seepage could be made using the Darcy's equation

$$Q = kiA$$

Where: Q = rate of seepage (m^3/s)

k = Permeability (Hydraulic conductivity) of the foundation material (m/s).

i = hydraulic gradient

$= \Delta H/\Delta L$ ΔH = upstream and downstream sections head difference (m).

ΔL = length of seepage path (m)

A = gross area of foundation through which flow takes place (m^2).

A better estimation of seepage quantity can be made by flow net analysis of the foundation. Using flow net technique,

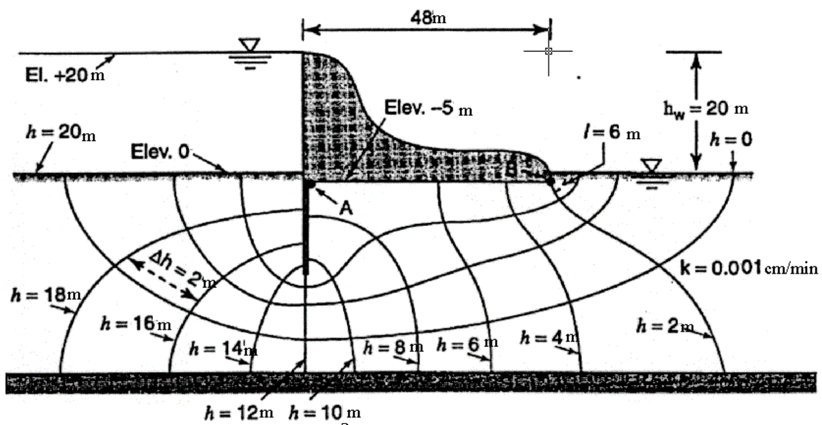
$$q = k\,\Delta H\,N_f / N_p$$

Where: q = seepage quantity per unit length ($m^3/s/m$)

N_f = number of flow channels

N_p = number of potential drops

Example: For the ogee spillway with sheetpiling cutoff shown below:



a. Compute the seepage in m^3/min.; and

b. Calculate the uplift force acting on the base of the dam.

To compute the seepage

Alternative I.

$\Delta h = 2m$, $H = 20m$

$k = 0.00001m/min$ [Hydraulic conductivity of pervious foundation]

$L = 48m$ [bottom length of the spillway]

$I = \Delta h/\Delta l = 2/6 = 0.33$

$q = KIA$ [Darcy's Equation]

$= 0.00001 \text{ m/min} * 0.33 * 24 \text{ m}^2/\text{m}$

$= 8.0 \times 10^{-5} \text{ m}^3/\text{min}$ Per meter length of structure

Alternative II

$H = 20m$ [head difference of head and tail water level]

$N_f = 4$ [number of Flow channels]

$N_d = 10$ [number of Equipotential lines]

$$q = kH \frac{N_f}{N_d}$$

$$q = kH \frac{N_f}{N_d} = 0.00001 \ m/\min * \ 20 * \frac{4}{10}$$

$$= 8.0 * 10^{-5} m^3 / \min/ \ m \text{ length}$$

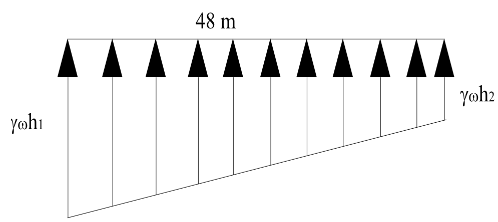
Uplift force on the dam

At point A $h1 \approx 7.5m$

At point B $h2 = 2m$

Uplift Pressure

$$P_u = \gamma_w \left[\frac{h1 + h2}{2}\right] = 10\left[\frac{7.5 + 2}{2}\right] = 47.5 KPa$$



Uplift force $F_u = P_u \times A = 47.5 * 48 = 2280.0$ kN per meter length of the structure.

2.3.4 Piping

Seepage forces

Seepage forces are developed as a result of friction between the seeping water and the walls of the particles.

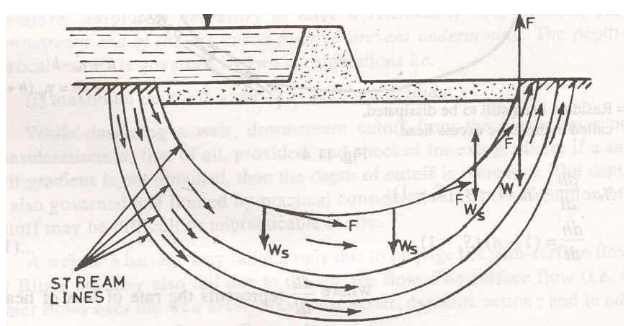


Figure 2-3 Seepage below a dam in pervious foundation.

Considering Figure 2-3, at the upstream side F increases Ws and tends to hold the soil particles in position. At the downstream side F decreases Ws and tends to lift the soil particles. If F >Ws, the soil particles would be floated out and thus erosion progresses backwards along the flow lines until a "Pipe" is formed resulting in loss of large quantities of water and soil particles and ultimate collapse of the foundation.

Piping is the movement of materials from the foundation caused by the velocity of the seeping water as it comes out from the soil below the dam. The danger of piping exists at any point when the pressure of seeping water is greater than the weight of the soil above that point.

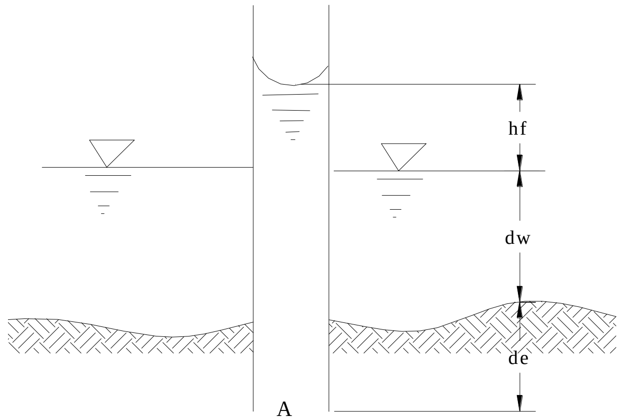


Figure 2-4 Illustration of seepage.

Consider the illustrative diagram of seepage shown in Figure 2-4.

Upward pressure force at A

$$A = \gamma_W (d_e + d_w + h_f)$$

Downward force at A = Weight of soil + Weight of water above A

$$W_A = \gamma_{ss} d_e + \gamma_W (d_e + d_w)$$

where γ_{ss} = submerged unit weight of the soil,

For equilibrium

$$\gamma_W (d_e + d_w + h_f) = \gamma_{ss} d_e + \gamma_W (d_e + d_w)$$

$$h_f / d_e = (\gamma - \gamma_w) / \gamma_w$$

$h_f/d_e = i_a$ (actual friction gradient)

$(\gamma-\gamma_w)/\gamma_w = i_c$ (critical gradient)

The factor of safety against piping is computed as

$S_f = i_c/i_a$

A value of $S_f \geq 4$ is usually considered in design.

2.3.5 Uplift pressure and control of seepage

Seepage quantity can be reduced by the following methods:

i. Using soil of low permeability for the body of the dam
ii. Placing core in earth structure and cut-off in the foundation
iii. Increasing the seepage path by employing upstream blanket

Cut-off is a core of impermeable material placed in the foundation. It may be of impervious soil (clay) in a cut-off trench; masonry (usually concrete); sheet piling (limited to foundations of silt, sand and fine aggregate), and grout curtain.

Cut-off penetrating upto the impervious layer is called complete cut-off. When properly constructed, it reduces seepage to a negligible amount.

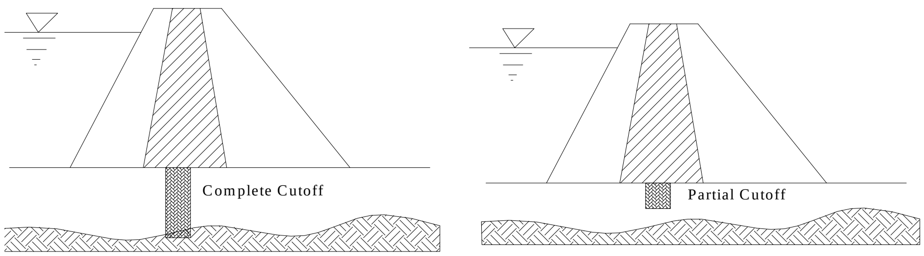


Figure 2-5 Complete and partial Cut-off

Cut-off penetrating only part of the pervious foundation is called partial cut-off. Its action is similar to an obstruction in a pipe. The flow across it is reduced because of the loss of head due to the obstruction. Cut-off extending through 50 % of the distance to the impervious stratum will reduce the seepage by only 25 %. An 80 % cut-off reduces the seepage to 50 %. Thus, partial cut-off is less effective in reducing seepage. However, it reduces danger of seepage along a line of contact of foundation and dam particularly when there is a differential settlement. It is effective in intercepting horizontal cracks and seams in rock foundation.

An adequate width of cut-off for a small dam may be determined by

$$W = h - d$$

Where: W = bottom width of cut-off trench

h = reservoir head above ground surface, and

d = depth of cut-off trench excavation below ground surface.

Upstream blanket

It is a layer of impervious soil placed on the foundation upstream from the structure. For earth dams it extends to the impervious core. It increases the length of the seepage path and thus reduces hydraulic gradient and quantity of seepage.

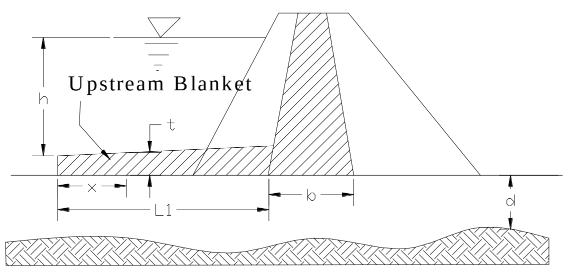


Figure 2-6 Upstream Blanket.

Length of blanket: the length of blanket L_1 required to reduce seepage to a required quantity can be determined from flow net analysis.

$$L_1 = (khd - PQ)b / (PQ)$$

Where: L_1 = length of upstream blanket

k = mean horizontal permeability coefficient of the pervious stratum

h = gross head on impervious upstream blanket

d = depth of pervious stratum

P = fraction to which Q is desired to be reduced by means of the blanket.

Thickness of blanket at a distance x from the upstream toe of blanket, the thickness t can be computed as:

$$t = (k_2/k_1)(L_1/d)x$$

Where t = thickness of blanket at the point under consideration

x = distance from the point under consideration to upstream toe of the blanket
k_1 = average permeability of stratum
k_2 = permeability of blanket
L_1 = length of blanket from upstream toe to impervious section, and
d = depth of pervious stratum

For normal conditions:

t = 1.5 – 3.0 meters

$L_1 = 10h$

In case of fine sand or silty foundation;

$L_1 = 15h$

Upstream apron

It can be of RC or impervious earth blanket. Differential settlement may crack the junction between apron and dam. A filter layer with clay blanket helps to remedy this danger.

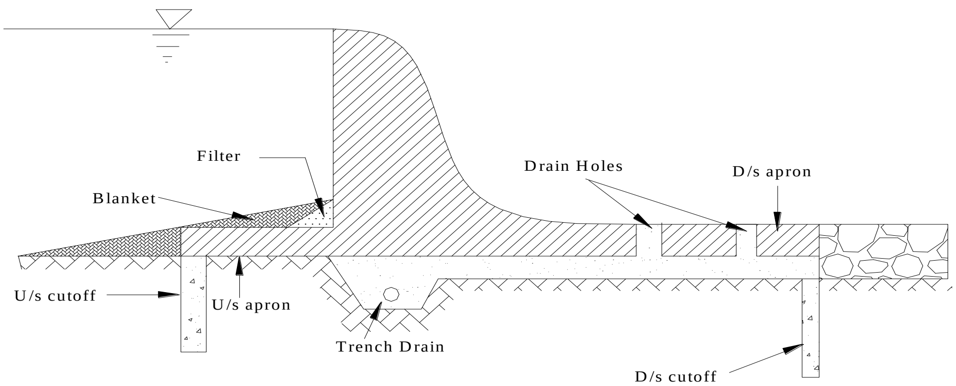


Figure 2-7 Aprons, Cut-offs, drains and blankets.

Downstream apron: It helps to increase the path of seepage, but its primary purpose is to balance the uplift pressure.

Downstream cut-off: A short downstream cut-off helps to keep the point of flow concentration (i.e. high gradient) well within the soil mass where it is protected by the weight of the soil above.

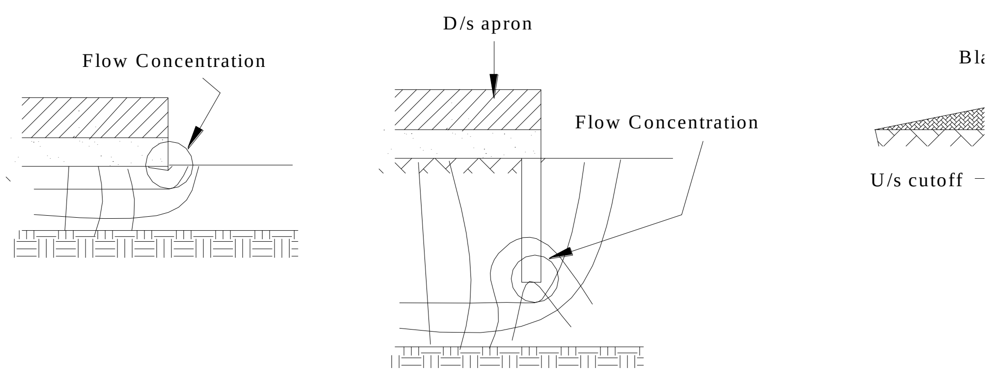


Figure 2-8 Regions of flow concentration.

Internal drainage (horizontal drainage layers and filters): It is effective in controlling excess pressure and exit gradients. It serves to short circuit the seepage. Various arrangements are possible

Trench drain

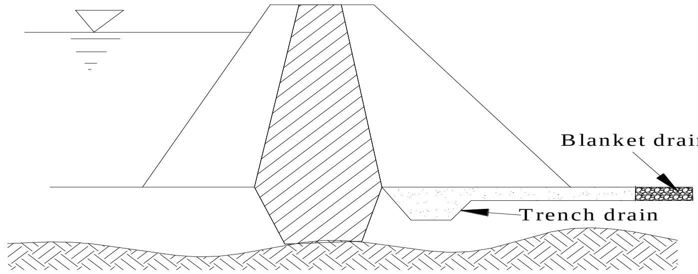


Figure 2-9 Trench and Blanket drains

Toe drain: it is used when the downstream shell is so pervious that it forms a drain.

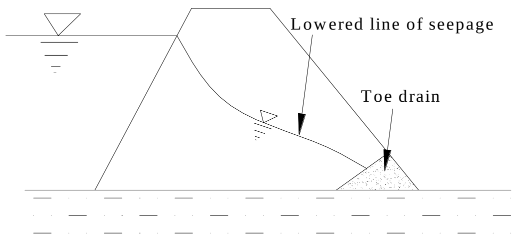


Figure 2-10 Toe drain.

Relief wells: These are holes or wells employed in masonry structures downstream from the cut-off and in the downstream apron where uplift is likely to cause a blow out. They serve to concentrate the seepage and reduce internal pressure.

Internal drain and relief wells have the disadvantage of increasing seepage quantity. They all need protective filters, thus, permitting the free drainage of water but preventing the movement of soil particles.

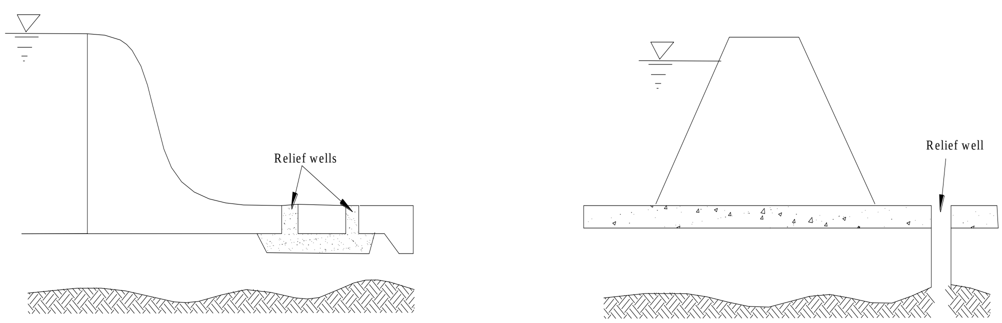


Figure 2-11 Relief Wells and Relief holes.

2.3.6 Theories of Seepage Flow

Whenever a hydraulic structure is founded on a pervious foundation, it is subjected to seepage of water beneath the structure, in addition to all other forces to which it will be subjected when founded on an impervious rock foundation. The concepts of failure of hydraulic structures due to subsurface flow were introduced by Bligh, on the basis of experiments and the research work conducted after the failure of Khanki weir, which was designed on experience and intution without rational theory.

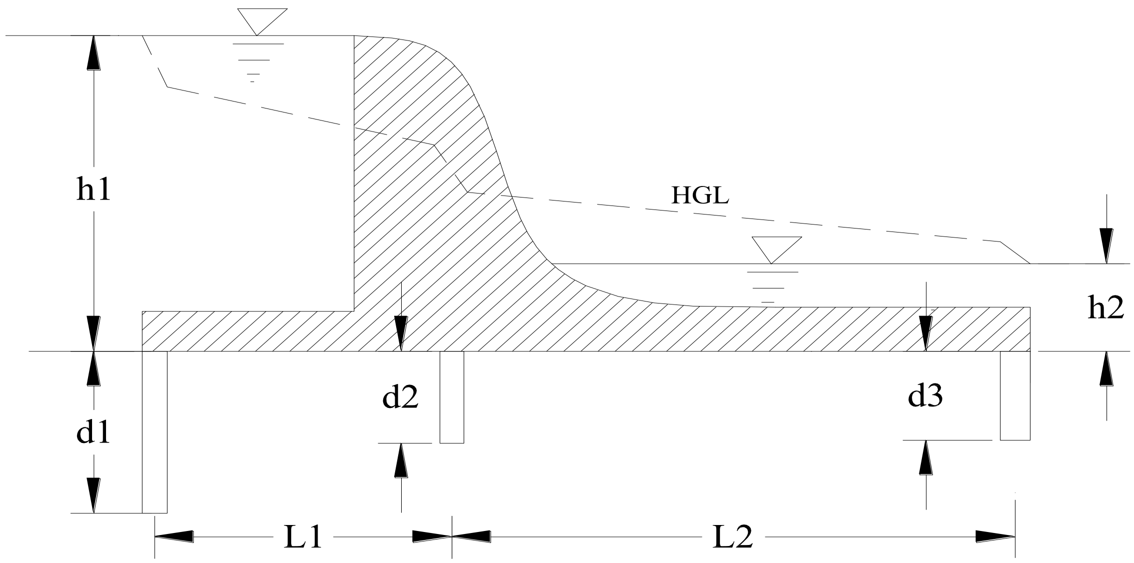


Figure 2-12 Bligh's and Lane's creep

Bligh's creep theory

The seeping water followes the outline of the contact surface of the structure and foundation soil. The length of the path traversed is the creep length [L]. The loss of head is assumed to be proportional to the creep length.

Refering to Figure 2-12:

The total head loss between upstream and downstream [H_L] = h_1- h_2

Creep Length [L] = $2d_1 + L_1 + 2d_2 + L_2 + 2d_3$

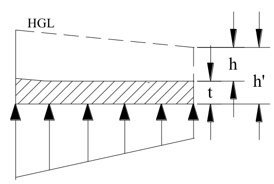
Head loss per unit length of hydraulic gradient = H_L /L

Safety against piping is ensured by providing sufficient creeplength,

$L = CH_L$ Where: C – Bligh's coefficient for the soil

$C = L/H_L$ C is reciprocal of the hydraulic gradient.

The HGL represents the residual uplift water head at each point.



h' = h + t

Uplift pressure = $\gamma_w h'$

Downward pressure =

2.3.7 Uplift pressure and seepage under masonry structures on pervious foundations

For designing low concrete dams on pervious foundations, the weighted creep theory, as developed by Lane is suggested for safety against uplift pressure and piping. According to this theory, the flow will concentrate along the line of creep, i.e., along the line of contact of the dam and cut-offs with the foundation.

After testing the theory on more than 200 dams on pervious foundations, the following conditions were drawn

The weighted creep length of a cross-section of a dam is the sum of the vertical creep distances (steeper than 45^0) plus 1/3 of the horizontal creep distance (less than 45^0).

The weighted - creep ratio is the weighted-creep length divided by the effective head.

When filter drains and relief wells are not used, the full Lane's weighted - creep ratio is to be used (case a). Where drains are properly provided (but no flow net analysis is made), use 80% of Lane's weighted creep ratio (case b).

Where both drains and flow net analysis are used, use 70% of weighed-creep ratio (case c). Take minimum weighted-creep ratio (WCR) = 1.5

The pressure to be used in design may be estimated by assuming that the drop in pressure from headwater to tail water along the contact line of the dam and foundation is proportional to weighted-creep length

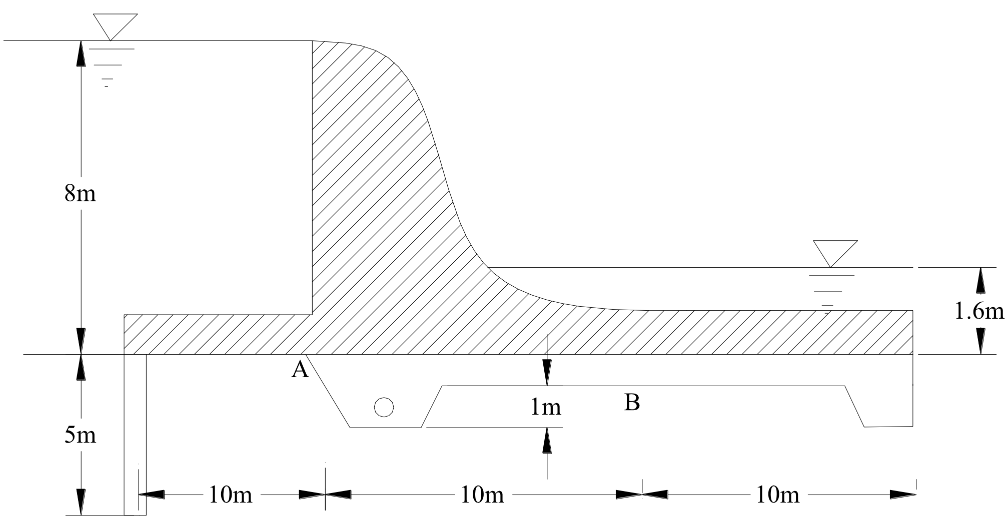
Table 2-1 Lane's recommended WCR for different materials

Material	Case a Lane 100%	Case b Lane 80 %	Case c Lane 70%
Very fine sand and silt	8.5	6.8	6
Fine sand	7.0	5.6	4.9
Medium sand	6.0	4.8	4.2
Coarse sand	5.0	4.0	3.5
Fine gravel	4.0	3.2	2.8
Medium gravel	3.5	2.8	2.5
Coarse gravel (including Cobbles)	3.0	2.4	2.1
Boulders with stone, cobbles, and gravel	2.5	2.0	1.8
Soft clay	3.0	2.4	2.1
Medium Clay	2.0	1.6	1.5
Hard clay	1.8	1.5	1.5
Very hard clay and hard pan	1.6	1.5	1.5

Example

For the dam section shown below determine

i. the type of the foundation on which the dam section shown below may be judged safe;
ii. the magnitude of the uplift force for the section A to B



Solution

Weighted creep length = 5 + 5 + 4*1 + (10 + 10 + 10)/3 = 24 m

Net head on structure = Head water – Tail water = 8-1.6 = 6.4 m

Weighted creep ratio = 24 / 6.4 = 3.75

According to Lane's ratios, the dam(spillway) would be safe on clay or on medium gravel and coarse gravel. With properly provided drains and filters, it may be considered safe on fine gravel foundations [case b]

$$\text{Uplift at point A} = 6.4 - \frac{(5+5+10/3)}{24} * 6.4 + 1.6 = 4.44\,m$$

$$\text{Uplift at point B} = 6.4 - \frac{(5+5+1+1+10/3+10/3)}{24} * 6.4 + 1.6 = 3.02$$

$$\text{Total Uplift on section AB} = \frac{(4.44+3.02)}{2}\gamma_w = 36.591\,k\text{N/m crest length}$$

2.3.8 Silt and clay foundation

Such foundation materials are sufficiently impervious. Thus seepage is not a problem. The main challenge is bearing capacity.

Methods of foundation treatment are based on:

Soil type

Location of water table

State of compactness of the soil

Methods of treatment

a. For saturated fine-grained soils

- Soil of low shearing strength is removed. This is practical for thin layers of soft soil overlying firm material if the cost of excavation and refill is less than the cost of special investigation and provision of flatter slopes of embankment.
- Drainage is provided to the foundation to permit increase of strength during construction.
- Flatter slopes for the embankment are used to reduce the magnitude of the average shearing stress along the potential surface of sliding. This is the most practical solution. For recommended slopes, refer to "Design of small dams, USBR" sec 129.

b. For relatively dry foundations

For a given void ratio, an impervious soil has greater bearing capacity in the unsaturated condition than in the saturated condition. Hence dry silt and clay foundations are generally satisfactory for small dams.

Soils like loess (very loose wind deposited soils) exhibit sufficient strength at low water content. Such low density soils are subject to large settlements when saturated by reservoir and may result in failure of the dam by differential settlements that may cause rupture of the impervious portion or by considerable reduction in free board resulting in overtopping.

Treatment here depends on the compression characteristics of the soil.

If appreciable post construction settlements are not expected upon saturation, little foundation treatment is necessary.

Remove organic top soil

Provide a key trench

Provide a toe drain so that the foundation at the downstream toe will not saturate

When appreciable post construction settlements upon saturation are expected, measures to minimize the settlements should be adopted.

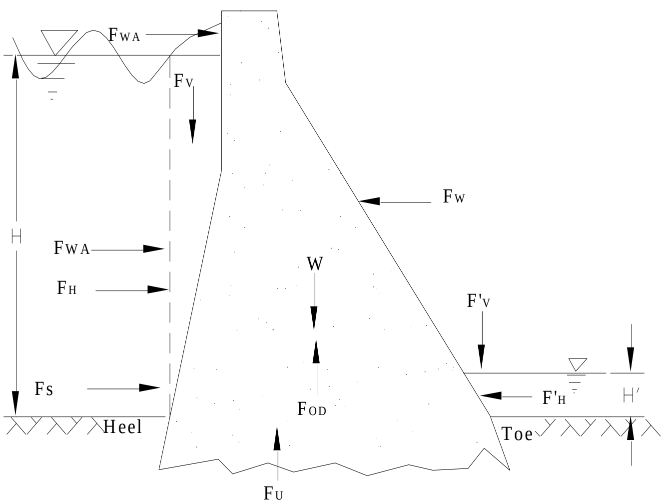
3 Concrete Gravity Dam

3.1 General

Basically, gravity dams are solid concrete structures that maintain their stability against design loads from the geometric shape and the mass and strength of the concrete. Generally, they are constructed on a straight axis, but may be slightly curved or angled to accommodate the specific site conditions. Gravity dams typically consist of a non-overflow section(s) and an overflow section or spillway.

3.2 Forces acting on gravity dams

The structural integrity of a dam must be maintained across the range of circumstances or events likely to arise in service. The design is therefore determined through consideration of the corresponding spectrum of loading conditions. In all foreseeable circumstances the stability of the dam and foundation must be ensured, with stresses contained at acceptable levels and watertight integrity essentially unimpaired.



Where:
H = Head water depth
H' = Tail Water depth
F_{WA} = Wave pressure force
F_H = Horizontal hydrostatic force
F_S = Silt/sediment pressure force
F_{EQ} = Earthquake/Seismic force
F_W = Wind pressure force
F_H' = Tail water hydrostatic force
W = Weight of dam
F_{OD} = Internal pore water pressure
F_U = Uplift pressure force [base of dam]
F_V = Weight of water above dam [u/s]
F_V' = Weight of water above dam [d/s]

Figure 3-1 Representation of typical loads acting on Gravity dam

3.2.1 Water pressure

Water pressure is the force exerted by the water stored in the reservoir on the upstream and the water depth at the tail of the dam.

i. External water pressure load

External water pressure can be calculated by the law of hydrostatics according to which in a static mass of liquid the pressure intensity varies linearly with the depth of liquid and it acts normal to the surface in contact with the liquid. For the non-overflow section of the dam water pressure may be calculated as follows and for the overflow portion the loading will be discussed in section six of the course.

F_H = horizontal component of hydrostatic force, acting along a line 1/3 H above the base
$= \frac{1}{2} \gamma_w H^2$
γ_w = Unit weight of water (=10 kN/m^3)
Fv = Vertical component of hydrostatic pressure
= Weight of fluid mass vertically above the upstream face acting through the center of gravity of the mass.

ii. Internal water pressure (Uplift Pressure)

Internal water pressure is the force exerted by water penetrating through the pores, cracks and seams with in the body of the dam, at contact surface between the dam and its foundation, and with in the foundation. It acts vertically upward at any horizontal section of the dam as well as its foundation and hence it causes a reduction in the effective weight of the portion of the structure lying above this section.

The computation of internal pressure involves the consideration of two constituent elements, i.e,

- Hydrostatic pressure of water at a point
- The percentage C, area factor, of the area on which the hydrostatic pressure acts

Both these elements are discussed below.

Hydrostatic pressure

In practice dams are usually provided with cut-off walls or grout curtains to reduce seepage and drain to relieve pressure downstream from the cutoff. Actually cutoff and grout curtains may not be perfectly tight and hence fail to dissipate the head ($h_1 - h_2$)

Usually a distribution like 1-2-3-4 is used with 3-4 a straight line as shown in Figure 3-3. Opinions about the value of uplift reduction factor, ζ (Zeta), are varied, the tendency is to take:

ζ = 0.85 (for normal loading cases)
ζ = 1.00 (for exceptional loading cases like earthquake)

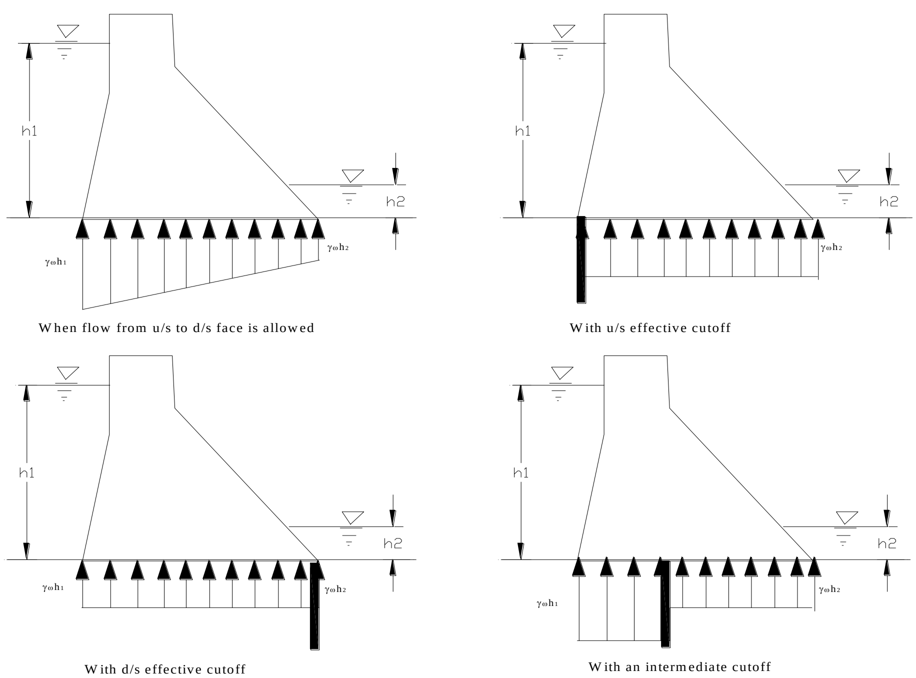


Figure 3-2 Uplift pressure distribution for perfectly tight cutoff walls.

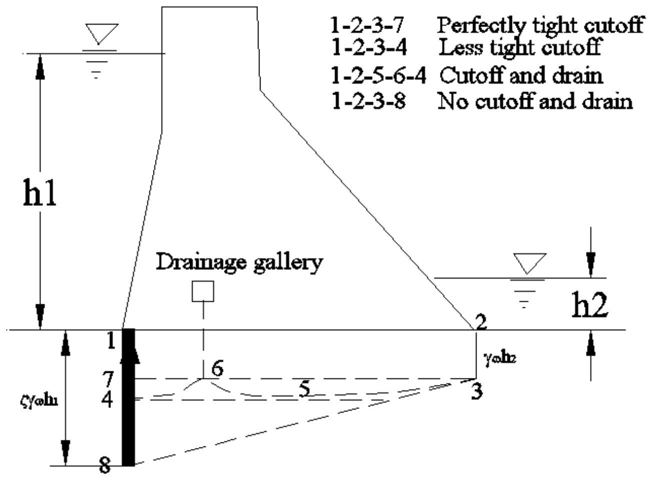


Figure 3-3. Uplift pressure distribution for less tight cutoff.

Uplift area factor, C

The value of area factor for concrete has been determined experimentally by several investigators. However, for the foundation rock the value of area factor is not determinable experimentally and hence the same has been estimated on the basis of theoretical considerations.

Some of the earliest investigators recommended, for both concrete and rock, a value of area factor ranging from one third to two-thirds of the area to be considered as effective area over which the uplift pressure acts. However, Harza, Terzaghi and Lelivakey have indicated that, for both concrete and rock, the value of area factor is nearly equal to unity.

Table 3-1Values suggested for uplift area factor are

Value of C	Suggested by
0.25 to 0.40	Henry
1.00	Maurice Levy
0.95 to 1.00	Terzaghi

As such the present practice followed in the design of dams is that the uplift pressure is assumed to act over 100 percent of the area with in the body of the dam as well as its foundation. Hence, under all conditions, the value C = 1.00 is recommended.

3.2.2 Wight of Structure

For a gravity dam the weight of the structure is the main stabilizing force, and hence the construction material should be as heavy as possible.

Structure self weight is accounted for in terms of the resultant, W, which acts through the centroid (center of gravity) of the cress-sectional area. The weight of the structure per unit length is

$$W = \gamma c * A$$

Where: γc is the unit weight of concrete

A is the cross-sectional area of the structure

The unit weight of concrete may be assumed to be 24 kN/m^3 in the absence specific data from laboratory test trials. For final designs the specific weights shall be based on actual test data. Where crest gates and other ancillary structures or equipments of significant weigh are present they must also be accounted for in determining the weight of the structure.

It is essential to make sure that the actual specific weight obtained for the construction material is more than or at least equal to that assumed in the design.

3.2.3 Earth and silt pressure

The gradual accumulation of significant deposits of fine sediment, notably silt, against the face of the dam generates a resultant horizontal force, F_s. The magnitude of this force in additional to water load, F_{WH}, is a function of the sediment depth, h_s, the submerged unit weight, $\gamma_{ss,}$ and the active pressure coefficient, $K_{a,}$ and is determined according to Rankine's formula.

$$Fs = ½ K_a\gamma_{ss} h_s^2$$

Where Ka = (1-sinϕ) / (1+sinϕ)

ϕ = angle of internal friction of material.

3.2.4 Wind pressure

When the dam is full, wind will act only on the downstream face, thus contributing to stability. When the dam is empty, wind can act on the upstream face, but the pressure is small compared to the hydraulic pressure of the water. Hence for gravity dams wind is not considered. For buttress dams, wind load on the exposed buttresses has to be considered.

3.2.5 Wave pressure and wave height

Wave exerts pressure on the upstream face. This pressure force, F_{wv} depends on fetch (extent of the water surface on which the water blows) and wind velocity. It is of relatively small magnitude and, by its nature, random and local in its influence. An empirical allowance for wave load may be made by adjusting the static reservoir level used in determining $F_{WV.}$ According to Molitor the following formula could be used to determine the rise in water level, h_w

$$h_w = 0.763 + 0.032\sqrt{vf} - 0.271 f^{1/4} \qquad \text{for } f < 32\,km$$

$$h_w = 0.032\sqrt{vf} \qquad \text{for } f \geq 32\,km$$

$$F_{wz} = 2.0\gamma_w h_w^2$$

where: h_w in meters

v wind velocity in km/hr and

f fetch in km

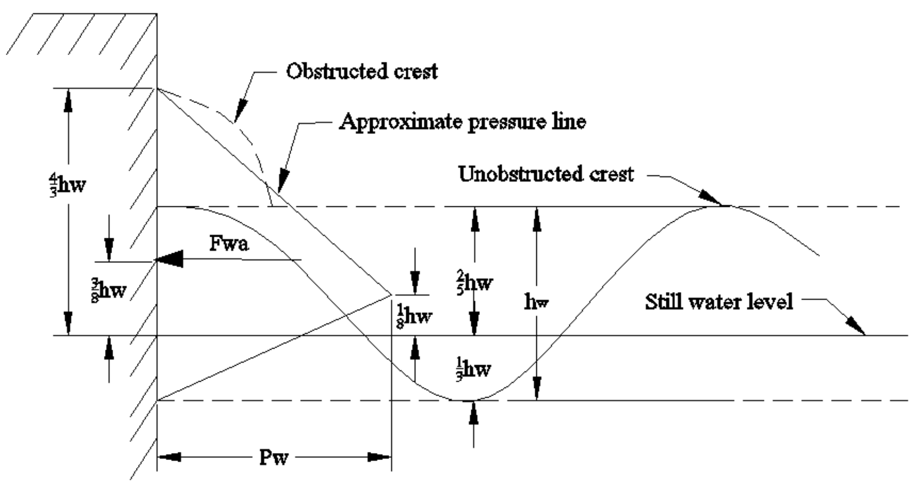


Figure 3-4 Wave configuration and wave pressure on a gravity dam

For high dams the wave pressure is small compared to other forces.

The point of application of F_{wv} can be taken as 3/8hw from the still water level.

The wave rides up higher on inclined dam faces as compared to the vertical one.

3.2.6 Earthquake forces

Dynamic loads generated by seismic disturbances must be considered in the design of all major dams situated in recognized seismic “high risk” regions. The possibility of seismic activity should also be considered for dams located outside those regions, particularly where sites in close proximity to potentially active geological fault complexes.

Seismic activity is associated with complex oscillating patterns of accelerations and ground motions, which generated transient dynamic loads due to the inertia of the dam and the retained body of water. For design purposes both should be considered operative in the sense least favorable to stability of the dam. Horizontal accelerations are therefore assumed to operate normal to the axis of the dam. Under reservoir full conditions the most adverse seismic loading will then occur when a ground shock is associated with:

1. horizontal foundation acceleration operating upstream, and
2. vertical foundation acceleration operating downward

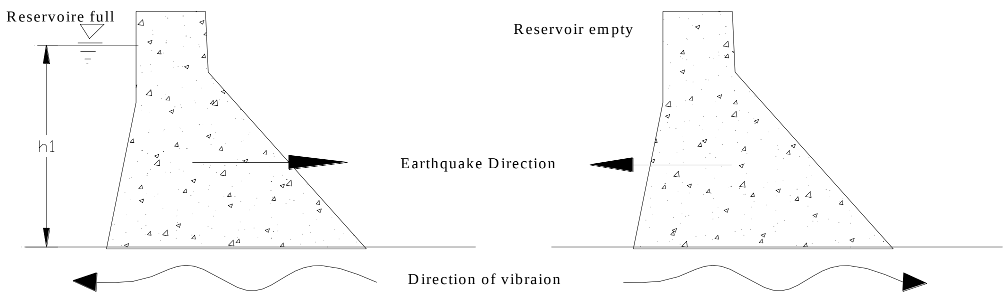


Figure 3-5 Direction of ground acceleration and the respective horizontal earthquake force on gravity dam

As a result of 1, inertia effects will generate an additional hydrodynamic water load acting downstream, plus a further inertia load attributable to the mass of the dam and also acting in a downstream sense. Foundation acceleration downwards, 2 above, will effectively reduce the mass of the structure. The more important recurring seismic shock waves have a frequency in the range 1-10Hz. Seismic loads consequently oscillate very rapidly and are transient in their effect. The strength of seismic event can be characterized by its magnitude and its intensity.

Ground motions associated with earthquakes can be characterized in terms of acceleration, velocity or displacement. Only peak ground acceleration, *pga*, generally expresses as a portion of gravitational acceleration, g, is considered in this course. It has been suggested that in general seismic events with a high *pga* of short duration are less destructive than seismic events of lower pga and greater duration.

The natural frequency of vibration, f_n, for a triangular gravity profile of height H (m) and base thickness B(m) constructed in concrete with an effective modulus of elasticity E=14GPa can be approximated as

$$f_n = 600\ B/H^2 \quad \text{(Hz)}$$

For a dam of H = 500m and B = 375m, f_n = 0.9 Hz. But the most important recurring seismic shock waves are in the order of magnitude of 1-10Hz. Hence resonance (the frequency of vibration of the structure and earthquake are equal) of an entire dam is unlikely and is not a series concern in design. But vulnerable portion of the dam should be detailed.

There are two methods to determine the seismic load on a dam

Pseudostaic (equivalent static load) method: inertia forces are calculated based on the acceleration maxima selected for design and considered as equivalent to additional static loads. This method generally is conservative and is applied to small and less vulnerable dams.

The acceleration intensities are expressed by acceleration coefficients α_h (Horizontal) and α_v(vertical) each representing the ratio of peak ground acceleration. Horizontal and vertical accelerations are not equal, the former being of greater intensity ($\alpha_h = (1.5 – 2.0\alpha_v)$.

Based on the vertical and horizontal acceleration, the inertial force will be

Horizontal force = $\pm\alpha_h$ * (static mass)

Vertical force = $\pm\alpha_v$ * (static mass)

Three loading cases can be used for the assessment of seismic load combination:

i. Peak horizontal ground acceleration with zero vertical ground acceleration
ii. Peak vertical ground acceleration with zero horizontal acceleration
iii. Appropriate combination of both (eg. Peak of the horizontal and 40-50% of the vertical)

Inertia forces

1. Mass of dam

Horizontal $F_{eqh} = \pm\alpha_h W$

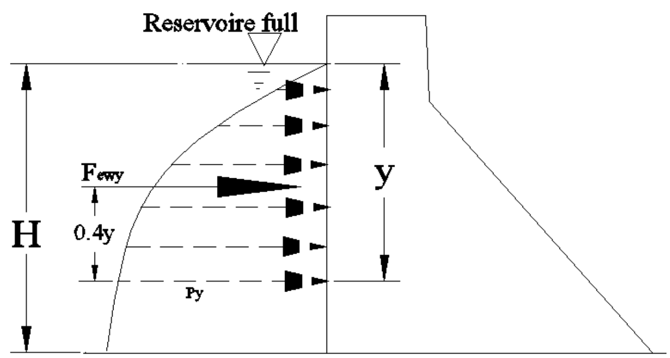
Vertical $F_{eqv} = \pm\alpha_v W$

2. Water body

As analyzed by Westerguard(1993)

$$P_y = k''\alpha_h \gamma_w \sqrt{H.y}$$

$$F_{ewy} = \frac{2}{3}\alpha_h \gamma_w y \sqrt{H.y}k''$$



where k” = earthquake factor for the water body

$$k'' = \frac{0.816}{\sqrt{1 - 7.75\left(\frac{H}{1000\,T}\right)^2}}$$

Where: T = period of earthquake

γ_w = in tone/m^3

H, y in meters

The force acts at 0.4y from the dam joint being considered.

For inclined upstream face of dam

$$P_y = k''\alpha_h \gamma_w \sqrt{H.y\cos\phi}$$

where ϕ is the angle the face makes with the vertical.

The resultant vertical hydrodynamic load, F_{ewv}, effective above an upstream face batter or flare may be accounted for by application of the appropriate seismic coefficient to vertical water load. It is considered to act through the centroid of the area.

$$F_{ewv} = \pm\alpha_v F_v$$

Uplift load is normally assumed to be unaltered by seismic shock.

Dynamic analysis: the dam is idealized as a two dimensional plane-strain or plane-stress finite element system, the reservoir being regarded as a continuum. The foundation zone is generally idealized as a finite element system equivalent to a visco-elastic half space. The complexities of such an approach are evident, and take it outside the scope of this course.

3.3 Load combination for Design

The design of a gravity dam is based on the most adverse combination of the loads/forces acting on it, which includes only those loads having a reasonable probability of simultaneous occurrence. The combination of transient loads such as those due to maximum flood and earthquake are not considered because the probability of occurrence of each of these phenomena is quite low and hence the probability of their simultaneous occurrence is almost negligible. Thus for the design of gravity dams according to Indian Standard is specified as the following load combination:

I. *Load combination A (construction condition or empty reservoir condition):* Dam completed but no water in the reservoir and no tail water.

II. *Load combination B (Normal operating condition):* Full reservoir elevation (or top of gates at crest), normal dry weather tail water, normal uplift, ice and uplift (if applicable)

III. *Load combination C (Flood Discharge condition):* Reservoir at maximum flood pool elevation, all gates open, tail water at flood elevation, normal uplift, and silt (if applicable)

IV. *Load combination D* - Combination A, with earthquake.

V. *Load combination E* - Combination A, with earthquake but no ice

VI. *Load Combination F* - Combination C, but with extreme uplift (drain inoperative)

VII. *Load Combination G* - Combination E, but with extreme uplift (drain inoperative)

3.4 Reaction of the foundation

The foundation should provide the required reaction to the resultant force for the dam to be stable.

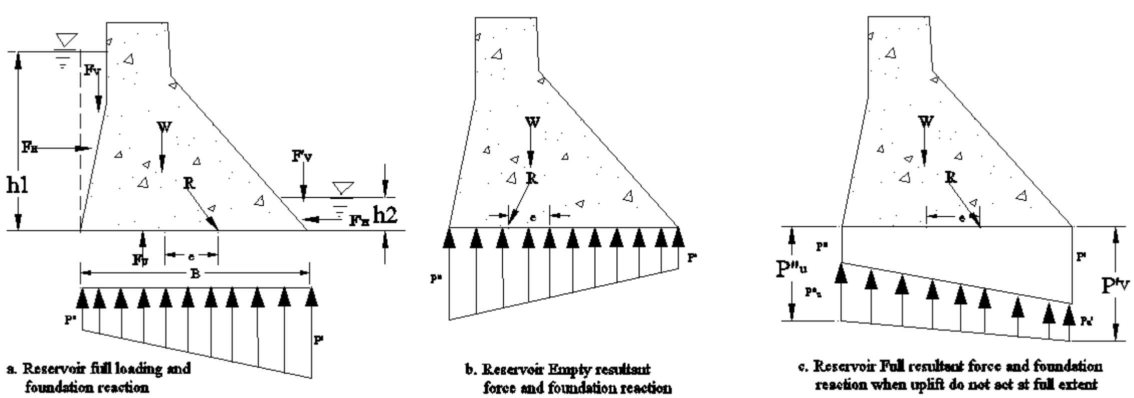


Figure 3-6 Foundation reaction for reservoir full and empty loading

For full reservoir

$$P', P" = \frac{\Sigma W}{A} + \frac{Mc}{I}$$
$$= \frac{\Sigma W}{B} \pm \frac{6\Sigma W.e}{B^2}$$
$$= \frac{\Sigma W}{B}\left(1 \pm \frac{6e}{B}\right)$$

For empty reservoir $P', P" = \frac{\Sigma W}{B}\left(1 \mp \frac{6e}{B}\right)$

where: B = Width of the base of the dam section
e = eccentricity
ΣW = Sum of vertical forces including uplift

$$e = \frac{\Sigma M_{VCG} + M_{HCG}}{\Sigma W}$$

Requirements for stability

A masonry of plain concrete dam must be free from tensile stress, i.e. neither *P'* nor *P"* shall be negative, or

$e \leq B/6$ (law of the middle third)

To limit compressive stress with in the dam body use:

P', P" if uplift always acts to the fullest extent.

P'_v , $P"_v$ if uplift does not act always.

Horizontal forces must be resisted both by shear and friction in the dam joint or in the foundation.

3.5 Rules Governing the Design of Gravity Dams

The following are basic assumptions that should be considered relative to the design of important masonry/concrete dams.

1. The rock that constitutes the foundation and abutments at the site is strong enough to carry the forces imposed by the dam with stresses well below the elastic limit at all places along the contact planes.
2. The bearing power of the geologic structure along the foundation and abutments is great enough to carry the total loads imposed by the dam without rock movements of detrimental magnitude.
3. The rock formations are homogeneous and uniformly elastic in all directions, so that their deformations may be predicted satisfactorily by calculations based on the theory of elasticity, by laboratory measurements on models constructed of elastic materials, or by combinations of both methods.
4. The flow of the foundation rock under the sustained loads that result from the construction of the dam and the filling of the reservoir may be adequately allowed for by using a somewhat lower modulus of elasticity than would otherwise be adopted for use in the technical analyses.
5. The base of the dam is thoroughly keyed into the rock formations along the foundations and abutments.
6. Construction operations are conducted so as to secure a satisfactory bond between the concrete and rock materials at all areas of contact along the foundation and abutments.
7. The concrete in the dam is homogeneous in all parts of the structure.
8. The concrete is uniformly elastic in all parts of the structure, so that deformations due to applied loads may be calculated by formulae derived on the basis of the theory of

elasticity or may be estimated from laboratory measurements on models constructed of elastic materials.

9. Effects of flow of concrete may be adequately allowed for by using a somewhat lower modulus of elasticity under sustained loads than would otherwise be adopted for use in technical analyses.
10. Contraction joints are properly grouted under adequate pressures, or open slots are properly filled with concrete, so that the dam may be considered to act as a monolith.
11. Sufficient drains are installed in the dam to reduce such uplift pressures as may develop along areas of contact between the concrete and rock materials.
12. Effects of increases in horizontal pressures caused by silt contents of flood waters usually may be ignored in designing high storage dams, but may require consideration in designing relatively low diversion structures.
13. Uplift forces adequate for analyzing conditions at the base of the dam are adequate for analyzing conditions at horizontal concrete cross sections above the base.
14. Internal stresses caused by natural shrinkage and by artificial cooling operations may be adequately controlled by proper spacing of contraction joints.
15. Internal stresses caused by increases in concrete temperature after grouting are beneficial.
16. Maximum pressures used in contraction joint grouting operations should be limited to such values as may be shown to the safe by appropriate stress analyses.
17. No section of the Ethiopia may be assumed to be entirely free from the occurrence of earthquake shocks.
18. Assumptions of maximum earthquake accelerations equal to one tenth of gravity are adequate for the design of important masonry dams without including additional allowances for resonance effects.
19. Vertical as well as horizontal accelerations should be considered, especially in designing gravity dams.
20. During the occurrence of temporary abnormal loads, such as those produced by earthquake shocks, some increases in stress magnitudes and some encroachments on usual factors of safety are permissible.
21. Effects of foundation and abutment deformations should be included in the technical analyses.
22. In monolithic straight gravity dams, some proportions of the loads may be carried by twist action and beam action at locations along the sloping abutments, as well as by the more usually considered gravity action.
23. Detrimental effects of twist and beam action in straight gravity dams, such as cracking caused by the development of tension stresses, may be prevented by suitable construction procedure.
24. In monolithic curved gravity and arch dams, some proportions of the loads may be carried by tangential shear and twist effects, as well as by the more usually considered arch and cantilever actions.
25. The distribution of loads in masonry dams may be determined by bringing the calculated deflections of the different systems of load transference into agreement at all conjugate points in the structure.

The aforementioned assumptions are rephrased as rule/guideline for design of concrete gravity dam as described below:

Rule1: Location of the resultant: No tension in any joint of the dam under all loading conditions (i.e. for full and empty reservoir). Thus, resultant of all forces (including uplift) must intersect the joint within the middle third.

Rule2a: Resistance to sliding when shear is neglected: the tangent of the angle between the vertical and the resultant (including uplift) above horizontal plane shall be less than the allowable coefficient of frictional force 'f'. If empirical values are taken, factor of safety, $S_f = 2$.

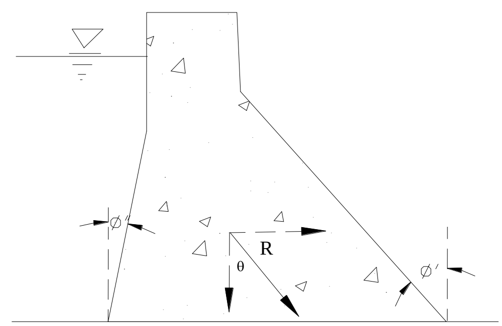
Table 3-2 Some values of Coefficient of friction *f*

Surface	***f***
Masonry on masonry or masonry on good rock or concrete on concrete	0.75
Concrete or masonry on gravel	0.50
Concrete or masonry on sand	0.40
Concrete or masonry on clay	0.30

However, the value of *f* for specific cases should be obtained by test

For foundation on earth, $\frac{\Sigma P}{\Sigma W} = \tan\theta = \frac{f}{S_f}$ S_f is taken as 3

Rule 2b: Resistance to sliding when shear is considered



The total friction resistance to sliding on any joint plus the ultimate shearing strength of the joint, must exceed the total horizontal force above the joint by a safe margin, i.e.

$$\Sigma P \leq \frac{f\Sigma W + r.S_n.A}{S_{sf}}$$

Where: S_n – ultimate shearing strength of material

S_{sf} = shear friction factor of safety

A = cross sectional area of joints

r = ratio of average to maximum shearing strength

Recommended values $S_{sf} = 5$, r = 0.5

$rS_{sf} = 200$ to $500 t/m^2$

While analyzing resistance to sliding, first compute tanϕ and if tanϕ > f apply Rule 2b. In that case, S_{sf} should equal or exceed the allowable value.

Rule 3: Governing compressive stresses: P'_v, or P''_v (maximum vertical stresses) are not the maximum stresses in the structure. The maximum stresses occur at the end joints, or inclined planes, normal to the face of the dam.

Maximum stress for downstream face, reservoir full:

$$P_i^{'} = P_v^{'}(1 + \tan^2\phi')$$

Maximum stress for upstream face, reservoir full

$$P_i^{''} = P_v^{''}(1 + \tan^2\phi^{''})$$

The inclined compressive stresses in the dam and foundation shall not exceed the allowable values.

Ultimate stress, σ'_c = 14 to 31 MPa (after 28 days curing)

Working stress $\sigma_c = \sigma'_c/6$

For foundation materials some indications for allowable stress are:

Limestone -------------200 to 350 t/m3

Granite -------------250 to 300t/m3

Rule 4: Governing internal tension: The dam shall be designed and constructed in such a manner as to avoid or adequately provide for tension on interior planes, inclined, vertical or horizontal.

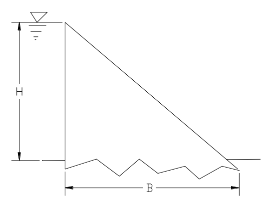
Rule 5: Margin of safety: all assumptions of forces acting on the dam shall be unquestionably on the safe side, all unit stresses adopted in design should provide an ample margin of safety against rupture and the shear-factors shall be considered.

Rule 6: Detail of design and methods of construction: all details shall support and confirm to the assumptions used in design; masonry should be of quality suited to the stresses adapted, protection against overflowing water shall be ample.

3.6 Theoretical versus practical section of a dam

Considering only the two major forces acting on the dam, i.e. the weight of the dam and the hydrostatic water pressure, the required section of the dam for its stability will be a triangle of base width,

$$B = \frac{H}{\sqrt{s}}$$



Where: H = depth of water

s = specific gravity of concrete

For this section, the resultant will pass through the upper middle third point of the base when reservoir is empty and through the lower middle third point when the reservoir is full.

Practical section:

i. The pointed crest of the theoretical dam is unstable to resist shock due to floating objects.
ii. There is need for a free board
iii. There is also need for top width for a roadway

For practical section

i. Crest of the dam shall be a certain thickness depending on the height of the dam. For non-overflow dams, most economical crest width ≈ 14 % of the height (10 – 15 %) is normal.
ii. Free board is provided and usually 3-4% of the dam height is used as a maximum height of the free board.

3.7 Design procedure of gravity dams

3.7.1 Design methods

The various methods used for the design of concrete gravity dams are as follows:

1. Stability analysis method
    a. Gravity method.
    b. Trial load twist method

i. Joints keyed but not grouted
ii. Joints keyed and grouted

c. Experimental method
   i. Direct method
   ii. Indirect method
d. Slab analogy method
e. Lattice analogy method
f. Finite element method

2. Zoned (multiple-step) method of determining profile of dam
3. Single step method

Two procedures of design will be discussed in this course: – multiple-step method and single-step method.

3.7.2 Multiple step method of determining profile of gravity dam

This method deals with designing the dam joint by joint (block by block) beginning at the top and making each joint confirm to all gravity dam design requirements. The procedure results in a dam with polygonal face that may be smoothened up for appearance with no appreciable change in stability or economy. The multiple-step method is almost always used for the final design of dams with a height that does not encroach greatly on Zone V.

Zoning of high non-overflow dams

A high gravity dam may be divided into seven zones according to design and stability requirements. The characteristics and limits of these zones are described below.

Zone I: is a rectangular section from the top of the dam to the water surface. The resultant force passes through the mid-point of the base.

Zone II: is also a rectangular section and extends to a depth where the resultant in the reservoir full condition reaches the outer middle third point of the base.

Zone III: upstream face of the dam is vertical but the downstream face is gradually inclined so that the resultant in the reservoir full condition has exactly at the outer middle third point of the base. This zone extends to a depth where the resultant in the reservoir empty condition reaches the inner middle third point of the base.

Zone IV: in this zone both the upstream and downstream faces are inclined so that the resultant both in the reservoir full and empty conditions lie at the middle third point. The zone extends to a point where maximum permissible compressive stress is reached at the *toe* of the dam.

Zone V: the slope of the downstream face is further increased to keep the principal stresses within permissible limits. Resultant in the reservoir full condition is kept well within the middle third section. The resultant in the reservoir empty condition follows the upper middle third section. This zone extends to a depth where the stress at the *heel* of the section reaches the permissible limits in the reservoir empty case.

Zone VI: the slope of the upstream face is rapidly increased so as to keep the principal stress at the heel with in the permissible limits in the reservoir empty condition. The inclination of the downstream face should also be adjusted so that the principal stress at the toe does not exceed the maximum allowable stress. The resultants in both reservoir empty and full conditions lie *within* the middle third section. This zone extends to a point where the slope of the *downstream* face reaches 1:1. This normally happens when the dam is 80 to 90 meters high.

Zone VII: in this zone the inclination of both upstream and downstream faces increase with the height of the dam. Consequently, at some plane the value of $(1 + \tan^2\Phi)$ may become so great that the principal stress at the downstream face may exceed the allowable limit. If one reaches this zone during design, it is better to avoid it and start again with a fresh design with increased crest width and/or better quality concrete.

Zoning of overflow dams (Spillways)

Zone I: the resultant in the reservoir full condition is outside the middle third point both horizontal and vertical forces are existing. End of zone I is at a depth where resultant intersects downstream middle third point. Upstream face needs reinforcement to take tension.

Zone Ia: this is the zone below zone1. The end of zone Ia is established by the plane where only friction is sufficient to resist sliding.

Zone II: similar to zone II of non overflow dam with the only difference that the downstream face is inclined in overflow dams. The rest of the zones are similar to those of non-overflow dams.

3.7.3 Single Step Method

This method considers the whole dam as a single block. It is used for final design of very high dams that extend well beyond zone V. it can also be used with an accuracy of 2 to 4% on the safe side; for preliminary designs to obtain the area of the maximum section of the dam.

The dam designed by single step method has a straight downstream face. When extended it intersects upstream face at the headwater surface.

Consider the sketch given:

L = 10-15% of h_1

H_{10} = 2L (when earthquake is considered)

= 3L (when earthquake is not considered)

H_6 = 1.33L

When designing (analyzing) a dam in the single step method, the dam is considered as a single block; and dam dimensions are determined in such a way that rules of Zone IV are satisfied.

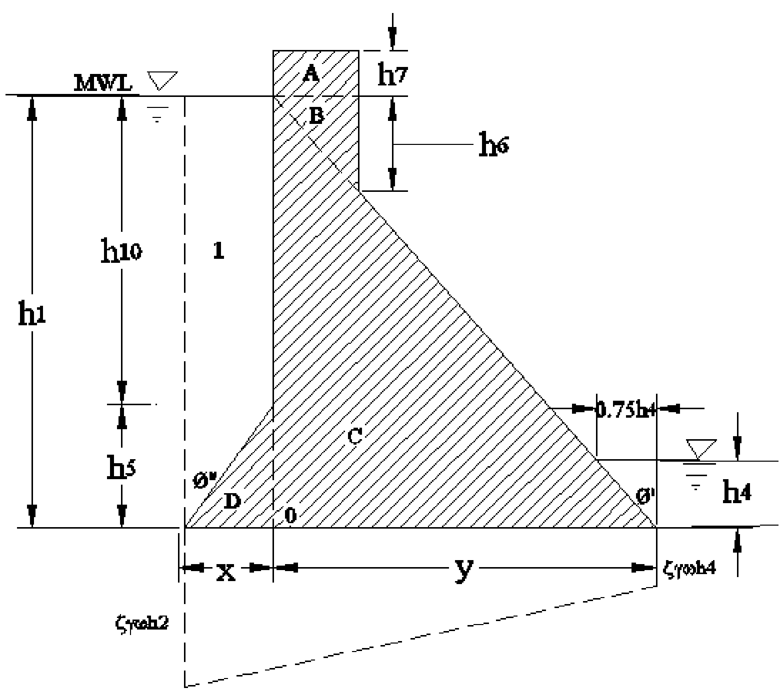


Figure 3-7 Gravity dam section relationship in single step method of design

Comparison of Single step and Multiple step design of gravity dam

- Dams of smaller height can be designed economically by Multiple step method
- High dams beyond zone IV are designed by Single step method so that convex curvature of downstream face and excessive flat slope of upstream face are avoided
- It may be economical to increase the concrete strength through the use of more expensive material, so that even a high dam designed by dividing it into only four zones, thus eliminating zone V and VI.

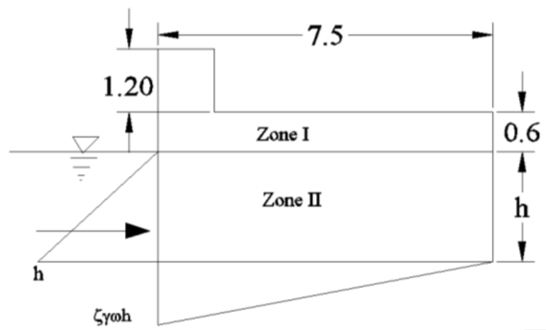
Design Example:

Design a non-overflow concrete gravity dam by the multiple-step method using the following data.

Item	Value	Item	Value
H_{max} (depth of headwater	60 m	*f*- Friction coeffnt.	0.75
h_e (spillway crest to MWL)	3 m	s_a	560 kPa
Tail water	None	s_{sf}	5
γ_c	24 kN/m^3	ζ	0.5
γ_w	10 kN/m^3	C	1
Minimum Top width	7.5 m	σ_c'	30 MPa
Earthquake and silt press	Ignore	F – Fetch length	6.4 km
		V	128 km/hr

Zone I

Determine the wave height by the empirical equations



$$h_w = 0.763 + 0.032\sqrt{vf} - 0.271 f^{1/4} \quad ;\text{for } f < 32\,km$$

$$h_w = 0.763 + 0.032\sqrt{128 * 6.4} - 0.271 * 6.4^{1/4}$$

$$= 1.25\text{m}$$

Rise of water wave = $1.33h_w$ = 1.66 m;

With an allowance of 0.14 m, free board = 1.8m

$$F_{wa} = 2.0\gamma_w h_w^2 = 2.0 * 10 * 1.25^2$$
$$= 31.25 \text{ kN/m}$$

Point of application = 3/8 *1.25 =0.47m above still water level.

Zone II

Taking moment about the outer middle third point of the base,

Line	Item	Description & dimension	Forces		Lever	Moment
			Horizontal	Vertical		
1	W0	Zone I: 0.6*7.5*24		108	1.25	135
2	W1	Zone II: 7.5 * h * 24		180h	1.25	225h
3	Wu	Uplift: 0.5*7.5*$\zeta\gamma_w$ h		18.75h	2.5	-46.9h
4	F_h	Water Pressure	5h^2		h/3	-1.67h^3
5	F_{wa}	Wave action	31.25		0.47 + h	-(14.7+31.2h)
		for h=				

M = 135 + 225h + 46.9h – 1.67h^3- (14.7 + 31.2h) = 0

By trial and error, h = 9.75m. Calculating actual values as follows for further check:

Line	Item	Description & dimension	Forces		Lever	Moment
			Horizontal	Vertical		
1	W0	Zone I: 0.6*7.5*24		108	1.25	135
2	W1	Zone II: 7.5 * h * 24		1755	1.25	2193.75
3	Wu	Uplift: 0.5B$\zeta\gamma_w$ h		-182.813	2.5	-457.03
4	Fh	Water Pressure	-475.31		3.25	-1544.76
5	Fwa	Wave action	-31.25		10.22	-319.38
		For h=9.75m	-506.56	1680.188		7.58

Check for sliding $\frac{\Sigma H}{\Sigma V} = \tan\theta = \frac{-506.56}{1680.12} = -0.3 < 0.75$;Friction alone is sufficient. (Safe!)

Stresses for Reservoir full:

$$P_V^{'} = \frac{\Sigma W}{B}\left(1+\frac{6e}{B}\right) = \frac{2\Sigma W}{B} = \frac{2*1680.2}{7.5} = 448.05$$

$$P_V^{''} = 0$$

$$\sigma_{c,all} = \sigma_{ult}/6 = 5000$$

$P_V^{'} << \sigma_{c,all}$ (safe!)

Stresses for Reservoir empty:

$$P_V^{'} = 0$$

$$P_V^{''} = \frac{\Sigma W}{B} = \frac{108+180*9.75}{7.5} = 248.4$$

$P_V^{''} << \sigma_{c,all}$ (safe!)

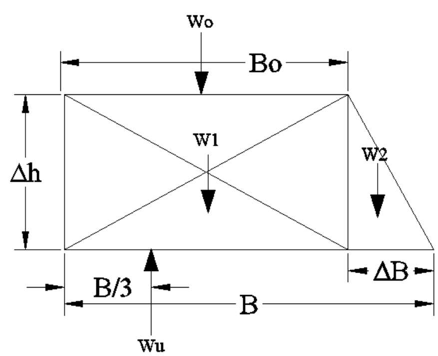
Zone III

Block I

ho=9.75m

Δh = 2.25m (step value)

Bo= 7.5 m



Line	Item	Description & dimension	Forces Horizontal	Forces Vertical	Lever	Moment
1	W0	Zone I & II: (0.6+9.75)*7.5*24		1863	3.75	6986.25
Trial I		ΔB_d =0.9	B = 8.4	B/3 =	2B/3=	
2	W1	Zone III: 7.5 *Δh * 24		405	3.75	1518.75
3	W2	0.5*0.9*2.25*24		24.3	7.8	189.54
		Reservoir Empty		2292.3	[3.79]	8694.54
4	Wu	Uplift: $0.5B\zeta\gamma_w h$		-252	2.8	-705.6
5	Fh	Water Pressure	720		4	2880
6	Fwa	Wave action	31.25		12.47	389.69
		Reservoir Full	751.25	2040.3	[5.52]	11258.63
Trial II		ΔB_d =0.84	B = 8.34	B/3 =	2B/3 =	
3	W2	0.5*0.84*2.25*24		22.68	7.78	176.45
		Reservoir empty		2290.68	[3.79]	8681.45
4	Wu	Uplift: $0.5*B\zeta\gamma_w h$		-250.2	2.78	-695.56
5	Fh	Water Pressure	720		4	2880
		Reservoir Full	751.25	2040.48	[5.52]	11255.58

Check for sliding $\frac{\Sigma H}{\Sigma V} = \tan\theta = \frac{751.25}{2040.5} = 0.368 < 0.75$; Friction alone is sufficient!

Check stresses for Reservoir full

$$P_V^{'}, P_V^{''} = \frac{\Sigma W}{B}\left(1 \pm \frac{6e}{B}\right) = \frac{2040.5}{8.34}\left(1 \pm \frac{6*1.35}{8.34}\right) = 482, 7.3$$

$$P_V^{'} = 482 \qquad P_V^{''} = 7.3$$

$$P_i^{'} = p_v^{'}(1 + \tan^2\phi^{'}) = 482(1 + 0.373^2) = 549$$

$$P_V^{''}, P_i^{'} << \sigma_{c,all} \qquad \text{(safe!)}$$

Check stresses for Reservoir empty

$$P_V^{'}, P_V^{''} = \frac{\Sigma W}{B}\left(1 \mp \frac{6e}{B}\right) = \frac{2290.7}{8.34}\left(1 \mp \frac{6*0.37}{8.34}\right) = 201, 347$$

$$P_V^{'} = 347 \qquad P_V^{''} = 201$$

$$P_i^{''} = P_V^{''}(1 + \tan^2\phi^{'}) = 201(1 + 0.373^2)$$

$$P_V^{''}, P_i^{'} << \sigma_{c,all} \qquad \text{(safe!)}$$

Continue with the design block by block until you arrive at the required dam height or the limit of Zone III, whichever comes first.

Zone IV

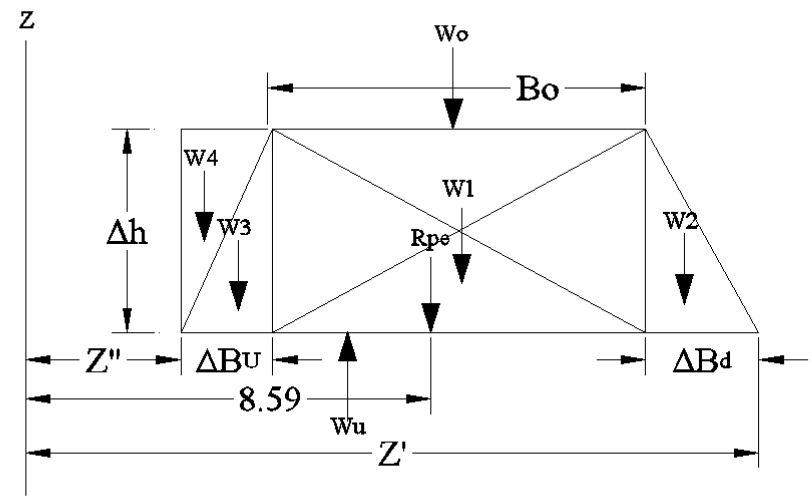
Block I

ho = 23m

Δh = 4m

h = 27m

Bo = 14.57m



Line	Item	Description & dimension	Forces: Horizontal	Forces: Vertical	Lever	Moment
1	W0	concreter above h = 23m		5230	7.88	41212.4
2	W1	4*14.57*24		1398.72	10.29	14392.83
	Trial I	$\Delta B_d = 3.0$				
3	W2	0.5*3*4.0*24		144	18.57	2674.08
4		Total Partial empty		6772.72	8.61	58279.31
		Estimation		Z'	20.57	
		2B/3=Z' - 8.6 = 20.57 - 8.6 =	11.98	B=	17.97	
		ΔBu =B - (Bo + ΔBd) =	0.4	Z" =	2.6	
		Z"+B/3 =	8.59	Z"+2B/3 =	14.58	
5	W3	0.5*0.4*4*24		19.2	2.87	55.1
6		Reservoir Empty		6791.92	8.59	58334.41
7	W4	Water column 0.4*25*10		100	2.8	280
8	Wu	Uplift: $0.5B\zeta\gamma_w h$		-1212.98	8.59	-10419.5
9	Fh	Water Pressure	3645		9	32805
10	Fwa	Wave action	31.25		27.47	858.44
		Reservoir Full	3676.25	5678.945	14.41	81858.39
	Trial II	$\Delta B_d = 2.8$				
11	W2	2.8*4*24*0.5		134.4	18.3	2459.97
12		Total Partial empty		6763.12	8.59	58065.2
		Estimation		Z'	20.37	
		2B/3=Z' - 8.59 = 20.37 - 8.59=	11.78	B=	17.67	
		ΔBu =B - (Bo + ΔBd) =	0.3	Z" =	2.7	
		Z"+B/3 =	8.59	Z"+2B/3 =	14.48	
13	W3	0.5*0.3*4*24		14.4	2.9	41.76
14		Reservoir Empty		6777.52	8.59	58106.96
15	W4	Water column 0.3*25*10		75	2.85	213.75
16	Wu	Uplift: $0.5B\zeta\gamma_w h$		-1192.73	8.59	-10245.5
17		Reservoir Full	3676.25	5659.79	14.44	81738.64

Check for sliding $\quad \frac{\Sigma H}{\Sigma V} = \tan\theta = \frac{3676.25}{5659.79} = 0.65 < 0.75$ Friction alone is sufficient !

Check stresses for Reservoir full

$$P_V^{'}, P_V^{''} = \frac{\Sigma W}{B}\left(1 \pm \frac{6e}{B}\right) = \frac{5659.8}{17.67}\left(1 \pm \frac{6 * 2.91}{17.67}\right) = 629, 3.84$$

$$P_V^{'} = 629 \qquad\qquad P_V^{''} = 3.8$$

$$P_i^{'} = p_v^{'}(1 + \tan^2\phi^{'}) = 629(1 + 0.525) = 959$$

$$P_V^{''}, P_i^{'} < \sigma_{c,all} \qquad \text{(safe!)}$$

Check stresses for Reservoir empty

$$P_V^{'}, P_V^{''} = \frac{\Sigma W}{B}\left(1 \mp \frac{6e}{B}\right) = \frac{6777.5}{17.67}\left(1 \mp \frac{6 * 2.97}{17.67}\right) = 0, 767.2$$

$$P_V^{'} = 0 \qquad\qquad P_V^{''} = 767.2$$

$$P_i^{''} = p_v^{''}(1 + \tan^2\phi^{''}) = 767.2(1 + 0.0875) = 832.3$$

$$P_V^{''}, P_i^{'} < \sigma_{c,all} \qquad \text{(safe!)}$$

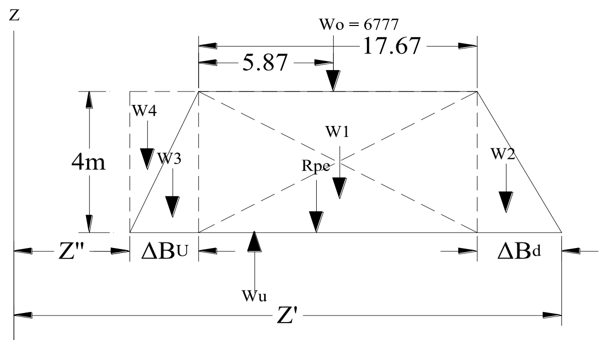
Block II

ho = 27m

Δh = 4m

h = 31m

Bo = 17.67 m



Line	Item	Description & dimension	Forces: Horizontal	Forces: Vertical	Lever	Moment
1	W0	concreter above h = 27m		6777	8.87	60111.99
2	W1	4*17.67*24		1696.32	11.83	20075.95
	Trial I	$\Delta B_d = 3.1$				
3	W2	3.1* *24		148.8	21.70	3229.46
4		Total Partial empty		8622.12	9.67	83417.4
		Estimation	Z' =	23.77		
		2B/3=Z' - 8.61 = 23.77 - 8.61=	14.1	B =	21.15	
		ΔBu =B - (Bo + ΔBd) =	0.38	Z" =	2.62	
		Z"+B/3 =	9.67	Z"+2B/3 =	16.72	
5	W3	0.5*0.38*4*24		18.24	2.87	52.35
6		Reservoir Empty		8640.36	9.66	83469.75
7	W4	Water column 0.38*25*10		110.2	2.81	309.662
7	Wu	Uplift: 0.5*B*0.5h*10		-1639.13	9.67	-15850.3
8	Fh	Water Pressure	4805		10.33	49651.67
9	Fwa	Wave action	31.25		31.47	983.44
		Reservoir Full	4836.25	7111.435	16.67	118564.2

The resultant for both reservoir empty and reservoir full case passes with in the middle third of the base. Furthermore, from the line of action of the resultant it can easily be deduced that the section is economical.

Check for friction Resistance $\tan\theta = \frac{\Sigma H}{\Sigma W} = \frac{4836}{7111} = 0.68 < 0.75 = f$; Friction alone is sufficient!

Check stresses for Reservoir Full

$$P_v', P_v'' = \frac{\Sigma W}{B}\left(1 \pm \frac{6e}{B}\right)$$

$$= \frac{7111}{21.15}\left(1 \pm \frac{6 * 3.48}{21.15}\right) = 668.1, 4.4$$

$$P_i', P_i'' = P_v'.P_v''(1 + \tan^2\phi', \tan^2\phi'')$$

$$\tan\phi' = \frac{3.1}{4} = 0.775 \qquad \tan\phi'' = \frac{0.38}{4} = 0.095$$

$$P_i' = 668.1(1 + 0.775^2) = 1072\ KPa < \sigma_{all} = 5000\ KPa$$ Safe!

Check stresses for Reservoir empty

$$P_v^{'}, P_v^{''} = \frac{\Sigma W}{B}\left(1 \mp \frac{6e}{B}\right)$$

$$= \frac{8640.36}{21.15}\left(1 \mp \frac{6 * 3.53}{21.15}\right) = 20.4, 796.6$$

$$P_i^{''} = P_v^{''}(1 + \tan^2 \phi'')$$

$$P_i^{''} = 796.6(1 + 0.095^2) = 803.8 KPa < \sigma_{all} = 5000\ KPa \qquad \text{Safe!}$$

Continue with the design block by block until you arrive at the required dam height or the limit of zone IV, whichever comes first. If the dam height could not be obtained in Zone IV, continue the design block by block in the remaining zones by fulfilling the design rules.

Example:
Design a non-overflow gravity dam by the Single-step method using the following data.

Item	**Value**	**Item**	**Value**
H_{max} (depth of headwater)	45 m	*f* (friction factor)	0.75
h_e (spillway crest to MWL)	3 m	s_a *(Shear strength)*	4.5 MPa
Tail water	None	s_{sf} (Shear safety factor)	5
Top width	7.5	ζ (Uplift factor)	0.5
γ_c (concrete Specific unit weight)	22 MPa	C (uplift area factor)	1
γ_w(water specific unit weight)	10 MPa	$\sigma_c^{'}$ (concrete ultimate strength)	30 MPa
Earthquake	small	F (Fetch length)	5 km
silt pressure	Ignore	V (Wind Velocity)	128 km/hr

Solution

Determine the wave height by the empirical equations

$$h_w = 0.763 + 0.032\sqrt{vf} - 0.271 f^{1/4} \qquad \text{for } f < 32\, km$$

$$h_w = 0.763 + 0.032\sqrt{128 * 5} - 0.271 * 5.0^{1/4}$$

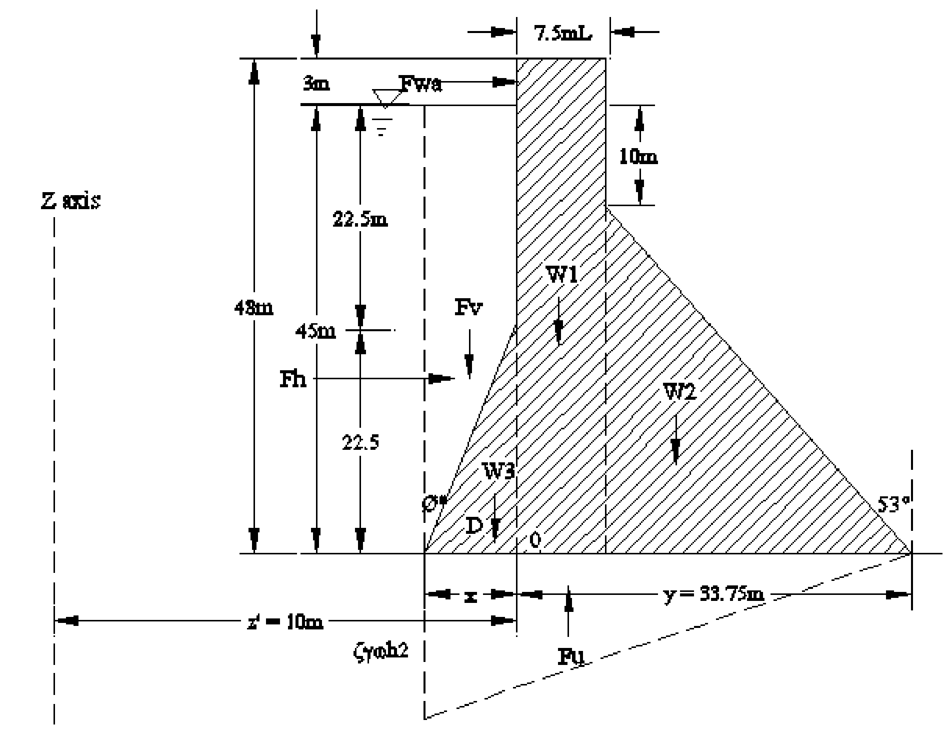
$$= 1.17 \text{ m}$$

Rise of water wave $= 1.33h_w$
$= 1.56$ m;

With an allowance of 0.14 m, free board = 1.70m

$F_{wv} = 2.0\gamma_w h^2_w$
$= 2.0 * 10 * 1.17^2$
$= 27.40$ kN/m

Point of application = 3/8 x 1.17 =0.44m above still water level.



Line	Item	Description & dimension	Forces Horizontal	Forces Vertical	Lever	Moment	Remark
1	W1	7.5*46.7*22		7705.5	13.75	105950.6	
2	W2	0.5*35*26.25*22		10106.25	26.25	265289.1	
		Total Partial empty		**17811.75**	**20.84**	**371239.7**	
Trial I		Estimation of x	Z'=	43.75			
		2B/3=Z' - 20.84 =	22.91	B=	34.37		
		x =	**0.75**	Z" =	9.25		
		Z"+B/3 =	20.71	Z"+2B/3 =	32.16		
3	W3	0.5*22.5*0.8*22		185.625	9.75	1809.84	
		Reservoir Empty		**17997.38**	**[20.73]**	**373049.5**	**Ok!**
4	Fv	Water column 0.3*25*10		253.125	9.63	2437.594	
5	Fu	Uplift: 0.5*B*0.5h*10		-3866.63	20.71	-80077.8	
6	Fh	Water Pressure	10125		15	151875	
7	Fwa	Wave action	27.4		45.44	1245.06	
		Reservoir Full	**10152.4**	**14383.88**	**[31.18]**	**448529.4**	**Ok!**

Check for sliding $\frac{\Sigma H}{\Sigma V} = \tan\theta = \frac{10152.4}{14383.88} = 0.71 < 0.75$ Friction alone is sufficient!

Check Stresses for Reservoir empty

$$P_V^{'}, P_V^{''} = \frac{\Sigma W}{B}\left(1 \mp \frac{6e}{B}\right) = \frac{17997.38}{34.37}\left(1 \mp \frac{6 * 5.71}{34.37}\right) = 1045.1, 0$$

$P_V^{'} = 1045.1$ $\qquad P_V^{''} = 0$

$$P_i^{'} = p_v^{'}(1 + \tan^2\phi^{'}) = 1045.1(1 + 0.0.033^2) = 1046.3kPa$$

$P_V^{''}, P_i^{'} < \sigma_{rock,all} = 4000\text{kPa}$ (safe!)

Check Stresses for Reservoir full

$$P_V^{'}, P_V^{''} = \frac{\Sigma W}{B}\left(1 \pm \frac{6e}{B}\right) = \frac{14383.88}{34.37}\left(1 \pm \frac{6 * 4.75}{34.37}\right) = 757.7, 70.88$$

$$P_V^{'} = 757.7 \text{ kPa} \qquad P_V^{''} = 70.88\, kPa$$

$$P_i^{'} = p_v^{'}(1 + \tan^2 \phi^{'}) = 757.7(1 + 1.327^2) = 2092.0\, kPa$$

$$P_V^{''}, P_i^{'} < \sigma_{rock,all} = 4000\ kPa \qquad \text{(safe!)}$$

Exercises:

1. Prepare a flow chart for a computer program which could be developed to design gravity dams by the multiple step method considering all the possible forces on the dam. The flowchart should clearly show the main program and sub programs.
2. The non-overflow dam previously designed by the multiple step method is to be constructed in a seismic area. Redesign the dam taking earth factor $\alpha h = 0.10$, period of earth quake vibration T = 0.4sec and a top width B = 12m. Dam height, H, is 40m
3. Design the dam of the previous example of multiple-step design method by the single-step method and compare the section obtained with the section of the multiple-step method
   iii. Foundation level is at 66m below the max water level
   iv. Allowable stress of foundation rock is 2Mpa
4. Design a non-overflow gravity dam by the Single-step method using the following data. (Consider earthquake and silt pressure)

Item	Value	Item	Value
H_{max} (depth of headwater)	45 m	*f* (friction factor)	0.75
h_e (spillway crest to MWL)	3 m	s_a *(Shear strength)*	4.5 MPa
Tail water	None	s_{sf} (Shear safety factor)	5
Top width	7.5	ζ (Uplift factor)	0.5
Hs (depth of silt-water mixture)	4 m	C (uplift area factor)	1
Ss(Specific gravity of silt)	1.5	$\sigma_c^{'}$ (concrete ultimate strength)	30 MPa
γ_{ss}(for horizontal silt water pressure)	14 kN/m^2	F (Fetch length)	5 km
α (earthquake factor)	0.12	V (Wind Velocity)	128 km/hr
T (period of EQ vibration)	0.80 sec	γ_c (concrete Specific unit weight)	22 MPa
		γ_w(water specific unit weight)	10 MPa

3.8 Gravity dam Construction, Quality control and the Future

3.8.1 The Construction Process

Dry construction area: Before construction can begin on any dam, the water in the streambed must be diverted or stopped from flowing through the site. As in the case of fill dams, a coffer-dam (a temporary structure to impound the water) must be built or the water must be diverted into another channel or area down-stream from the dam site. For large projects, this construction may be done several seasons before building of the dam begins. The flow of water is closed off at the very last moment.

Foundation: The foundation area for any concrete dam must be immaculate before the first concrete for the dam is placed. As for fill dams, this is a detailed process of excavating, cleaning, and repairing the rock throughout the foundation "footprint" and on both abutments (the sides of the canyon that form the ends of the dam). Sites immediately downstream of the dam for any power-plant, stilling basin, or other structure must also be prepared.

At some sites, extensive work may be required. If the rock in the foundation or abutments is prone to fracturing because of the load imposed by the dam and its reservoir, earthquake activity, or the properties of the rock, it may be necessary to install extensive systems of rock bolts or anchor bolts that are grouted into the rock through potential fracture zones. On the abutments above the dam, systems of rock bolts and netting may be required to keep large rock fragments from falling onto the dam. Instruments to monitor groundwater levels, joint movement, potential seepage, slope movements, and seismic activity are installed beginning during the early stages of foundation preparation through completion of the dam.

A cutoff wall may be excavated deep into rock or holes may be drilled in the foundation for the installation of reinforcing steel, called rebars, that extend up into the dam and will be tied to the steel inside the first lifts of the dam. The idea is to build a reservoir that, like a bowl, is equally sound around its perimeter. The water is deepest and heaviest at the dam (when the reservoir is near capacity) so the dam and its foundation cannot be a weak point in that perimeter.

Formwork and concrete casting: Forms made of wood or steel are constructed along the edges of each section of the dam. Rebar is placed inside the forms and tied to any adjacent rebar that was previously installed. The concrete is then poured or pumped in. The height of each lift of concrete is typically only 1.5-3 m and the length and width of each dam section to be poured as a unit is only about 15 m. Construction continues in this way as the dam is raised section by section and lift by lift. Some major dams are built in sections called blocks with keys or inter-locks that link adjacent blocks as well as structural steel connections.

The process is much like constructing a building except that the dam has far less internal space; surprisingly, however, major concrete dams have observation galleries at various levels so the condition of the inside of the dam can be observed for seepage and movement. Inlet and outlet tunnels or other structures also pass through concrete dams, making them very different from fill dams that have as few structures penetrating the mass of the dam as possible.

Early dam performance: As soon as a significant portion of the dam is built, the process of filling the reservoir may begin. This is done in a highly controlled manner to evaluate the stresses on the dam and observe its early performance. A temporary emergency spillway is constructed if dam building takes more than one construction season; lengthy construction is usually done in phases called stages, but each stage is fully complete in itself and is an operational dam. The upstream cofferdam may be left in place as a temporary precaution, but it is not usually designed to hold more than minimal stream flows and rainfall and will be dismantled as soon as practical. Depending on design, some dams are not filled until construction is essentially complete.

Appurtenances: The other structures that make the dam operational are added as soon as the elevation of their location is reached as the dam rises. The final components are erosion protection on the upstream (water) side of the dam (and sometimes downstream at the bases of outlet structures), instruments along the crest (top) of the dam, and roads, side-walks, streetlights, and retaining walls. A major dam like Hoover Dam has a full-fledged roadway along its crest; small dams will have maintenance roads that allow single-file access of vehicles only.

Away from the dam itself, the powerhouse, instrument buildings, and even homes for resident operators of the dam are also finished. Initial tests of all the facilities of the dam are performed.

Completion: The final details of constructions are wrapped up as the dam is put into service. The beginning of the dam's working life was also carefully scheduled as a design item, so that water is available in the reservoir as soon as the supply system is ready to pump and pipe it downstream, for example. A program of operations, routine maintenance, rehabilitation, safety checks, instrument monitoring, and detailed observation will continue and is mandated by law as long as the dam exists.

3.8.2 Quality Control

There is no dam construction without intensive quality control. The process of building alone involves heavy equipment and dangerous conditions for construction workers as well as the public. The population living downstream of the dam has to be protected over the structure itself; the professionals who design and construct these projects should absolutely be committed to safety, and they are monitored by local, regional, and federal agencies.

3.8.3 The Future

The future of concrete dams is the subject of much debate. Each year, over 100,000 lives are lost in floods, and flood control is a major reason for building dams, as well as protecting estuaries against flooding tides and improving navigation. Lives are also benefited by dams because they provide water supplies for irrigating fields and for drinking water, and hydroelectric power is a non-polluting source of electricity. Reservoirs are also enjoyed for recreation, tourism, and fisheries.

However, dams are also damaging to the environment. They can change ecosystems, drown forests and wildlife (including endangered species), change water quality and sedimentation patterns, cause loss of agricultural lands and fertile soil, regulate river flows, spread disease (by creating large reservoirs that are home to disease-bearing insects), and perhaps even affect climate. There are also adverse social effects because human populations are displaced and not satisfactorily resettled.

For years before the start of construction in 1994 of the Three Gorges Dam in China, environmentalists the world over organized protests to try to stop this huge project. They have not succeeded, but controversy over this project is representative of the arguments all proposed dams will face in the future. The balance between meeting human needs for water, power, and flood control and protecting the environment from human eradication or encroachment must be carefully weighed.

4 Concrete Arch and Concrete Buttress dams

4.1 Concrete Arch Dam

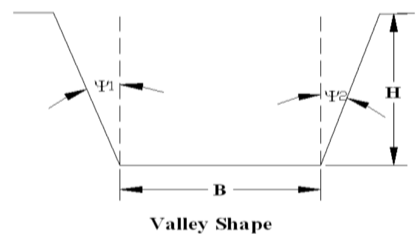
Concrete arch dam is a concrete dam with a considerable upstream curvature, structurally resisting the imposed loads by arch and cantilever action. Arch dam transmits the major portion of the water load to the abutments or valley sides rather than to the floor of the valley, hence, large horizontal reactions are required by the abutments.

Arch dams are restricted to relatively narrow valley sections with strong abutments. They are structurally more efficient than the gravity or buttress counterparts, greatly reducing the volume of concrete required.

The structural interaction between the loaded arch dam and its supporting abutments is extremely complex and is beyond the scope of this course.

Valleys suited to arch dams are narrow gorges. The ratio of crest length to dam height is recommended not to exceed five. To determine the site suitability for an arch dam the following equation of canyon shape factor (CSF) is proposed:

$$CSF = \frac{B + H(\sec\psi_1 + \sec\psi_2)}{H}$$



Usual values of CSF are from 2 to 5. The lower the CSF value the thinner the section.

Table 4-1 Classification of valley shapes based on CSF value

Valley type	Bottom width B	ψ_1	ψ_2	CSF
U shaped	< H	$< 15^0$	$< 15^0$	< 3.1
Narrow V shaped	0	$< 35^0$	$< 35^0$	< 2.4
Wide V-shaped	0	$> 35^0$	$> 35^0$	> 2.4
Composite U-V shaped	< 2H	$> 15^0$	$> 15^0$	≅ 4.1
Wide and flat shapes	> 2H	ψ_1	ψ_2	> 4.1
Unclassified	Highly irregular valley shape			

Arch dams may be grouped into two main divisions: Massive arch dams and multiple arch dams.

Massive arch dam:- the whole span of the dam is covered by a single curved wall usually vertical or nearly so.

Multiple arch dam:- series of arches cover the whole span of the dam, usually inclined and supported on piers or buttresses. These are usually considered as a type of buttress dam and will be described later.

Massive arch dams in turn are divided into the following types:

i. Constant radius arch dams,
ii. Constant angle arch dams
iii. Variable radius arch dams
iv. Double curvature or Cupola arch dams
v. Arch gravity dams

4.1.1 Constant radius arch dam

Constant radius is the simplest geometric profile combining a vertical upstream face of constant extrados (outside curved surface of the arch dam) radius with a uniform radial downstream slope. Though the constant radius arch dam is not the most economical profile in volume, it is simple to analyze and construct. Besides, this profile is suitable to relatively symmetrical "U" shaped valley. For a site with variable span length "V" shaped valleys a constant radius can have the correct central angle only at one elevation. Therefore, smallest masonry volume for the whole dam is obtained by increasing the top angle to get the best average angle. Usually a maximum of 150^o is used for the top arch.

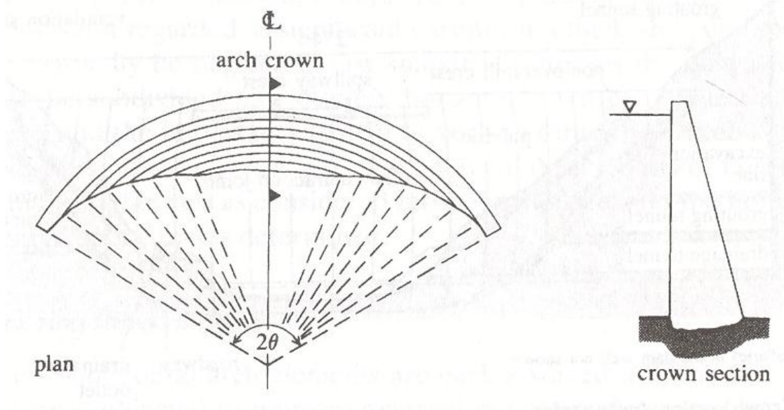


Figure 4-1 Constant Radius Arch dam

4.1.2 Constant Angle Arch dam

Central angle 2θ of different arches has the same magnitude from top to bottom. In practice $2\theta = 100^0$ to 150^0 is used. It uses about 70% concrete as compared to constant radius arch dam.

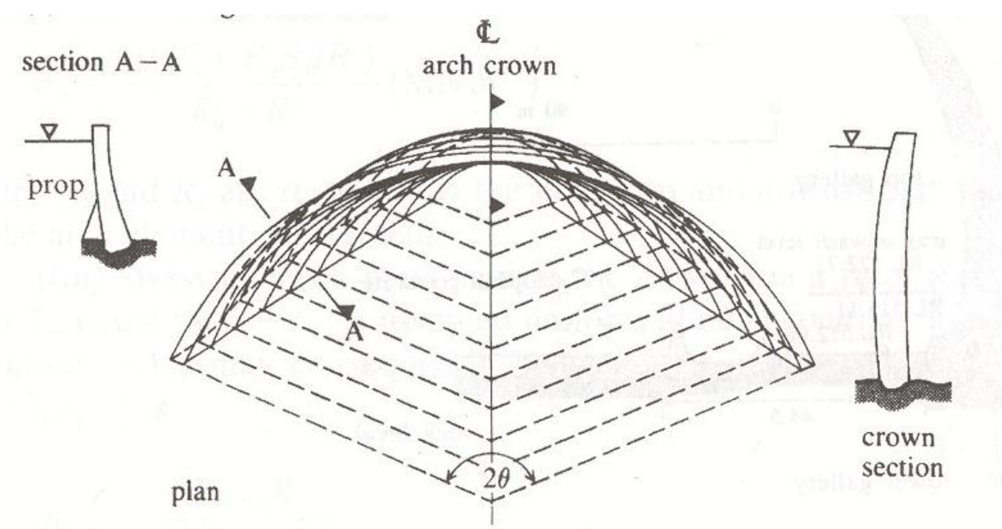


Figure 4-2. constant angle arch dam.

4.1.3 Variable radius arch dam

It is a compromise between constant radius and constant angle arch dams, i.e., neither the radius nor the angle is constant. The radii of the extrados and intrados surfaces vary from the top to bottom, usually maximum at the top and minimum at the base. The central angle of the different arches is not constant; it usually ranges from 80^0 to 150^0. The central angle for the top arch is made as wide as possible. The dam is suitable for V and U-V shaped valleys.

The radius is varied to cut the face at the required contour interval so that there is no overhang. Masonry volume consumed is about 82% of that for constant radius arch dam of the same height.

[include figure]

4.1.4 Loads on arch dam

The forces acting on arch dam are the same as that of gravity dams. Uplift forces are less important (not significant). Internal stresses caused by temperature changes and yielding of abutments are very important. Foundation stresses are generally small.

4.1.5 Methods of design of massive arch dams.

- thin cylinder theory
- elastic theory
- trial load method (discussion beyond the scope of this course)

4.1.6 The thin cylinder theory

It is envisaged that the weight of concrete and water in the dam is carried directly to the foundation. The horizontal water load is carried entirely by arch action. The theory assumes that the arch is simply supported at the abutments and that the stresses are approximately the same as in thin cylinder of equal outside radius r_o.

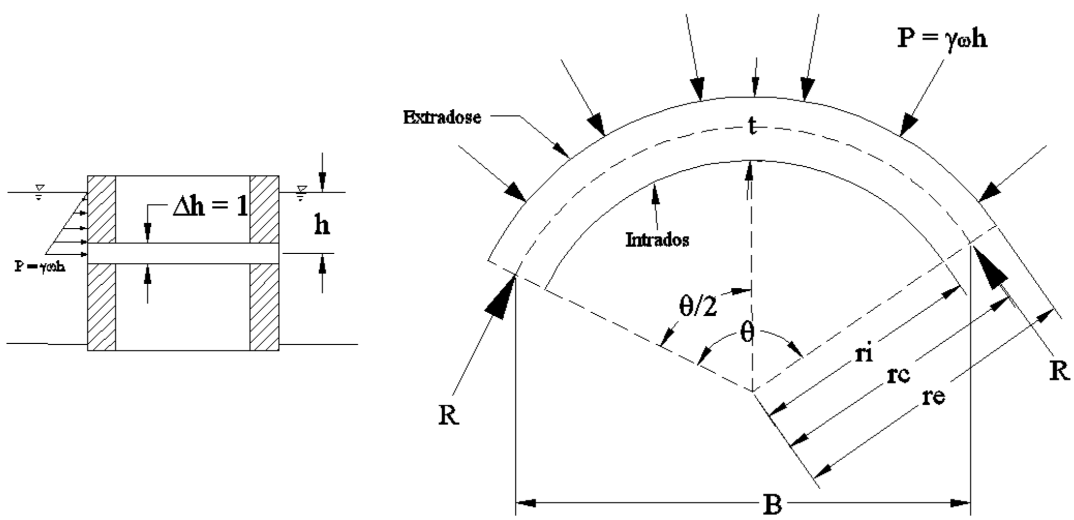


Figure 4-3 Thin cylinder model of an Arch dam

Summing forces parallel to the stream axis

$$2R\sin\theta/2 = 2\gamma_w h r_e \sin\theta/2$$

$$R = \gamma_w h r_e$$

The transverse unit stress

$$\sigma = \frac{R}{t*1} = \frac{\gamma_w h r_e}{t}$$

for a given stress

$$t = \frac{\gamma_w h r_e}{\sigma_{all}}$$

Note: the hydrostatic pressure $\gamma_w h$ may be increased by earth quake and other pressure forces where applicable:

since $r_e = r_c + 0.5t$ and $r_e = r_i + t$

$$t = \frac{\gamma_w h r_c}{\sigma_{all} - 0.5\gamma_w h} \quad \text{or} \quad t = \frac{\gamma_w h r_i}{\sigma_{all} - \gamma_w h}$$

Condition for least volume of arch.

$$V = (t*1)r\theta$$

$$t = \frac{\gamma_w h r}{\sigma} = kr$$

$$V = kr^2\theta = k\theta\left[\frac{B}{2\sin\theta/2}\right]^2$$

Differentiating V with respect to θ and setting to zero, $\theta = 133.5^o$ which is the most economical angle for arch with minimum volume.

For $\theta = 133.5^0$ $r = 0.544B$

Design example

Design a 100m height constant radius arch dam, by the thin cylinder theory for a valley 100m wide at the base and 150m wide at a height of 100m σ_{all} = 4MPa

Solution:

The top arch is taken to be 140^o

$$R = \frac{B}{2\sin\theta/2} = 75/\sin 70 = 79.8$$

Take r = 80m

The extrados radius r_e of all arches is kept as 80m. Calculations are shown in table below:

h	B	r_e	P	t	r_i	θ
0	150	80	0	0	80	139.27
10	145	80	100	2	78	129.98
20	140	80	200	4	76	122.09
30	135	80	300	6	74	115.08
40	130	80	400	8	72	108.68
50	125	80	500	10	70	102.75
60	120	80	600	12	68	97.18
70	115	80	700	14	66	91.90
80	110	80	800	16	64	86.87
90	105	80	900	18	62	82.03
100	100	80	1000	20	60	77.36

Note: Provide a nominal thickness of 1.5 m when necessary.

Procedure to layout a constant radius arch dam:

i. Draw excavated rock contours,
ii. Draw the center line and locate the arch center O,
iii. Draw the extrados and intrados curves for the top arch,

iv. Starting at the point of intersection of the center line and the extrados curve, lay off the arch thickness t at successive contour intervals toward the point of intersection of the center line and intrados curve of the last arch,

v. With center at O, draw arcs through these points to the respective contours,

vi. Draw the x-section on the center line. It may also be drawn before the plan.

Example:

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

Solution

Taking $\theta = 133.44^0$

$r_i = 0.544B$

$$t = \frac{\gamma_w h r_i}{\sigma_{all} - \gamma_w h}$$

$r_e = r_i + t$

h	B	r_i	P	$P*r_i$	σ_{all} –p	t	r_e
0	240	130.56	0	0	5000	0	130.56
10	220	119.68	100	11968	4900	2.44	122.12
20	200	108.80	200	21760	4800	4.53	113.33
30	180	97.92	300	29376	4700	6.25	104.17
40	160	87.04	400	34816	4600	7.57	94.61
50	140	76.16	500	38080	4500	8.46	84.62
60	120	65.28	600	39168	4400	8.90	74.18
70	100	54.40	700	38080	4300	8.86	63.26
80	80	43.52	800	34816	4200	8.29	51.81
90	60	32.64	900	29376	4100	7.16	39.80
100	40	21.76	1000	21760	4000	5.44	27.20

4.1.7 Design procedure for variable radius arch dam

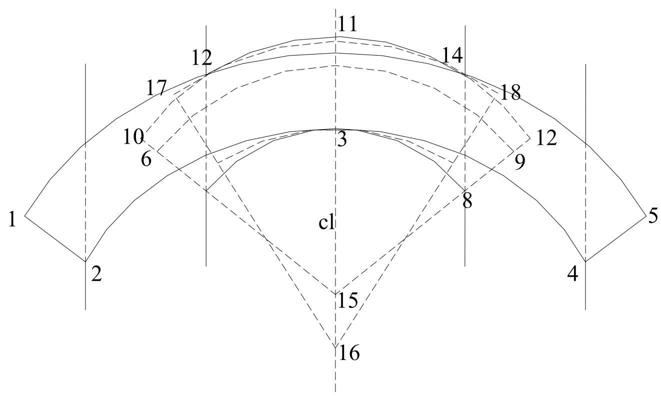


Figure 4-4. overhang of arches

Design is begun at the top, θ for the top arch being as wide as possible.

1-2-3-4-5-1: top arch

6-7-3-8-9: the constant angle design for the next contour interval. Thickening the arch to 10-11-12, overhang can be eliminated. If the arch 6-7-3-8-9-6 fulfills the equation

$$t = \frac{\gamma_w h r_e}{\sigma_{all}} \quad ,$$

10-7-3-8-12 is thicker than necessary. Hence, lengthening the radius on arch 16-17-18 is found by trial which just avoids overhang and fulfills the requirement of the above equation. The dimensions of successive arches, proceeding downward, are determined in the same manner.

4.1.8 Elastic arch theory (Arch dam analysis)

The theory assumes complete transfer of load by arch action only. Horizontal arch rings are assumed fixed to the abutments, but acting independently of neighboring rings. Effect of temperature variation on arch stress is considered. This method can be used for preliminary design to determine the adequacy of the section designed by the thin cylinder theory.
Modified Cain's Equations are used for calculating forces and moments at the crown and at abutments.

$$Ho = pr - \frac{pr}{D} 2\phi \sin\phi \frac{t^2}{12r} \quad ; \quad \text{where } \phi \text{ is in radians}$$

$$D = \left(1 + \frac{t^2}{12r^2}\right)\phi\left(\phi + \frac{\sin 2\phi}{2}\right) - 2\sin^2\phi \quad ; \quad \text{if shear is neglected.}$$

$$D = \left(1 + \frac{t^2}{12r^2}\right)\phi\left(\phi + \frac{\sin 2\phi}{2}\right) - 2\sin^2\phi + \frac{3t^2}{12r^2}\phi\left(\phi - \frac{\sin 2\phi}{2}\right) ; \text{if shear is considered .}$$

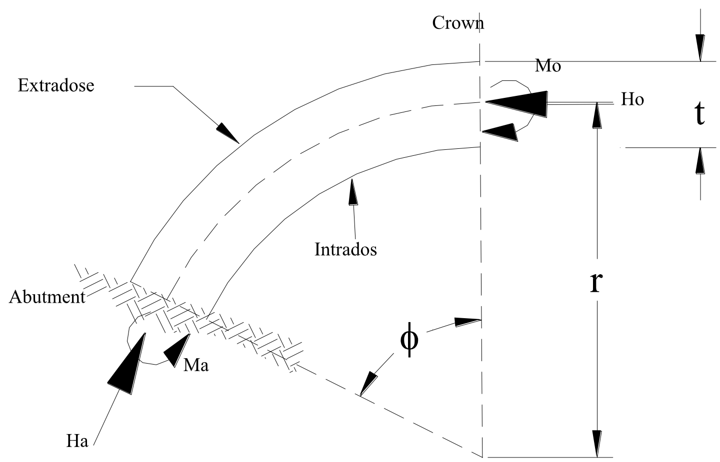


Figure 4-5 Constant thickness circular arch, fixed at abutments.

$$\text{Moment at crown :} \quad Mo = r(pr - Ho)\left(1 - \frac{\sin\phi}{\phi}\right)$$

$$\text{Force at abutment :} \quad Ha = pr - (pr - Ho)\cos\phi$$

$$\text{Moment at abutement :} \quad Ma = r(pr - Ho)\left(\frac{\sin\phi}{\phi} - \cos\phi\right)$$

After determining forces and moments, stresses at intrados and extrados are calculated from

$$\sigma = \left(\frac{H}{t} \pm \frac{6M}{t^2}\right)$$

Exercises:

1. Design a 55m high constant angle arch dam, by the thin cylinder theory, for a valley 12m wide at base and 68m at a height of 55m. Draw to scale the plan and section on the centerline of the dam. Take σall 200t/m^2.

2. Determine the stresses at the intrados and extrados of the crown and abutment for the constant angle arch dam of the previous example at h = 40m.

4.2 Buttress dams

Buttress dams consist of a slopping u/s membrane which transmits the water load to the axis of the dam. The principal structural elements of a buttress dam are the water supporting u/s deck and the buttresses that in turn support the deck. The buttresses are carefully spaced, triangular walls proportionate to transmit the water load and the weight of the structure to the foundation.
Buttress dams are adaptable to both overflow and non-overflow conditions. In overflow dams a downstream deck is provided to guide the flowing stream.

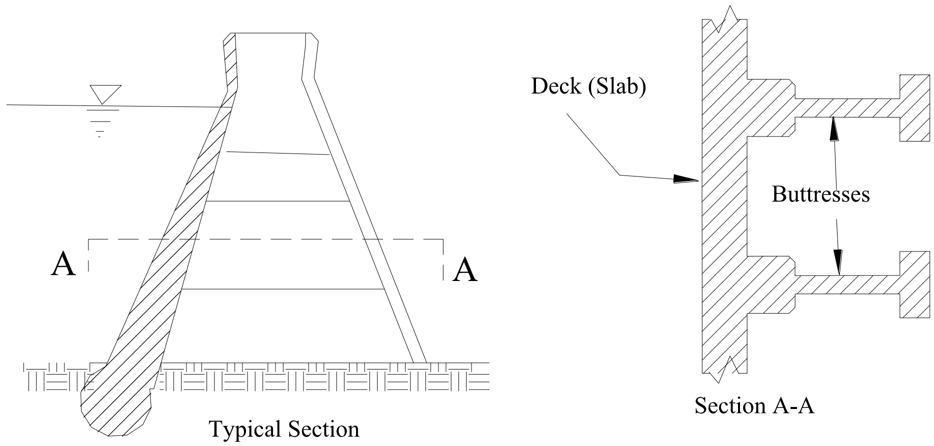


Figure 4-6 Typical section and plan view of a buttress dam

4.2.1 Classification of buttress dams

Buttress dams can be classified according to the water supporting membrane utilized in the body of the structure. The main types are shown in Figure 4-7 below and there are other types emerged from the flat deck types with modification in the buttress configuration.

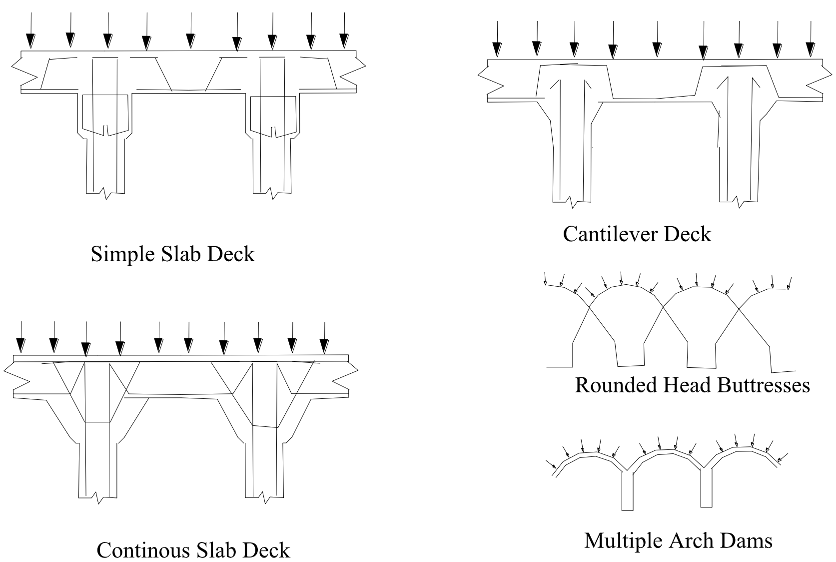


Figure 4-7 Types of Buttress Dam

4.2.2 Advantages and Disadvantages of Buttress Dams

Advantages of buttress dams

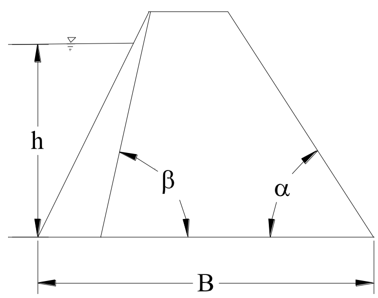
1. - less concrete is used compared to a gravity dam of the same height,
   - Increased surface area to volume ration

- Better heat dissipation
- Increased speed of construction

2. More safety against overturning and sliding because of the larger vertical component of hydrostatic force exerted on the dam (highly inclined u/s face)
3. More equal distribution of stresses of foundation.
4. Less massive than gravity dam hence may be used on weak foundation not suitable for gravity dam
5. Decreased uplift pressure (if no spread footing, joining the buttresses is used)

Disadvantages of Buttress dam

1. needs reinforcement and expensive shuttering
2. needs more skilled labor
3. slabs and columns are highly stressed; danger of deterioration of concrete of the u/s deck
4. more susceptible to damage by sabotage



Usually, $\beta = 40^0 - 60^0$
To increase stability
$\alpha = 70^0 - 80^0$
To provide for adequate Buttress width
$B = (1.2 \text{ to } 1.5)h$

4.2.3 Forces on buttress dams

Essentially buttress dams are subjected to the same forces as gravity dams. Uplift forces may be insignificant as in the case of arch dams. Wind load on buttress faces may be considerable when high velocity winds blow diagonally from the downstream side, hence struts (beams) are usually provided.

4.2.4 Design Principle for Buttress Dams

The stability analysis for buttresses is done in a similar fashion as for a gravity dam. However, the design element is not taken to be a slice of unit thickness as in gravity dams, but the full panel is considered. In addition to satisfying the stability criteria the buttresses are designed to conform to the design rules for structural concrete members.

The buttress width is determined by considering the buttress to be a vertical cantilever beam. The width has to be sufficient to avoid tension at the upstream face when fully loaded and also to avoid excessive compression at the downstream face. In order to determine the thickness of the buttress required to prevent buckling they are considered to be bearing walls instead of beams. The minimum allowable thickness is same as that for columns. The unsupported length is generally reduced by providing struts at intermediate points.

Simple slab (Ambersen type) buttress dams

The slab is simply supported and the joint between the slab and buttresses is filled with asphalt putty or any flexible compound.

The slab is designed by assuming that it consists of a series of parallel beams acting independent of one another and simply supported on the buttresses.

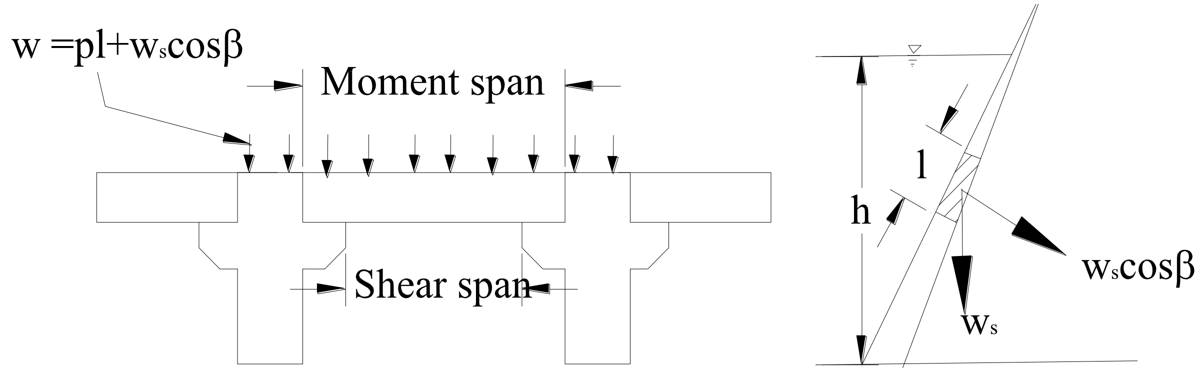


Figure 4-8 Modeling of simply supported Buttress dam

Spacing of the buttress is governed by:

i. length of the dam
ii. pressure of spread footing or a continuous floor slab
iii. presence of spillway over the dam
iv. slope of the upstream water supporting membrane(slab)
v. Unusual foundation or side hill condition.

For high dams greater spacing may be economical. Wider spacing may entail increased thickness and reinforcement of slabs. The spacing that will give the most economical balance of concrete steel and formwork area is determined by the total cost of the items.

Mean height	Economical buttress spacing (distance b/n CL)
15 to 30	4.5 to 6.0
30 to 45	9.0 to 12.0
Above 45	12.0 to 12

The buttress is designed as a system of columns each carrying the load by column action to the foundation. These columns are proportioned to develop a uniform compressive stress and curved to avoid any serious eccentricity on any horizontal or normal plane when the water and concrete loads are resolved.

5 Embankment Dam

5.1 Introduction

Embankment dam is a water impounding structure constructed from fragmental natural materials excavated or obtained close to the dam site. The natural fill materials are placed and compacted without the addition of any binding agent, using high capacity mechanical plant. They rely on their weight to resist the flow of water, just like concrete gravity dams.

Embankment dam derive its strength from position, internal friction and mutual attraction of particles. Relative to concrete dams, embankment dams offer more flexibility; and hence can deform slightly to conform to deflection of the foundation without failure.

Broadly, depending upon the material used during construction, embankment dams are classified in to two:

1) *Earth fill Embankments*: if compacted soils, i.e. clays/silts & sands, account for over 50% of the placed volume of material
2) *Rock fill Embankment*: if compacted rock particles larger than a man can easily lift, i.e. coarse grained frictional material, accounts for over 50% of the placed volume of materials.

Embankment dam possesses many outstanding merits which could be summarized as follows:

A. Suitability of the type to different site conditions such as wide valleys, steep sided gorges, etc.
B. Adaptability to a broad range of foundation condition such as rock and pervious soil formation,
C. use of natural materials,
D. Extreme flexibility to accommodate different fill materials,
E. Highly mechanized and effectively continuous construction process,
F. Appreciable accommodation of settlement-deformation without risk of serious cracking and possible failure.

The relative disadvantages of the embankment dam are

A. Inherently susceptible to damage or destruction by overtopping
B. Necessity of separate spillway structure
C. Vulnerability to concealed leakage and internal erosion in dam or foundation

5.2 Key elements and appurtenances of Embankment dam

Every embankment dam consists of three basic components plus a number of appurtenances which enable the basic components to function efficiently shown in Figure 5-1.

5.2.1 Foundation:

The foundation of embankment dam could either be earth or rock material. The foundation provides support resisting both vertical and horizontal loads. It may also resist seepage beneath the embankment

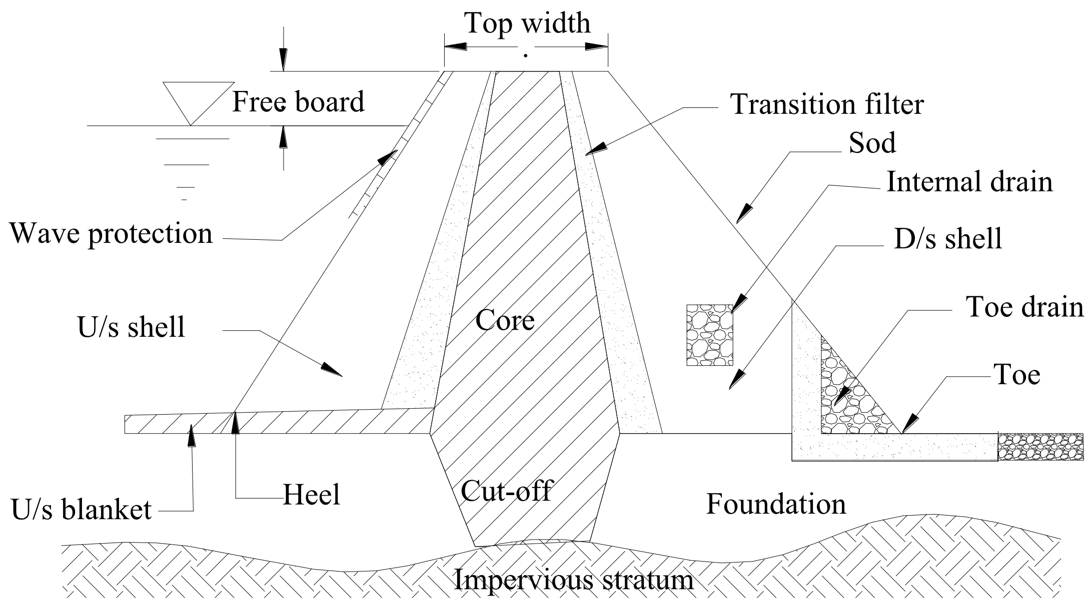


Figure 5-1 Basic components and appurtenances of Embankment dam

5.2.2 Core or membrane

The primary purpose of the core or membrane is to hold back free water. Depending on the structural requirements of the dam, the core may be placed at the center or upstream from the center, or on the upstream face (in the case of certain rock fill dams)

When the foundation is incapable of resisting under seepage the core is extended down into the foundation to impervious layer. Such an extension of the core is termed cut-off.

Core material:

Earth, concrete or masonry, steel sheeting, etc. are used as core material. Lack of flexibility of concrete and masonry make them undesirable. An earth core (when suitable material is available) is usually cheaper and more water tight than any other type. Suitability of earth core depends on the character of the available soil.

Table 5-1: Permeability of different soil types

Permeability coefficient	Typical soil	Value as core
2 – 0.002	Sand	Considerable leakage
0.002 – 0.0002	Silty clay	Usable with good control if some leakage is tolerable
0.0002 – 0.000006	Silts	Little leakage if well compacted
≤ 0.000006	Silty clay, clay	Impervious

A core should not be composed of silt which tends to swell upon saturation. To avoid swelling tendencies, the elasticity index should not exceed 30.

Typical requirements for core compaction are

90 – 97% of standard proctor maximum, or
87 – 95% of modified proctor maximum.

Water content: - as high as possible consistent with the above requirements.
Core thickness: - to control erosion and provide good compaction a minimum core thickness in meters is given by

b = 6 + 0.1Δh (clay)
b = 6 + 0.3Δh (silt)

Where: Δh = head difference at that point;
b = core width at that point.

5.2.3 Shell

The purpose of shell is to provide structural support for the core and to distribute the loads over the foundation. The shell also acts as foundation for most of the appurtenances. Sometimes the core and shell of a dam are constructed of the same material (homogenous dam).

Shell (embankment) materials

Availability and strength are the requirements for selection.

Strength: - the strength for the upstream side should be that at the inundated condition. The same strength should be used for the downstream face which is below the maximum phreatic line.

Permeability: - high permeability is desirable from the standpoint of pressure buildup during construction and stability during sudden drawdown.

Typical compaction requirements:

95 – 100% of standard proctor maximum;
92 – 97% of modified proctor maximum.

Slopes: - shell slopes are based on stability analysis. When the stability is insufficient, improvements are possible by adopting

a. Flatter slopes;
b. Increasing strength through high density;
c. Treatment for weak foundation;
d. Drainage of the foundation and embankment.

Table 5-2: Tentative slopes of shoulder for different embankment materials

Soil type	Upstream	Downstream
Gravel, sandy gravel with core	2.5H : 1V	2.0H : 1V
Clean sand with core	3.0H : 1V	2.5H : 1V
Low density silt, micaceous silt	3.5H : 1V	3.0H : 1V
Low plasticity clay	3.0H : 1V	2.5H : 1V

Composite slopes: - are used for large dams. They can be found in two ways: a series of straight slopes or a constant slope with berms.

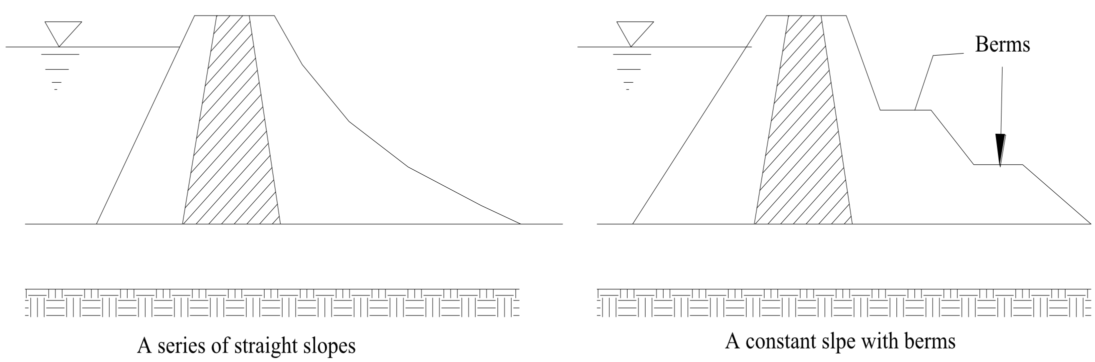


Figure 5-2 composite slopes for shell of embankment dam

A berm is a level surface on the slope that can serve the following purposes

1. Increases slope stability by increasing dam width;
2. Breaks the continuous downstream slope to reduce surface erosion
3. Provides level surface for maintenance operations, roads, etc.

Berm is also used at the bottom of a zone of riprap to provide supporting shoulder.

5.2.4 Height of dam:

Required height of an embankment dam is the vertical distance from the foundation to the water surface in the reservoir, when the spillway is discharging at design capacity, plus a free board allowance.

Free Board = maximum wave run-up height + allowance for settlement + allowance for splash

Maximum wave run-up height = $4h_w/3$

Where: h_w = effective wave height (with expectancy of 1%)

Table 5-3: Wave run-up to maximum wave height ratio on slopes

Slope	Ratio of run-up to maximum wave height	
	Smooth Surface	Riprap surface
1.5H : 1V	2.5	1.6
1H : 1V	2.0	1.3

Maximum vertical height of run-up = Expected wave height * appropriate factor from Table 5-3

Settlement allowance: the following may be used as guide.

For foundation: 1% of height of dam

For embankment: 1-2% of height of embankment

Splash allowance could be taken 0.30 – 0.50m.

5.2.5 Top Width:

- Should be sufficient to keep the phreatic line with in the dam when the reservoir is full
- Should be sufficient to withstand wave action and earthquake shock
- Has to satisfy secondary requirements such as minimum roadway width.

5.2.6 Appurtenances

Transition filter: - it is provided between core and shell to prevent migration of the core material into the pores of the shell material. It is particularly needed between clay cores and rock and gravel shells.

The objective of transition filter is to carry away seepage that has passed through the core and cut-off and to prevent stratum of the upper part of the downstream shell.

Toe drain: - it helps to prevent sloughing of the downstream face as a result of rain water or seepage saturation. In small dams, the toe drain serves also as internal drain. In large dams with pervious foundation, the toe drain and the internal drain are sometimes combined. Drains need protective filter (inverted filter) to prevent clogging of the drain.

Riprap: - required to cover the upstream/downstream face.

Normally riprap extended from above the maximum water level to just below the minimum.

Sod: - required on the downstream face to prevent rain wash.

For economic reasons, the material available at the particular site has to be employed as much as possible for the construction of the earth dam and the quantity of imported material should be minimized.

Internal drains: - they are essential in large dams where the d/s shell is not so pervious.

5.2. Types of Embankment dam

The materials available locally control the size and configuration of the dam. Many small embankment dams are built entirely of a single type of material such as stream alluvium, weathered bedrock, or glacial till. These are *homogeneous* dams, constructed more or less of uniform natural material as shown in Figure 5-3.

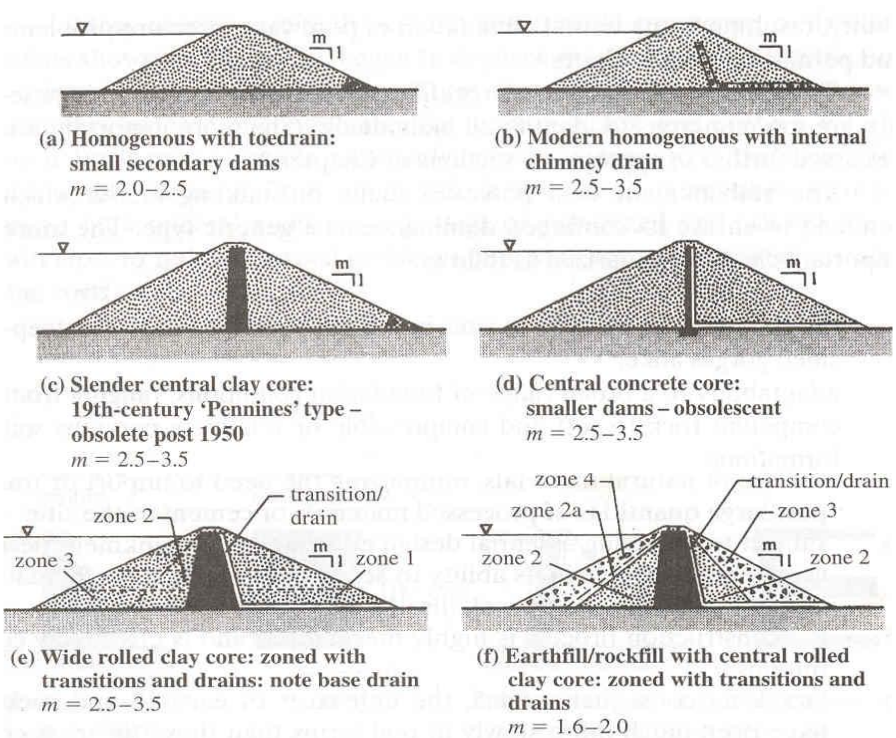


Figure 5-3 Principal variants of earth fill embankment dams (Values of m are examples)

The central core earth fill profile, shown in Figure 5-3 (c) and (d), is the most common for larger embankments dams. Larger embankment dams are also zoned and constructed of a variety of materials Figure 5-3 (f), either extracted from different local sources or prepared by mechanical or hydraulic separation of source material into fractions with different properties.

An important element in a zoned dam is an impermeable blanket or core which usually consists of clayey materials obtained locally. In locations where naturally impermeable materials are unavailable the dams are built of rock or earth-rock aggregates as shown in Figure 5-4, and the impermeable layers of reinforced concrete, asphalt concrete, or riveted sheet steel are placed on the upstream face of the dam.

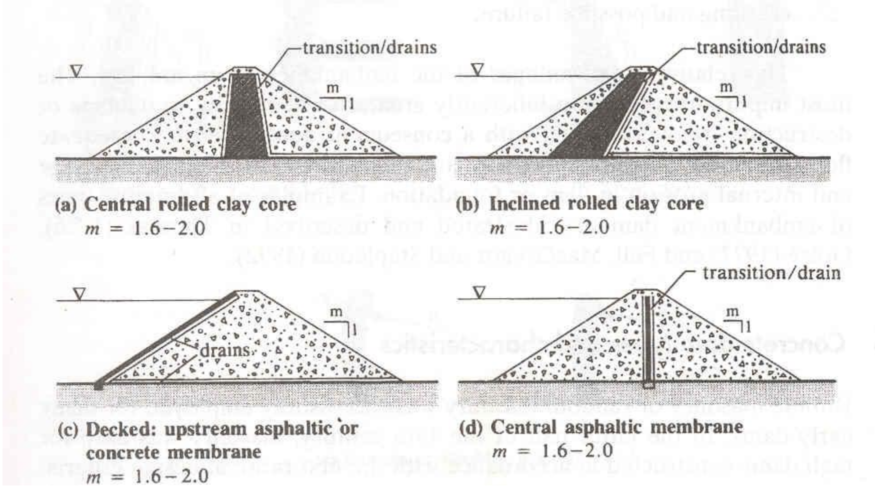


Figure 5-4 Principal variants of rock fill embankments dams (values of m are examples)

Selection of the optimum type of embankments for a specific location is determined largely by the nature and availability of different fill materials in sufficient quantity.

The primary loads acting on an embankment do not differ in principle from those applicable to gravity dams. There are, however, the conceptual differences there referred to with regard to the water load which is exerted inside the upstream shoulder fill. Self weight load, similarly a distributed internal body load, is significant with respect to stability and internal stress for the embankment and for a compressible soil foundation.

Because of such differences, embankments dam analysis is less formalized and is carried out quite differently from concrete dam analysis.

5.3. Causes of Failure of Embankment dams

Embankment dams, like any other engineering structure, may fail due to improper design, faulty constructions, lack of maintenance, etc. Generally, causes of failure are grouped into three classes: Hydraulic failure, Seepage failure and Structural failure.

Hydraulic failures: About 40% of earth dam failures have been attributed to these causes due to;

1. *Overtopping*. Occurs when the design flood is less than the coming flood. Spillway and outlet capacity must be sufficient to prevent overtopping. Freeboard should also be sufficient to prevent overtopping by wave action.
2. *Erosion of upstream faces*. Wind waves of water developed due to wind near the top water try to notch-out the soil from u/s face and may even sometimes cause the slip of the u/s slope.(upstream slope pitching or rip rap should be applied.)
3. *Erosion of downstream face by gully formation*. Heavy rains falling directly over d/s face and the erosive action of the moving water may lead to the formation of gullies on the d/s face, ultimately leading to the dam failure.
4. *Cracking due to frost action*. Frost in the upper portion of dam may cause heaving of soil with dangerous seepage. Consequently failure. Provide an additional free board allowance up to a maximum of say 1.5m may be provided.

Seepage failure: controlled seepage or limited uniform seepage is inevitable in all embankments and it does not produce any harm. However, uncontrolled or concentrated seepage through the dam or the foundation may lead to piping[*] or sloughing[†] and the subsequent failure of the dam.

[*] The progressive erosion and subsequent removal of soil grains from within the body of the dam or the foundation of the dam

[†] The progressive removal of soil from the wet d/s face.

Structural failure: about 25% of the dam failures have been attributed to structural failures. Structural failures are generally caused by shear failures, causing slides.

Causes of failure as categorized based on time of occurrence
During construction

- Unstable slop
- Heavy rainfall that washes the d/s face
- Weak foundation

After construction

- Failure of u/s face due to sudden drawdown
- Failure of d/s when the reservoir is full
- Overtopping
- Seepage failure.

5.4. Design features

Some of the more important features that should be considered in the design of embankment dams are:

1. *Zoning of shoulder-fills*: the permeability of successive zones should increase toward the outer slopes, materials with a high degree of inherent stability being used to enclose and support the less stable impervious core and filter.
2. *Spillway location*: geotechnical and hydraulic design considerations require that to minimize the risk of damage to the dam under flood conditions the spillway and discharge channel are kept clear of the embankment.
3. *Freeboard*: is the difference between maximum reservoir level and minimum crest level of the dam. The provision necessary for long-term settlement within the overall minimum freeboard is determined by the height of dam and the depth of compressible foundation at any section.
   The overall minimum freeboard from spillway sill to dam crest should be at least 1.5m on the smallest reservoir embankment, and it will be very much greater for larger embankments and/or reservoir.
   The minimum height of freeboard for wave action is, generally, $1.5h_w$
   $$h_w = 0.032\sqrt{v.F} + 0.763 - 0.271\sqrt[4]{F} \qquad \text{For } F < 32\,km$$
   $$h_w = 0.032\sqrt{v.F} \qquad \text{For } F > 32\,km$$
   Where; v is wind velocity (km/hr)
   F is fetch or straight length of water expansion in km
4. *Foundation seepage control:* seepage flows and pressure within the foundation are controlled by cut-offs and by drainage. *Cut-offs is impervious barriers which function as extensions of the embankments core into foundation*. The cut-offs are generally two types:
   a) Fully penetrating cut-off: penetrate to impervious strata
   b) Partially penetrating cut-off: terminate where the head loss across the cut-off is sufficient to effect the required degree of control

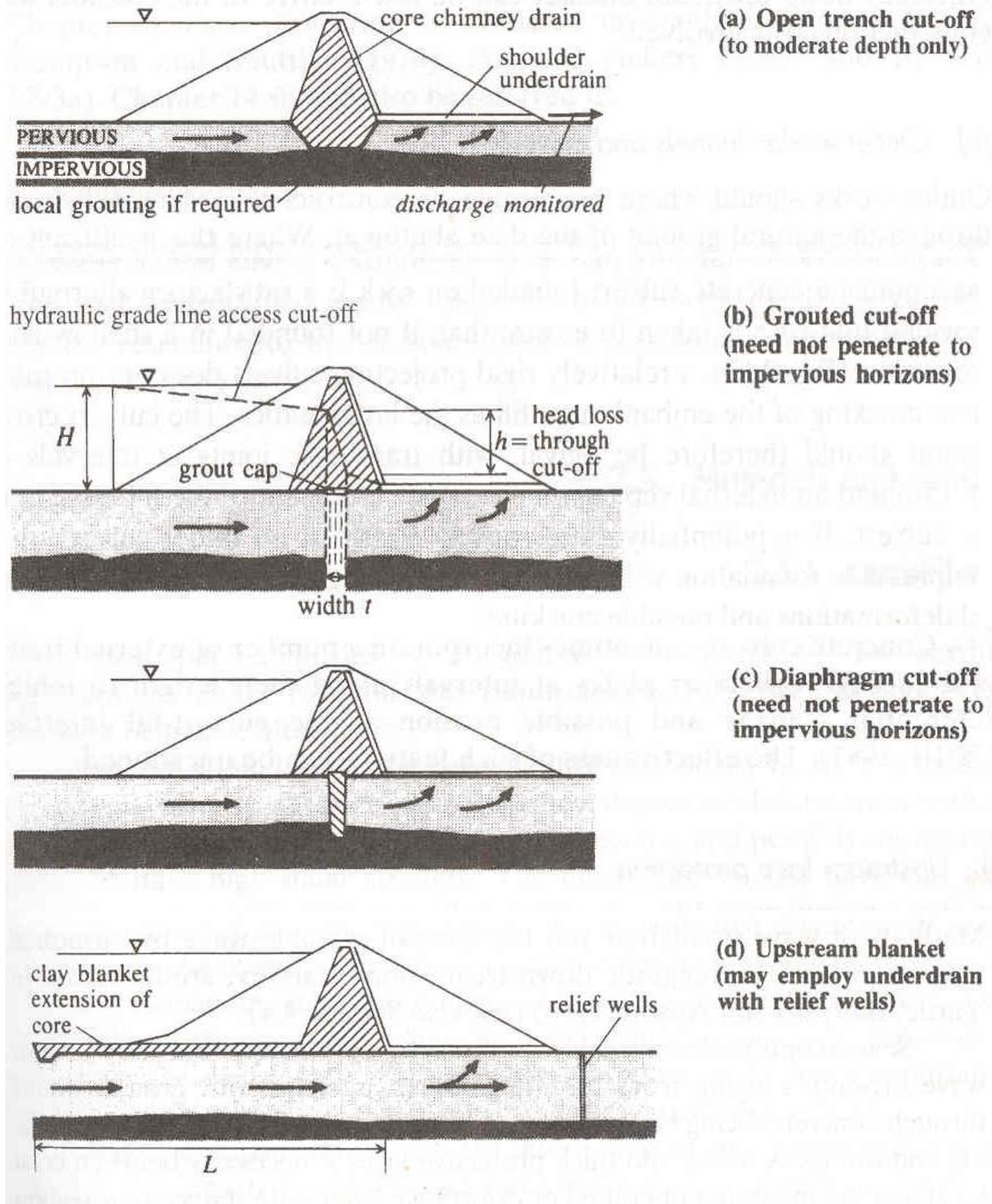


Figure 5-5 Cut-offs and control of under seepage

5. *Outlet works (tunnels and culverts):* outlet works should where practicable be constructed as a tunnel driven through the natural ground of the dam abutments. Where this is difficult or uneconomical a concrete culvert founded on rock is a satisfactory alternative.
6. *Upstream face protection:* several options are available for protection of the upstream face against wave erosion, ranging from traditional stone pitching with grouted joints through concrete facing slabs to the use of concrete block work, rock armoring and riprap.
7. *Embankments crest:* the top width of larger earthen dam should be sufficient to keep the seepage line well within the dam, when reservoir is full. The crest should have a width of not less than 5m, and should carry a surfaced and well-drained access road. The top width (W) of the earth dam can be selected as per the following recommendation:

$$W = \frac{H}{5} + 3 \qquad \text{, For very low dams} \qquad 4.1$$

$$W = 0.55\sqrt{H} + 0.2H \qquad \text{, For dams lower than 30m} \qquad 4.2$$

$$W = 1.65\left(H + 1.5\right)^{\frac{1}{3}} \qquad \text{, For dams higher than 30m} \qquad 4.3$$

Where: H is the height of the dam.

5.5. Seepage analysis

Seepage occurs through the body of all earthen dams and also through their pervious foundation. The phreatic surface of the seepage regime, i.e. line within the dam section below which there is positive hydrostatic pressures in the dam, must be kept well clear of the downstream face to avoid high pore water pressures which may promote slope instability.

The amount of seepage can be easily computed from the flow net, which consists of two sets of curves, known as '*Equipotential line*' and '*stream lines*', mutually perpendicular to each other. For homogeneous embankments dam, discharge per unit width (q) of the dam passing through a flow net is described as:

$$q = kH \frac{N_f}{N_d}$$

Where: H is the head differential.
N_f is number of stream lines.
N_d id number of Equipotential lines.

5.6. Determination of Phreatic Lines

It is absolutely essential to determine the position of the phreatic line, as its position will enable to determine the following:

i. The divide line between the dry (or moist) and submerged soil.
ii. The top stream line and hence, helps us in drawing the flow net.
iii. To ensure that the phreatic line doesn't cut the downstream face of the dam, which is extremely necessary for preventing softening of the dam.

A. Homogeneous dam section with horizontal filter

It has been found that the seepage line is pushed down by the filter and it is very nearly parabolic except near its junction with u/s face. Since the u/s face of the dam (i.e. GB in Figure 5-6) becomes an equipotential line when fully covered with water, the seepage line shall be perpendicular to the face near its junction point B.

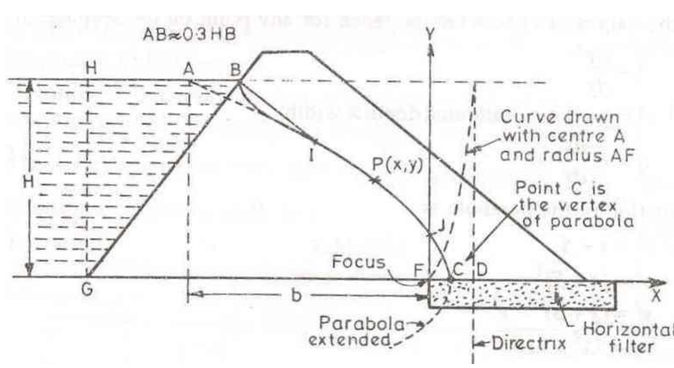


Figure 5-6 Seepage through homogenous dam section with horizontal filter

Equation of the base parabola

Let a base parabola with focus at F is drawn and produced so as to intersect the water surface at a point A as shown in Figure 1-1.Taking the focus (F) as the origin, equation of the parabola p(x, y) can be written as

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

Where; *FD* is the distance of the focus from the directrix, called focal distance and is represented by *S*.

Hence the equation of the parabola of the seepage line becomes:

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

Location of A is approximately 0.33HB horizontal distance upstream from point B according to Cassagrande. Where, H is the projection of the point G on the water surface.

If the horizontal distance between the already determined point A and the focus (F) is taken as say *b*, then (b, H) represents the coordinates of the point A on the parabola. And hence;

$$\sqrt{b^2 + H^2} = b + S$$

$$S = \sqrt{b^2 + H^2} - b$$

The center point (C) of FD will then be the vertex of the parabola. When x = 0, y= S. Hence the vertical ordinate FJ at F will be equal to S.
Knowing the points A, C, and J and working out a few more points from the equation, the parabola can be easily drawn and corrected for the curve BI, so as to get the seepage line BIJC.
The amount of seepage can also be calculated easily from the equation of the seepage line as derived below.

Darcy's law is defined as, $q = KiA$. When steady conditions have reached, the discharge crossing any vertical plane across the dam section (unit width) will be the same. Hence, the value *i* and *A* can be taken for any point on the seepage line

$$i = \frac{dy}{dx}$$

$$A = y * 1$$

$$q = K\frac{dy}{dx}y$$

But from the equation of the parabola,

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

$$q = K\left[\frac{1}{2}.(S^2 + 2xs)^{\frac{1}{2}-1}.2S\right].\left[\sqrt{S^2 + 2xs}\right]$$

$$q = KS$$

Example:

An earth dam made of a homogeneous material has a horizontal filter and other parameters as shown in the figure. Determine the phreatic line and the seepage quantity through the body of the dam.

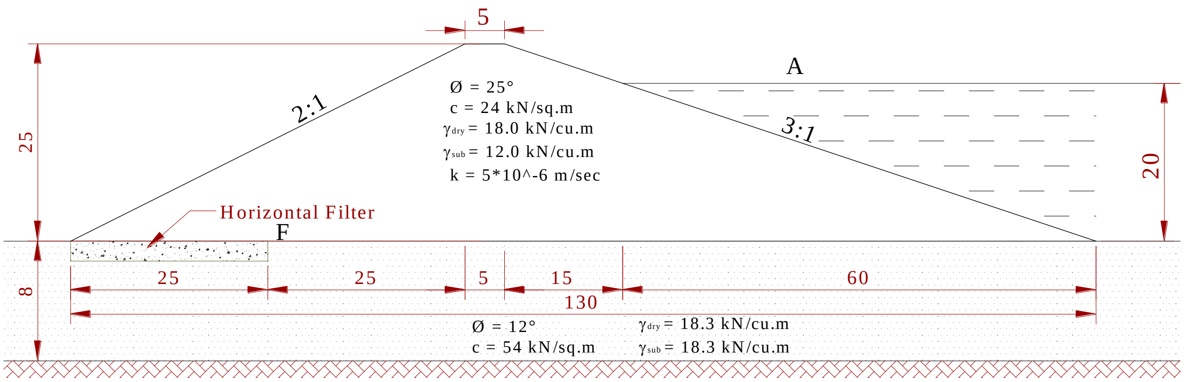


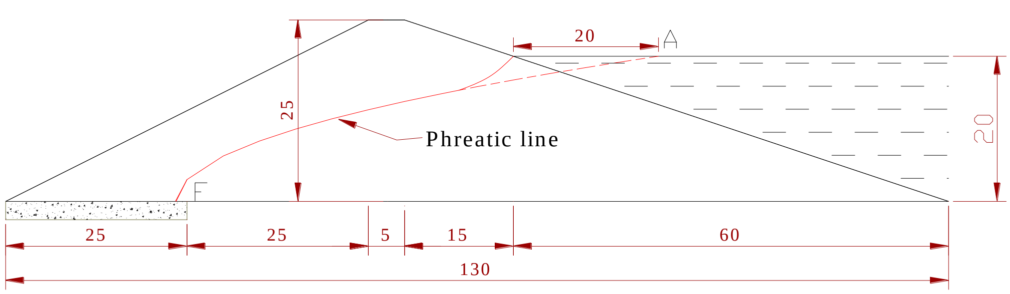
Figure 5-7 Section of a homogenous earth dam

For the origin of the Cartesian co-ordinate system at the face of the filter (point F), the equation of the parabola of the seepage line can be expressed as:

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

At point A, x = 65m, and y = 20m. Inserting into the parabola equation, S = 3.07m. Working out a few more points from the equation, the parabola can be easily drawn and corrected for the curve at the upstream face of the dam, so as to get the seepage line.

x	-1.51	0	10	15	25	30	40	45	55	65
y^2	0	9.06	69.26	99.36	159.56	189.66	249.86	279.96	340.16	400.36
y	0	3.01	8.32	9.97	12.63	13.77	15.81	16.73	18.44	20.01



The amount of seepage flow is

$Q = kS$

$= 5 * 10^{-6} * 3.07$

$= 15.35 * 10^{-6} m^3$/sec per meter width of dam

B. *Homogeneous dam section without horizontal filter*

The focus (F) of the parabola will be the lowest point of the downstream slope as shown in Figure 5-8. The base parabola BIJC will cut the downstream slope at J and extend beyond the dam toe up to the point C i.e. the vertex of the parabola.

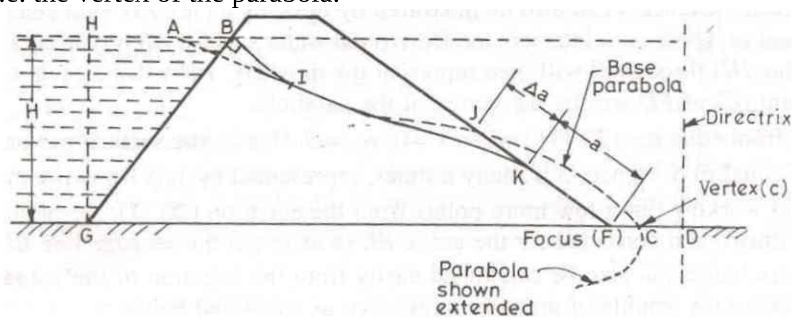


Figure 5-8 Homogeneous dam section without filter

The seepage line will, however, emerge out at K, meeting the downstream face tangentially there. The portion KF is known as discharge face and always saturated. The correction JK (say Δa) by which the parabola is to be shifted downward can be determined as follows:

α[‡] in degrees	$\frac{\Delta a}{a + \Delta a}$
30^o	0.36
60^o	0.32
90^o	0.26
120^o	0.18
135^o	0.14
150^o	0.10
180^o	0.0

α is the angle which the discharge face makes with the horizontal. a and Δa can be connected by the general equation;

$$\Delta a = (a + \Delta a)\left[\frac{180^o - \alpha}{400^o}\right]$$

Example

An earth dam made of a homogeneous material has the coefficient of permeability $K = 5*10^{-4}$ cm/sec and the other parameters are as shown in the Figure 5-9. Determine the phreatic line and the seepage quantity through the body of the dam.

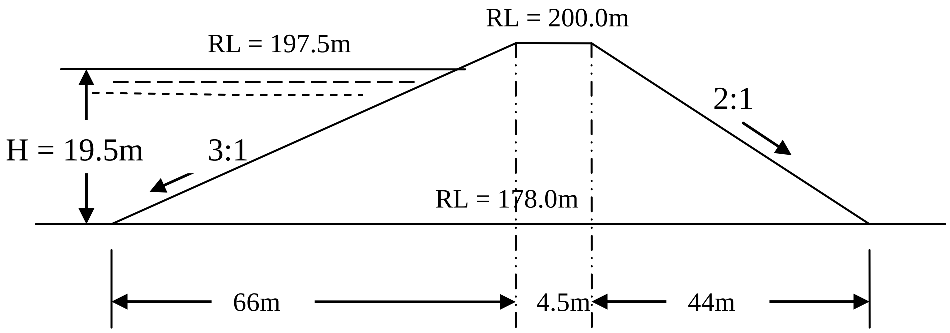


Figure 5-9 Body of homogeneous earth dam

‡ α will be equal to 180^o for a horizontal filter case and it will be less than 90^o when no drainage is provided.

5.7. Stability analysis

Three considerations govern the design of an earth embankment.

i. Side slopes must be stable;
ii. Dimensions must be sufficient to control seepage;
iii. Base width must be long enough to distribute weight of dam over sufficient area to prevent overstress in the foundation.

An earthen embankment usually fails because of the sliding of a large soil mass along a curved surface.

5.7.1. Stability of side slopes of earth dam

Forms of side slope failure:

Toe failure: - most likely to occur when the slopes are relatively steep or when the soil below the toe of the slope is strong.

Base failure: - occurs when the slopes are flat or when the soil below the toe is relatively weak.

Face or slope failure: - occurs only when there is a relatively weak zone in the upper part of the slope or when there is a very strong stratum above the toe level.

The method used for examining the stability of slopes of earthen embankments is called the *Swedish Slip Circle Method* or the *Slices Method*. It assumes the condition of plane strain with failure along a cylindrical arc. The location of the centre of the possible failure arc is assumed. The earth mass is divided into a number of vertical segments called slices as shown in Figure 5-10, *O* is the center and *r* is the radius of the possible failure.

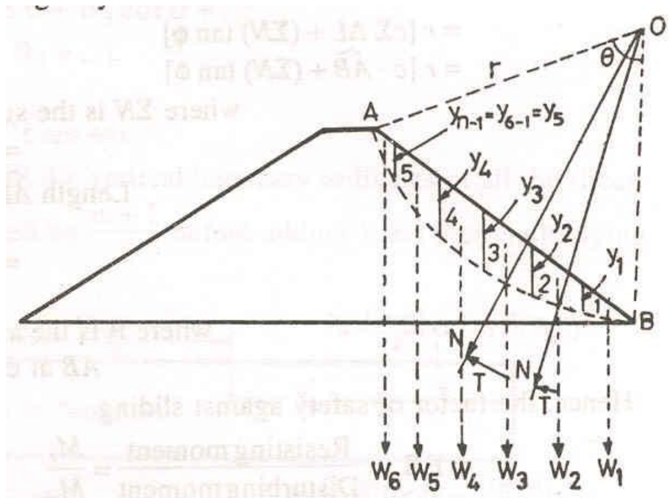


Figure 5-10 Possible slip surface in Earth fill dam

The side slopes of earth dam will be stable if the soil mass is not dislodged from the slopes. However, the soil mass in an earthen dam is subjected to forces which tend to cause movement or sliding of the soil mass. These forces are known as actuating, driving or disturbing forces which mainly consists of gravity forces. The movement or sliding of the soil mass in an earthen dam is resisted by the resisting or stability forces which are provided mainly by the shearing strength of the soil.

The stability of the side slope of an earthen dam is thus analyzed by assuming a surface slippage within the soil mass and by determining the resisting and the disturbing forces acting on this surface and the moments of these forces about the center of rotation, and then factor of safety against sliding is calculated.

The forces acting on the slices are:

1. The self weight W of the slice acting vertically downward through the center of gravity.
2. The cohesive forces acting tangentially opposite to the direction of probable slippage
3. The soil reaction across the arc. When the soil mass is about to slide, the soil reaction will act at an angle ϕ (the angle of internal friction of the soil) to the normal i.e. radial direction
4. The soil reaction on the two vertical sides of the slice exercised by the adjacent slices on the right and left respectively.
5. Pore pressures at the base of the arc, and left and right side of the slice.

Usually it is assumed that the soil reactions on the two vertical sides of the slice cancel each other and so also the pore pressures on the two sides balance each other

The disturbing force is the component of weight of slice in tangential direction i.e.,

$$T = W \sin \alpha ,$$

Where: *α is the angle which the slope makes with the horizontal.*

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

$$\sum T = T_1 + T_2 + T_3 + \ldots.$$

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

$$M_d = \sum T_i r_i = r \sum T_i$$

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

$$= c\Delta L + N \tan \phi$$

Where; *c* is the unit cohesion of the soil

ΔL is curved length of the slice

Φ is the angle of internal friction

N is equal to *Wcosα*

The total resisting force will be summation of resisting forces for all slices;

$$= \sum c\Delta L + \sum N \tan \phi$$
$$= c\sum \Delta L + \left(\sum N\right)\tan \phi$$

The total resisting moment over the entire sliding surface will be equal to

$$M_r = r\left(c\sum \Delta L + \left(\sum N\right)\tan \phi\right)$$

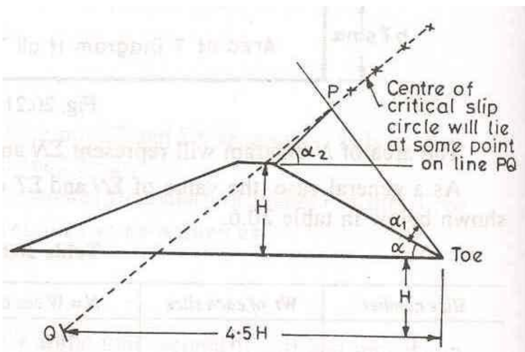
Hence the factor of safety against sliding

$$FS = \frac{M_r}{M_d} = \frac{c\sum \Delta L + \tan \phi \sum N}{\sum T}$$

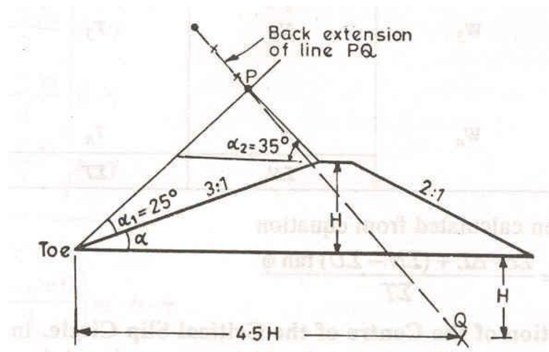
For determining the stability of the proposed side slope of an earth dam it is necessary to find the least factor of safety which may occur on any of the possible surfaces of slippage or slip circles. Slip circle which yields the least factor of safety is the most critical and hence it is known as critical surface slippage or critical slip circle. For locating the critical surface of slippage, it is necessary to try several different surfaces of slippage as one trial gives the value of factor of safety for that arc only.

For preliminary analysis 4 to 5 slices may be sufficient; however, 10 to 15 slices are considered in general. It is not necessary for the analysis to make all the slices of equal width, but for the sake of convenience it is customary to have slices of equal width.

In order to reduce the number of trials, Fellenius has suggested a method of drawing a line, representing the locus of the critical slip circle. The determination of this line PQ is shown in Figure 5-11. The point P is obtained with the help of directional angles α_1 and α_2 as shown in Table 5-4



a. downstream slope



b. upstream slope

Figure 5-11 locus of critical circle

Table 5-4 Slope and respective directional angle

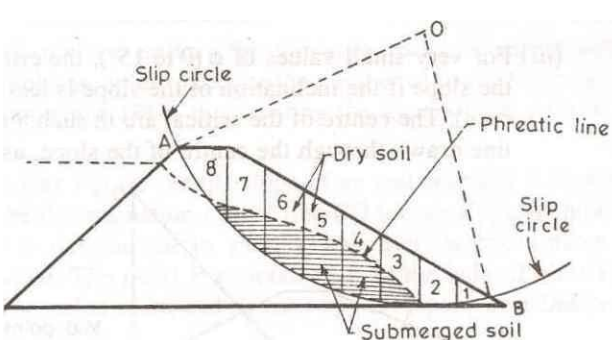
Slope	*Directional angles*	
	α_1 in degrees	*α_2 in degrees*
1:1	27.5	*37*
2:1	25	*35*
3:1	25	*35*
4:1	25	*35*
5:1	*25*	*35*

Design parameters to be employed in stability analysis may be summarized as follows:

a. Stability of downstream slope during steady seepage (reservoir full)

The most critical condition for the d/s slope occurs when the reservoir is full and the seepage is taking place at full rate.

The seepage water below the phreatic line exerts a pore pressure on the soil mass which lies below the phreatic line, see Figure 5-12.



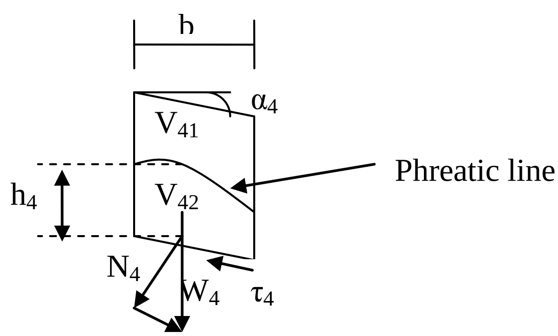


Figure 5-12 stability of Downstream slope during steady seepage

Consider slice number 4 in Figure 5-12, the weight of the slice is defined as;

$$W_4 = \gamma_{dry} * V_{41} + \gamma_{sat} * V_{42}$$

α_4 read from the scaled drawing of the earth fill dam. And tangential component of W_4 is defined as, whish is shear stress developed at failure plane,

$$T_4 = W_4 \sin \alpha_4$$

The pore pressure for slice 4 is represented by the piezometric head h_4. Hence pore water pressure is

$$U_{w4} = \gamma_w h_4$$

Shear strength developed for the slice is quantified from two soil parameters, apparent cohesion c and angle of shearing resistance ф. Shear strength at failure plane is defined as

$$\tau_4 = cL_4 + (W_4 \cos \alpha_4 - \gamma_w h_4 L_4) \tan \phi$$

Where; L_4 is $\dfrac{b}{\cos \alpha_4}$

The factor of safety of slide 4 is

$$FS_4 = \frac{T_4}{\tau_4} = \frac{cL_4 + (W \cos \alpha_4 - \gamma_w h_4 L_4) \tan \phi}{W_4 \sin \alpha_4}$$

The factor of safety for the entire circle is then given by the equation

$$FS = \frac{\sum c_i L_i + \sum (W_i \cos \alpha_i - \gamma_w h_i L_i) \tan \phi}{\sum W_i \sin \alpha_i}$$

b. Stability of Upstream Slope during sudden drawdown

For the u/s slope, the critical condition can occur, when the reservoir is suddenly emptied. In such case, the water level within the soil will remain as it was when the soil pores were full of water. The weight of this water within the soil now tends to slide the u/s slope along a circular arc.

The tangential components of the saturated soil lying over the arc will create a disturbing force; while the normal component minus the pore pressure shall supply the shear strength of the soil.

Table 5-5: General format of computation

Slice#	W	T	N	U	l	Ul	N'=N-ul	tanΦ	N'tanΦ	Cl	N'tanΦ+cl
1											
…											
n											
		Σ T									Σ (N'tanΦ+cl)

The factor of safety is finally obtained from the equation

$$FS = \frac{\sum c_i L_i + \sum N^{'} \tan \phi}{\sum T^{'}}$$

Where: $N^{'}$ represents normal components on submerged density

$T^{'}$ represents tangential components on saturated unit weight of the soil

5.7.2. Stability of earth dam against horizontal shear developed at the base of the dam

Approximate method for checking the stability of u/s and d/s slopes under steady seepage from consideration of horizontal shear at base

1. Stability of u/s slope during sudden drawdown

It is based on the simple principle that a horizontal shear force P_u is exerted by the saturated soil. The resistance to this force R_u is provided by the shear resistance developed at the base of the soil mass, contained within the u/s triangular shoulder GMN of Fig.

Considering unit length of the dam, the horizontal force Pu is

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

Where; $$\gamma_1 = \frac{\gamma_{sub} h_1 + \gamma_{dry}\left(h - h_1\right)}{h}$$

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

$$R_u = cB_u + W\tan\phi = cB_u + \left(\gamma_{sub}\frac{1}{2}B_u h\right)\tan\phi$$

Where; W is the weight of the u/s triangular shoulder of the dam

The factor of safety against can be easily calculated, using

$$FS = \frac{R_u}{P_u}$$

It should be more than 1.5.

2. Stability of d/s slope under steady seepage

It is based on the consideration of horizontal shear at base under the d/s slope of the dam. The horizontal shear force Pd is given by, referring Fig;

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

Where; $$\gamma_2 = \frac{\gamma_{sub} h_2 + \gamma_{dry}\left(h - h_2\right)}{h}$$

Shear resistance Rd of d/s slope portion of the dam is given by;

$$R_d = cB_d + W\tan\phi = cB_u + \left(\gamma_{dry}A_1 + \gamma_{sub}A_2\right)\tan\phi$$

The downstream profile RTS of the downstream slope portion of the dam has an area A1 and of dry soil above seepage line and the area of submerged soil say A2 below the seepage line.

The entire weight W may be calculated on the basis of submerged soil as it will be on a still safer side. In that case,

$$W = \gamma_{sub} B_d h$$

The factor of safety against shear can be easily determined as;

$$FS = \frac{R_d}{P_d}$$

5.8. Foundation analysis

Foundation stress in earth dams are not usually critical except when the foundation material consists of unconsolidated clay or silt with low shearing strength.

Consider a dam on homogeneous, unconsolidated earth foundation of thickness t

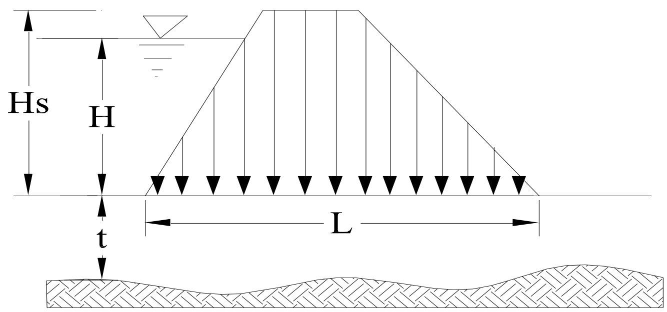


Figure 5-13 Homogenous embankment dam with pervious foundation of thickness t

The downward force exerted on the foundation at the center of the dam tends to squeeze the foundation material from under the dam. But shear stress develops in the foundation resisting this action. Assuming the foundation loading to vary as indicated above, Leo Jugenson suggested the following maximum stresses:

If $t > L$, $\tau_{max} = 0.256\gamma_f H_s$; Where γ_f = specific weight of fill

Usually $t < L$

If $t < L/10$, $\tau_{max} = \gamma_f H_s t/L$

Shear strength = $S_s = c + \sigma \tan\Phi$

The factor of safety against overstress is

$FS = S_s / \tau_{max}$

A minimum value of FS = 1.5 is recommended.

Example:

Design the embankment dam shown in Figure 5-7 used as an exaple for analysis of seepage flow. Detail all the necessary procedures and important consideration in the process.

Design Solution

The stability design process starts by determining the phreatic line profile which is done before. The critical slipage circle is then drawn by following the suggestion made by Fellinus. Here a single slipage circle is considered for illustration and four slices were considered for both upstream and downstream slope failure. The geometric informations were then determined as shown below.

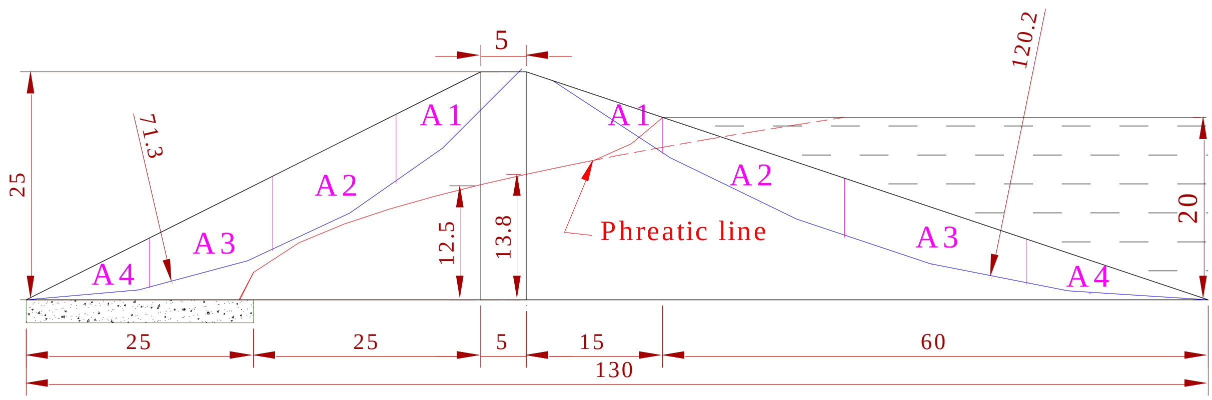


Fig. Sample of failure circle, slices and related measurments of the earth dam section

Geometric properties of slices

	Upstream				Downstream		
Slice	Area(m^2)	α (deg)	h_w (m)	L (m)	Area (m^2)	α (deg)	L (m)
A1	25.97	32	0	14.5	68.58	40	18
A2	111.68	24	6.1	22	111.48	28	15.4
A3	120.83	15	6.2	20.7	96.1	17	14.2
A4	55.52	4	3.6	20.1	40.11	7	13.7

Area in relation with phreatic line.

Area (m^2)	Dam	U/s shoulder	D/s shoulder
Under seepage line (saturated)	1102.08	838.1	221
Dry portion	584.92	99.4	404
Total	1687	937.5	625

To assess the overall stability of the dam considering 1m length,

	Dam		U/s shoulder		D/s shoulder	
Item	Area(m^2)	Weight(kN)	Area(m^2)	Weight(kN)	Area(m^2)	Weight(kN)
Under seepage line	1102.08	13224.96	838.1	10057.2	221	2652
Dry portion	584.92	10528.56	99.4	1789.2	404	7272
Total	1687	23753.52	937.5	11846.4	625	9924

The stability design then proceeds by first considering the entire embankment and its interaction with the foundation.

Shear resistance of the dam at the base(R)

$R = C + W \tan\Phi$

Where: C = total cohesive resistance of the soil at the base

$= c*B*1 = 24 * 130 * 1$

$= 3120$ kN

$W\tan\phi = 23753 * \tan 25^0$

$= 11076$ kN

$R = 3120 + 11076$

$= 14196$ kN

Horizontal force due to hydrostatic pressure of water

$P = ½ \gamma_w h^2 = ½ * 10 * 20^2$

= 2000kN

Factor of safety against failure due to horizontal shear at the base

FS = R/P = 7.1 > 1.5 Safe!

Check ***stresses*** in the foundation

t = 8m < L/10 = 130/10 =13,

Hence,

$\tau_{max} = \gamma_f H_s t/L$

$= 18.3 * 20 * 8/130$

$= 22.52 kN/m^2$

Shear strength $= S_s = c + \sigma \tan\Phi$

$= c + W/L \tan\phi$

$= 54 + 23754/130 * \tan 12^0$

$= 92.8 kN/m^2$

The factor of safety against overstress is

$FS = S_s/\tau_{max} = 92.8/22.52 = 4.12 > 1.5$ Safe!

Stability of u/s and d/s slopes against sliding shear.

Upstream slope (under sudden drawdown):

Considering unit length of the dam, the horizontal force P_u is

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

$$= \frac{12 * 13.8 + 18(25 - 13.8)}{25}$$

$$= 14.7$$

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

$$= \left[\frac{14.7 * 25^2}{2} \tan^2\left(45 - \frac{25}{2}\right) + 10 \frac{13.8^2}{2}\right]$$

$$= 2816.6$$

Shear resistance R_u of upstream slope portion of the dam developed at base GN is given by,

$$R_u = cB_u + W \tan\phi = 54 * 75 + 11846.4 * 0.47 = 9574.1$$

Where; W is the weight of the upstream triangular shoulder of the dam.

The factor of safety against shear can be easily calculated,

$$FS = \frac{R_u}{P_u} = \frac{9574.1}{2816.6} = 3.4 > 1.5$$ Safe!

It has been known that the maximum intensity of shear stress occurs at a distance $0.6B_u$ (where B_u is the base length of the upstream shoulder) from the heel and is equal to 1.4 times the average shear intensity.

Hence, maximum shear stress induced $(\tau_{max}) = 1.4(P_u/B_u)$

$= 1.4 (2816.6/75)$

$= 52.6$

The unit shear resistance developed at the same point is

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

$= 24 + 0.6 * 25 * 12 * \tan 25^0$
$= 107.9$

FS at the point of maximum shear should be greater than unity.

$FS = \tau_f / \tau_{max} = 107.9 / 52.6 = 2.0 > 1$ Safe!

For the downstream shoulder, similarly,

$$\gamma_1 = \frac{12 * 12.5 + 18(25 - 12.5)}{25} = 15$$

$$P_u = \left[\frac{15 * 25^2}{2} \tan^2\left(45 - \frac{25}{2}\right) + 10\frac{12.5^2}{2}\right] = 2683.7$$

$$R_u = cB_u + W \tan\phi = 54 * 60 + 9924 * 0.47 = 7904.3$$

$$FS = \frac{R_u}{P_u} = \frac{7904.3}{2683.7} = 2.9 > 1.5$$ Safe!

Maximum shear stress induced (τ_{max}) = $1.4(P_u/B_d)$
$= 1.4\ (2683.7/50)$
$= 75.1$

The unit shear resistance developed at the same point is

$\tau_f = c + 0.6h\gamma_{sub}\tan\phi$
$= 24 + 0.6 * 25 * 12 * \tan 25^0$
$= 107.9$

FS at the point of maximum shear should be greater than unity.

$FS = \tau_f / \tau_{max} = 107.9 / 75.1 = 1.44 > 1$ Safe!

Analysis of upstream and downstream slopes by Swedish Circle method:

Upstream slope

Slice	Area	α	W	T	N	U	L	UL	N'=N-ul	tanΦ	N'tanΦ	Cl	N'tanΦ+cl
A1	25.97	32	467.46	247.72	396.43	0	14.5	0	396.43	0.47	186.32	348	534.32
A2	111.68	24	1340.16	545.09	1224.3	61	22	1342	-117.7	0.47	-55.32	528	472.68
A3	120.83	15	1449.96	375.28	1400.55	62	20.7	1283.4	117.15	0.47	55.06	496.8	551.861
A4	55.52	4	666.24	46.47	664.62	36	20.1	723.6	-58.98	0.47	-27.72	482.4	454.68
			Σ T	1214.56							Σ (N'tanΦ+cl)		2013.54

$$FS = \frac{\sum c_i L_i + \sum N' \tan \phi}{\sum T'} = \frac{2013.54}{1214.56} = 1.66 > 1.5$$

Safe!

Downstream slope

Slice	Area	α	W	T	N	l	N'=N-ul	tanΦ	N'tanΦ	Cl	N'tanΦ+cl
A1	68.58	40	1234.44	793.48	945.64	18	945.64	0.47	444.45	432	876.45
A2	111.48	28	2006.64	942.06	1771.76	15.4	1771.76	0.47	832.73	369.6	1202.33
A3	96.1	17	1729.8	505.74	1654.22	14.2	1654.22	0.47	777.48	340.8	1118.28
A4	40.11	7	721.98	87.99	716.6	13.7	716.6	0.47	336.8	328.8	665.6
				2329.27					Σ (N'tanΦ+cl)		3862.66

$$FS = \frac{3862.66}{2329.27} = 1.66 > 1.5$$

Safe!

5.9. Internal drainage system

General

Purpose of drainage:

1. To reduce the pore pressure thereby increasing the stability of the dam
2. To prevent piping so that soil particles are not carried away from the embankment.

A drainage system consists of two components.

a. the protective filter

b. the conduit which collects and disposes of the seepage.

Protective filter: - serves to allow free drainage and to prevent erosion.

It is provided between

Riprap and embankment

Core and embankment

Embankment and drains

Experiments by Terzaghi, Bertram and others have shown that a filter need only hold the coarse 15% of the grain size. These coarse particles D_{85} and over, will collect over the filter opening bridging over it and trapping finer particles.

Size of filter holes, $D_f \leq D_{85}$ (of the soil being filtered)

From tests, the following criteria are established.

D_{15} (filter) ≤ 4 to 5 D_{85} (soil); to satisfy prevention of migration of soil particles.

D_{15} (filter) ≥ 4 to 5 D_{15} (soil); for free drainage.

Filter gradation curve should be smooth and parallel to the soil being filtered. When the soil is gap graded recompute and re-plot the grain size distribution using only the fraction finer than the break as representing the entire soil; apply the filter criteria to this distribution

Filter thickness:

Thin filter is desirable to minimize flow resistance. Practical considerations, however, put minimum sizes as shown in Table 5-6.

Table 5-6: Representative mean filter thickness

Filter	Thickness for given head, cm		
	0 – 25 m	25 – 50 m	50 – 100 m
Fine sand	15	30	45
Coarse sand	25.5	45	60
Gravel	30	60	75

For every fine grained soil a multiple layered filter is necessary. Each successive layer is designed to fit the D_{15} and D_{85} of the finer layer it must filter. The last layer must fit the openings of the drain conduit which carries the water away.

Example:

Determine the size of the bed material for the embankment dam with the grain size shown below.

Grain size	Riprap	Dam
D_{15}	70 mm	0.3 mm
D_{85}	130 mm	2.0 mm

Drain Conduit:

Function: to collect water from the filter and carry it away with as little head as possible.

Quantity of Flow: Estimated from the seepage analysis. A factor of safety of 5 is not uncommon.

Simplest conduit: uniform coarse fragmental material (coarse sand, gravel, crushed rock, etc.)

A properly designed filter must surround the drain. For high discharges or when suitable crushed rock is not available, pipe conduits wig perforated and flexible joints are employed.

Types of drain

Trench drain:

Trench drain is used for intercepting seepage through homogeneous foundations and those containing horizontal pervious strata or seams

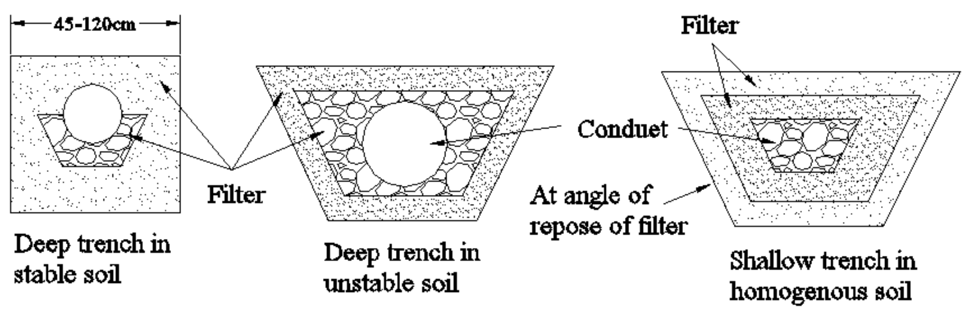


Figure 5-14 Trench Drain

Mound drain: used when the need for embankement drainage exceeds that of the foundation.

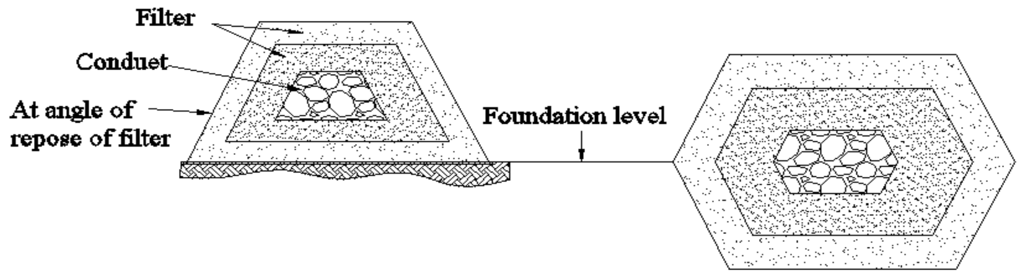


Figure 5-15 Mound drain

Position- when there is cut-off, the drain is placed immediately downstream from the cut-off to relieve any pressure build up.

When there is no cut-off, the position depends on the seepage analysis of dam and foundation.

Upstream location

Increases stability at the cost of increased seepage and cost of conduit.

Minimum distance from downstream toe to provide substantial increase in stability is 1/3 of the base width. Maximum distance is 2/3 of base width, from downstream toe.

Blanket drain:

This is horizontal drain placed on top of foundation.

To intercept water from vertical fissures in the foundation;

To lower the seepage line in the embankment

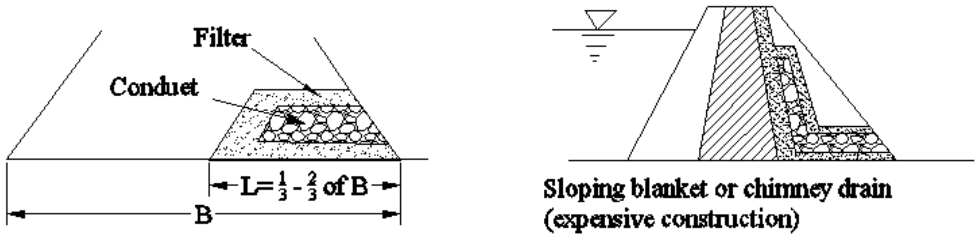


Figure 5-16 Blanket drain

Riprap:

Riprap is required on the upstream slope and the downstream slope below the tail water level. An estimation of the required weight of rock pieces required for riprap is given by Hudson as

$$W \geq (H2\ \rho st \tan \alpha) / (3.2\Delta 2)$$

Where: the factor 3.2 is for smooth quarry stone

ρst = density of rock

α = angle of slope

$\Delta = (\rho st - \rho w)/ \rho w$

ρw = density of water

Types of riprap

Dumped riprap: consists of angular broken rock dumped from truck and spread.

Hand-placed riprap: consists of more or less prismatic stone placed on end to form rough pavements.

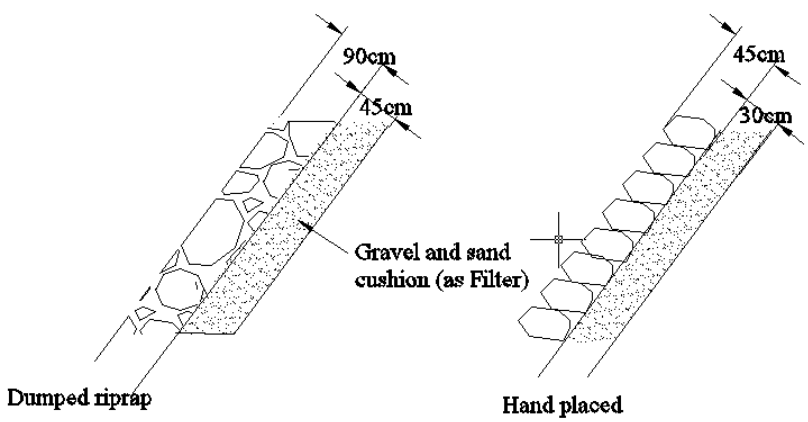


Figure 5-17 Types of riprap

6 Spillways

6.1 General

Spillway is the most important component of the dam which serves to release excess flood from a reservoir efficiently and safely. It is the most expensive of all the appurtenances structure. Its capacity is determined from the hydrological studies over the drainage area.

Spillway components include;

a. Entrance channel: to minimize head loss and to obtain uniform distribution of flow over the spillway crest
b. Control structure: to regulate and control the outflow. It may consist of a sill, weir, orifice, tube, or pipe.
c. Discharge channel: to convey the discharge from the control structure to the terminal structure/stream bed. The conveyance structure may be the downstream face of a concrete dam, an open channel excavated along the ground surface, a closed cut-and-cover conduit placed through or under a dam, or a tunnel excavated through an abutment.
d. Terminal structure: to dissipate excess energy of the flow in order to avoid scouring of the stream bed
e. Outlet channel: to safely convey the flow from the terminal structure to the river channel.

Types of spillway taking the hydraulic as criteria are broadly

a. Controlled (Gated) spillway: a spillway having a certain means to control the outflow from the reservoir.
b. Uncontrolled (Ungated) spillway: is a spillway, the crest of which permits water to escape automatically, as the water level in the reservoir rises above the crest.

Taking the most prominent feature as criteria, spillway types are

a. Free overfall (straight drop) spillway
b. Ogee (overflow) spillway
c. Side channel spillway
d. Siphon spillway
e. Chute (open channel or trough) spillway
f. Drop inlet (shaft or morning glory) spillway

6.2 Types of Spillway

6.2.1 Free overfall (straight drop) spillway

A free overfall spillway has a low height narrow crested weir as control structure and a vertical or nearly vertical downstream face. The overflowing water may be discharged as in the case of a sharp crested weir or it may be supported along the narrow section of the crest. However, in either case the water flowing over the crest of this spillway drops as a free jet clearly away from the downstream face of the spillway. Occasionally the crest of free overfall spillway is extended in the form of an overhanging lip to direct small discharges away from the downstream face of the overfall section. The underside of the nappe is ventilated sufficiently to prevent pulsating fluctuating jet.

If no artificial protection is provided on the downstream side of the overflow section, the falling jet usually cause the scouring of the streambed and will form a deep plunge pool. To protect the stream bed from scouring, an artificial pond may be created by constructing a low auxiliary dam downstream of the main structure or by excavating a basin which is then provided with a concrete apron. However, if tailwater depths are sufficient, a hydraulic jump will form when the jet falls freely from the crest, in which case a sufficiently long flat apron may be provided. In addition, floor blocks and an end sill may be provided in this case to help in the establishment of the jump and thus reduce the downstream scour.

The free overfall spillway is used:

i. most commonly for low earth dams (or earthen bunds),

ii. for thin arch dams,

iii. or other dams having nearly vertical downstream face and would permit free fall of water, and

iv. where, in general, the hydraulic drops from head pool to tailwater are not in excess of about 6m.

However, free overfall spillways are not suitable for high drops on yielding foundations, because the apron will be subjected to large impact forces at the point of impingement. The impact force causes vibrations which may crack or displace the apron and may result in failure by piping or undermining.

6.2.2 Ogee (overflow) spillway

The ogee spillway has a control weir which is ogee or S-shaped in profile. The profile is derived from the lower envelop of the overall nappe flowing over a high vertical rectangular notch with an approach velocity $V_o \approx 0$ and a fully aerated space beneath the nappe ($P = P_o$).

The following crest profile has been found to give good agreement with the prototype measurement by U.S. Waterways Experimental Station (WES). Such shapes are known as WES Standard Spillway Shapes as shown in Fig 6-1.

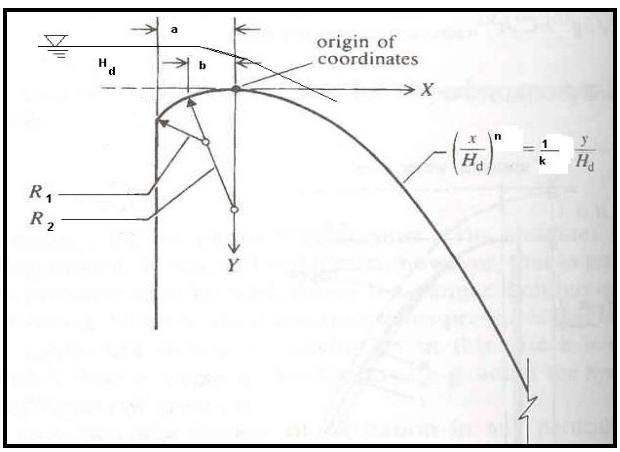


Fig 6-1 WES Standard Spillway Shapes

Table 6-1 Values of a, b, R_1, R_2,K and n for different U/S slope

U/S slope	*a/H_d*	*b/H_d*	*R_1/H_d*	*R_2/H_d*	*K*	*n*
0H:3V	0.175	0.282	0.50	0.20	-0.5	1.85
1H:3V	0.139	0.237	0.68	0.21	-0.516	1.836
2H:3V	0.115	0.214	0.48	0.27	-0.515	1.81
3H:3V	0	0.199	0.45	∞	-0.534	1.776

The spillway discharge is given by:

$$Q = CL_e (H + H_v)^{3/2}$$

Where: Q- discharge

C- Coefficient which depends on u/s and d/s flow condition (1.65-2.5)

L_e- effective crest length

H- head on the crest

H_v- approach velocity head

Where crest priers and abutments are shaped to cause side contractions of the overflow, the effective length, L_e, will be less than the net length of the crest. The effect of the end contraction may be taken into account by reducing the crest length as follows:

$$L_e = L' - 2(NK_p + K_a)(H + H_V)$$

Where: L'- net length of the crest

N- Number of piers

K_p- piers contraction coefficient

K_a- abutment contraction coefficient

The pier contraction coefficient, K_p, is affected by the shape and location of the pier nose, the thickness of the pier, the head in relation to the design head, and the approach velocity. The average pier contraction coefficient may be assumed as follows:

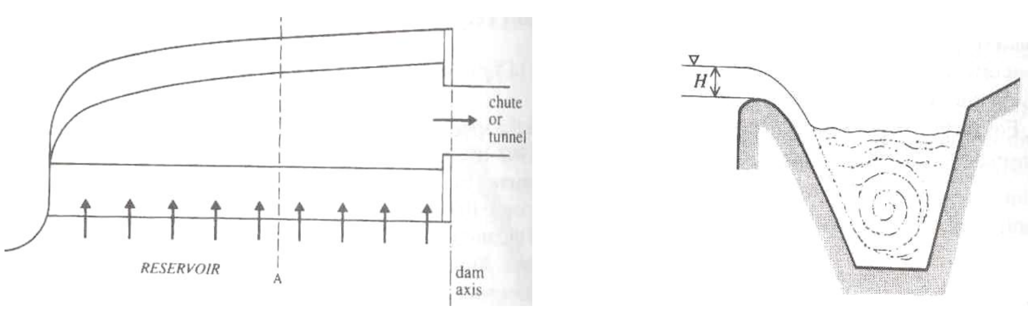
Pier condition	$\mathbf{K_p}$
Square nosed pier with corners rounded on a radius equal to about 0.1 of the pier thickness	0.02
Rounded nosed piers	0.01
Pointed nose piers	0

The abutment contraction coefficient is affected by the shape of the abutment, the angle between the upstream approach wall and the axis of flow, and the head in relation to the design head, and the approach velocity. The average abutment contraction coefficient may be assumed as follows:

Abutment condition	**Ka**
Square abutments with head wall at 90^o to direction of flow	0.20
Rounded abutments with head wall at 90^o to the direction flow	0.10
Rounded abutments with head wall placed at not more than 45^o to the direction of flow	0

6.2.3 Side channel spillway

Side channel spillways (Fig 6-2) are mainly used when it is not possible or advisable to use a direct overfall spillway as, e.g., at earth and rock fill dams.



(a) (b)

Fig 6-2 Side channel spillway: (a)Plan (b) section A-A, side view

They are placed on the side of the dam and have a spillway proper, the flume (channel) downstream of the spillway, followed by the chute or tunnel. The spillway proper is usually designed as a normal overfall spillway. The depth, width, and bed slope of the flume must be designed in such a way that even the maximum flood discharge passes with a free overfall over the entire horizontal spillway crest, so that the reservoir level is not influenced by the flow in the channel. The width of the flume may therefore increases in the direction of the flow. From the energy dissipation point of view, the deeper the channel and the steeper the side facing the spillway, the better; on the other hand , this shape is in most cases more expensive to construct than a shallow wide channel with a gently sloping side.

6.2.4 Siphon spillway

Siphon spillways (Fig 6-3) are closed conduits in the form of an inverted U with an inlet, short upper leg, throat (control section), lower leg, and outlet.

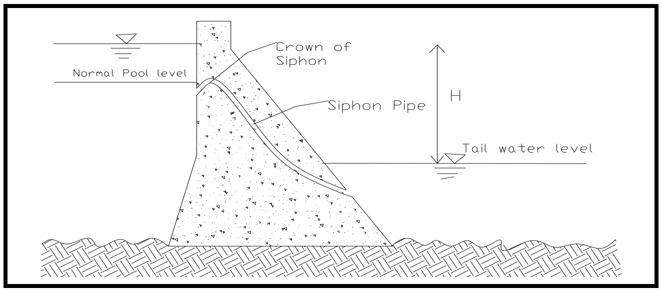


Fig 6-3 siphon pipe installed within the gravity dam

For very low flows a siphon spillway operates as a weir; as the flow increases, the upstream water level rises, the velocity in the siphon increases, and the flow in the lower leg begins to exhaust air from the top of the siphon until this primes and begins to flow full as a pipe, with the discharge given by

$$Q = C_d A(2gH)^{1/2}$$

Where: A is the (throat) cross-section of the siphon, H is the difference between the upstream water level and siphon outlet or downstream water level if the outlet is submerged and

$$C_d = 1/(K_1 + K_2 + K_3 + K_4)^{1/2}$$

Where: K1, K2, K3, and K4 are head loss coefficients for the entry, bend, exit, and friction losses in the siphon.

6.2.5 Chute spillway

A chute spillway is a steep channel conveying the discharge from a low overfall, side channel, or special shape spillway over the valley side into the river downstream.

For earthen and rock fill dams, a separate spillway is generally constructed in a flank or a saddle, away from the main valley. Sometimes, even for gravity dams, a separate spillway is required because of the narrowness of the main valley. In all such circumstances, a separate spillway like chute could be provided.

A chute spillway essentially consists of a steeply sloping open channel, placed along a dam abutment or through a flank or a saddle. It leads the water from the reservoir to the downstream channel below.

The entire channel spillway can hence be divided into the following parts:

I. Entrance channel
II. Control structure (Low Ogee weir)

III. Chute channel or discharge carrier
IV. Energy dissipation arrangements at the bottom in the form of the stilling basin

6.2.6 Shaft spillway

A shaft ('morning glory') spillway consists of a funnel-shaped spillway, usually circular in plan, a vertical (sometimes sloping) shaft, a bend, and a tunnel terminating in an outflow as shown in Fig 6-4.

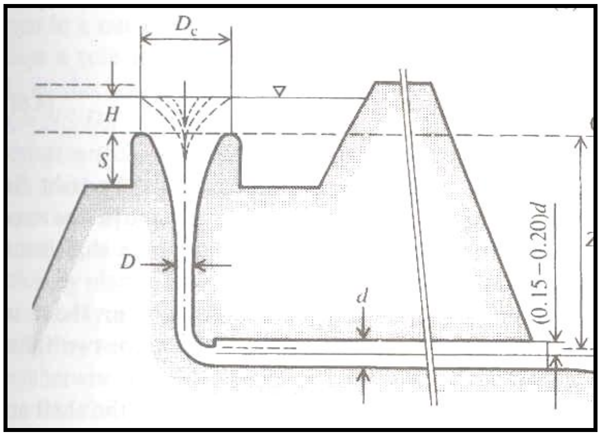


Fig 6-4 Shaft spillway

The shape of the shaft spillway is derived in a similar manner to the overfall spillway from the shape of the nappe flowing over a sharp-edged circular weir. Clearly, in this case the shape for atmospheric pressure on the spillway is a function of H_s/D_s, where H_s is the head above the notch crest of the diameter D_s. For ratios $Hs/D_s < 0.225$ the spillway is free-flowing and for $H_s/D_s > 0.5$ the overflow is completely drowned. For the free overfall the discharge is given by

$$Q = \frac{2}{3} C_d \pi D_c \sqrt{2g} H^{3/2}$$

And for the drowned (submerged) regime, the discharge is given by

$$Q = \frac{1}{4} C_{d1} \pi D^2 [2g(H + Z)]^{1/2}$$

Where: D is the shaft diameter, D_c is the crest diameter ($D_c<D_s$), H is the head of the reservoir level above the crest ($H<H_s$), Z is the height of the crest above the outflow from the shaft bend, C_d and C_{d1} are discharge coefficients.

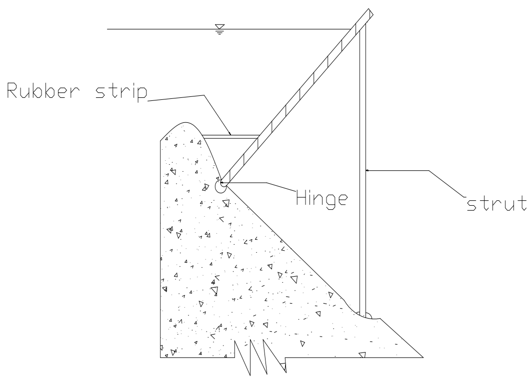
6.3 Spillway Crest Gates

Various types of gates have been evolved to control the flow of water over the spillway when the reservoir is full. The common types of gates are:

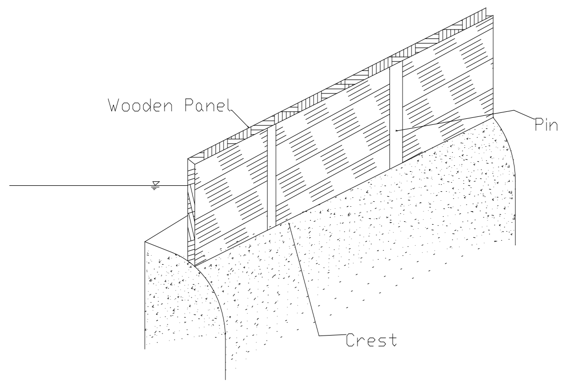
i. Flashboards
   - Temporary
   - permanent
ii. Stop logs & needles
iii. Rectangular lift gates
iv. Radial (Tainter) gates
v. drum gates
vi. Rolling (roller) gate
vii. Tilting (Flap) gate

Flash boards, Stop logs and needle are the simplest and oldest types of movable gates used for small heads. The rest are used for major works.

Flashboards: A temporary arrangement of flash boards for heads of 1.20 to 1.50m consists of individual wooden panels supported by vertical pins that are expected to carry a certain predetermined head of water and bend and fail when that head is exceeded (Fig 5.5(b)). A permanent arrangement may be a hinged flash board made up of panels which can be raised or lowered from an overhead cable way without damage to it (Fig 5.5(a)). The panels are supported by wooden struts.



(a)



(b)

Fig 6-5 Flashboards

Flash boards have the advantage that an unobstructed crest is provided when they and their supports are removed. However, they have the following disadvantages.

i. They present a hazard if not removed in time to pass floods, especially where the reservoir is small and the stream is subject flash boards
ii. They require the attendance of operator or crew to remove them, unless they are designed to fail automatically
iii. If they are designed to fail when the water reaches certain stages their operation is uncertain, and when they fail they release sudden and undesirable large outflows
iv. Ordinarily they cannot be restored to position while the flow is passing over the crest
v. If the spillway function frequently, the repeated replacement of flashboards may be costly

Stop logs: they are timber planks spanning horizontally between vertical grooves in adjacent piers (Fig 5.6). They are built up one on another, a vertical bulkhead formed from the crest of the spillway to the headwater level. The timber planks may vary in size from short , which can be handled by one man to sizes limited by the span and the capacity of a power which to raise them. These gates are used for small installation.

Stop logs must be removed before the floods occur, or they must be arranged so that they can remove while being overtopped.

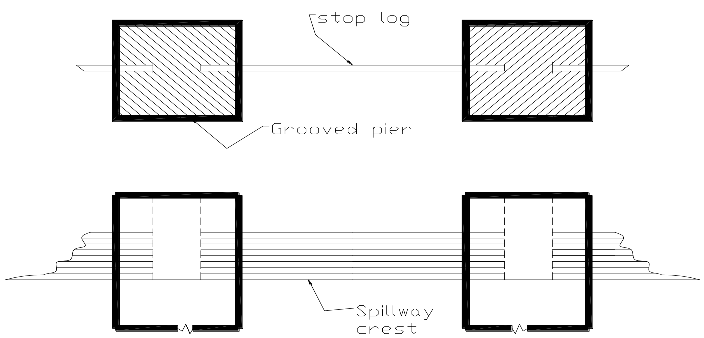


Fig 6-6 Stop logs

Needles are timber planks set on end side by side to close an opening (Fig 5.7). They are supported from top by a runway (bridge), from which they are handled, and at the bottom they are resting in a key way on the spillway crest. Needles are difficult to place in swift waters of considerable depth, but they are easier to remove than stop logs. The arrangement may present a hazard to the safety of the dam if the stop logs are not removed in time to pass floods, especially where the reservoir is small and the stream is subject to flash floods.

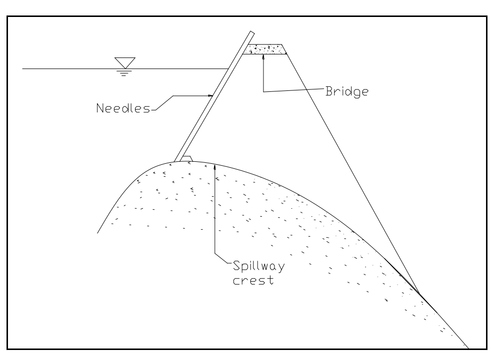


Fig 6-7 Needles

Rectangular lift gate: it is a simple timber or steel gate on the crest of a dam which span horizontally between the guide grooves in the supporting piers. The guides may be placed either vertically or inclined slightly downstream. The gate is raised or lowered by a host mounted on a bridge overhead. It consists of a framework to which a skin plate is attached, normally on the upstream face. The high friction on the guides limits its size since a relatively large hoisting capacity is required to operate the gate. Sliding friction is reduced by means of rollers. Depending on the method of providing the rollers, lift gates are classified into fixed wheel gates and Stoney gates.

Fixed wheel gate: In this type the roller are mounted on the downstream face of the gate. Axle friction as well as roller friction exists in this case.

Stoney gate: In this type a train of roller is placed between the side walls of grooves on the piers and the downstream face of the gate (Fig 6.8). The train of rollers is neither attached to the gate nor the side walls of the grooves. It is supported in the space in between the two by means of chain which passes over a pulley. One end of the chain is attached to the counter weight and the other to the gate. An advantage of the arrangement is that the frictional forces are nearly eliminated except at a negligible amount of roller friction.

Forces to be considered in a lift gate are hydrostatic force on the gate, the hoisting force, the weight of the gate and the roller friction.

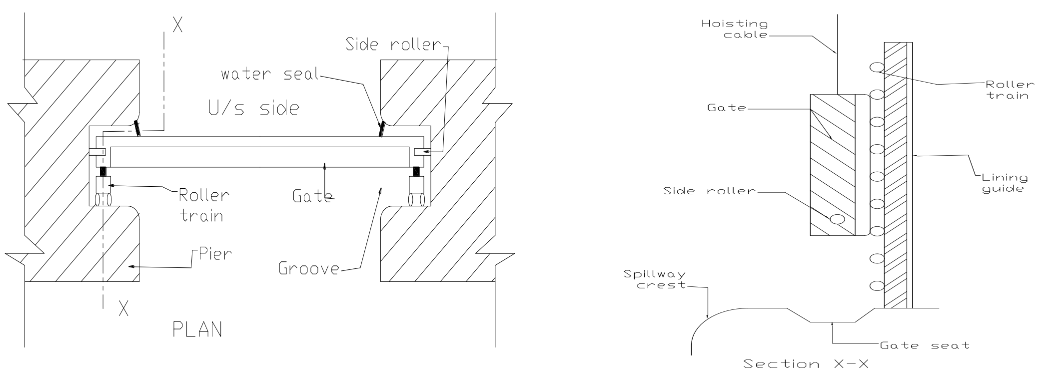


Fig 6-8 Stoney gate

Radial (Tainter) gate: It is a steel frame work with a circular (cylindrical) segmental plate for its face which is attached to supporting bearings by radial arms. It rotates about its center of curvature. All the hydrostatic forces are radial, i.e., pass through the center of curvature (trunion bearing) of the gate.

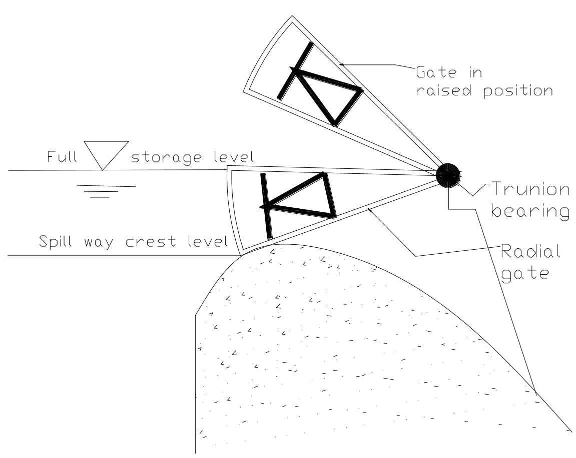


Fig 6-9 Radial gate

The housing load consists of the weight of the gate, the friction between the side seals and piers and the frictional resistance at the pins. The gate is often counter weighted to partially counter balance the effect of its weight, which further reduces the capacity of of the hoist. The small hoisting effort needed to operate the gate makes hand operation practical on small installation which otherwise might require power. The small hoisting forces involved also make the radial gate more adaptable to operation by relatively simple automatic control apparatus.

Sizes of radial gates vary from 1 to 10m in height and 2 to 20m in length.

Rolling (Roller) gate: it is a steel cylinder spanning between spillway crest piers. It is opened by rolling up an inclined toothed rack on the piers. A cylindrical segment is commonly attached to the lower limbs of the roller to give greater height of the gate.

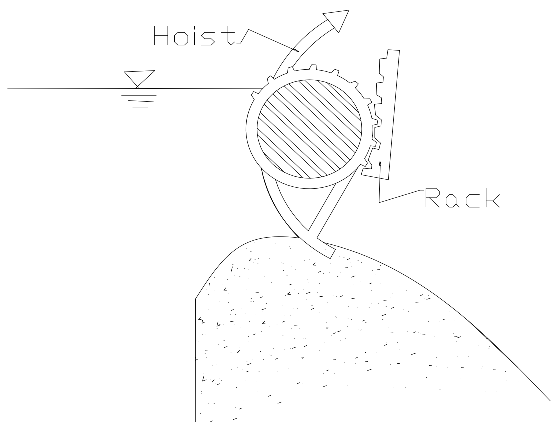


Fig 6-10 Rolling gate

Drum gate: this is an acute circular sector in cross-section, formed by skin plates attached to internal bracing. It is hinged at the center of curvature which may be either upstream or downstream. In the lowered portion, it fits in the shape of the crest.

Its crest is lowered automatically by rising headwater. The plates are so proportioned the headwater in the recess supports the movable structure against the headwater pressure above the sill at low water. When the water surfaces rises, the head water pressure becomes relatively grate on the upper skin and the crest is depressed. As the water level falls to the low water level, the crest returns to that level.

Drum gate is not adapted to small dams because of the large recess required.

Tilting (Flap) gate: It consists of a flat frame hinged at the lower edge. The upper edge can be moved with the help of chains or rods about the lower hinge to pass the flood over the crest of the gate. It is normally suitable for small sizes of openings.

Weight of gate:

$$W = KL^m H^n$$

Where L – gate opening (ft)

H- depth of water against the gate (ft)

K, m, n - constants

Type of gate	**m**	**n**	**Range of K**	**Mean K**
Rectangular lift	1.5	1.75	0.80-2.00	1.2
Radial	1.9	1.35	0.85-1.45	1.16
Rolling	1.5	1.67	2.40-3.40	2.85
Drum	1.33	1.33	26.00-35.00	31.00

6.4 Spillway design

6.4.1 Background

A multi-purpose medium dam is planned to be constructed on the selected location. The dam is intended to serve for irrigation, power generation and control of flood which has been destroying property worth millions of birr whenever it occurs.

The overall construction of the project and its future implementation is well accepted by the society living in the surrounding and the catchment. The socioeconomic importance of the structure is deeply understood by the people and the cooperation of them is granted before hand by the assessment done for this particular purpose. This is done because there were structures built with huge amount of investment and couldn't be functional just because of the community unawareness.

A site, which is ideal for the construction of any type of dam, is obtained in a narrow gorge. Then the contour map of the site is properly prepared and the different spillway options are considered. A preliminary cost analysis is done for the various types. The preference of each option is justified based on the site and social condition prevailed in the surrounding and the fund available for the construction of the dam

The pertinent dimensions of the dam are decided considering different parameters and conditions. For instance the dam height is selected based on the following conditions.

iv. The fetch of the dam upstream will be prone to flooding and nearby villages will be submerged if the specified height is exceeded.
v. The amount of storage obtained at this elevation is comparable with the demand projected.
vi. The budget allocated for the project could not go beyond this limit.
vii. The available spilling options function optimally in the elevation already selected.

Therefore, after the appropriate dam height and site is selected the type of dam needs to be decided upon. For the ogee spillway a gravity dam is proposed along the dam. For the rest cases of spillways an earth dam can be constructed for it is quite easy to obtain construction material from queries around the dam. These make the earth dam much economical as compared to the former.

Much of the design procedure is based on the USBR Design of Small Dams and on the experimental results of the Waterways Experiment Station. Various tables and charts were used from these references.

General description of the dam site and the available data.

	Elevation m.a.s.l
Bottom of the dam	1390.0
Top of the dam crest	1428.0
Ogee spillway crest	1420.0
Ogee spillway design water level	1425.0
Chute spillway design water level	1425.0
Normal reservoir water level	1419.0

Dam and spillway

Type: Gravity for the case of ogee spillway and arch or Earth and rock fill (Zoned) with side channel, siphon spillway or chute spillway.

Length of dam	96.0 m
Design discharge (1000 years return period)	1410 m3/s
Height of dam	35.0 m
Mean annual temperature	250C
Vapour pressure of water	3.595kpa

AlternativeI

Free overflow ogee spillway.

For the free overflow ogee a sound rock foundation is assumed to exist for the construction of the gravity dam and a ski jump is found to be satisfactory at the toe of the ogee for the dissipation of energy. From the topography it is observed that there is no need for the construction of an approach channel.

Design data

Design discharge (Q) = 1410 m3/s

River bed elevation = 1390 m

The design head is 6m, but a negative pressure head of 1.0 m is assumed to develop in the crest of the spillway for economic reasons and the workmanship is assumed to be good enough not to create rough surface for this negative head to result in cavitation problem. The vapor pressure of water for the spillway site is 3.595m

Therefore, from the negative pressure head (h_u) specified the corresponding design head (h_{des}) is

$$h_u = h(1-h/h_{des})$$
$$-1. = 6(1- 6/h_{des})$$
$$h_{des} = 5.14 \text{ m}$$
$$P/h = 6$$

This value (P/h = 6) hence the effect of approach velocity is too small and can be neglected. But a case where the dam is filled by sediment is considered and P is decreased. Therefore P is assumed to be 2m.

$$P/h = 2/6$$
$$= 0.333$$

The respective value of C_o (coefficient of discharge) from chart is

$$C_o = 2.175$$
$$q_o = C_oH^{1.5}$$
$$= 2.175 * 6^{1.5}$$
$$= 32 \text{ m}^3\text{/s/m}$$
$$v_o = q/(P + h)$$
$$= 32/(2+6)$$
$$= 4 \text{ m/sec}$$

Velocity head (h_a)

$$h_a = v_o^2/2*g$$
$$= 16/19.62$$
$$= 0.81 \text{ m}$$

adding 10% of h_a for entrance and other losses

$$h_a = 0.9 \text{ m}$$

Therefore, H_e = 6.9 m

Correction for the coefficient of discharge

P/H_e = 0.29

C_o = 2.18 hence, no appreciable change from the previous value.

For an upstream slope of 2:3

$$C_i/C_o = 1.026$$

Submergence effect is not considered here because the downstream apron is much below the crest level for any submergence to occur for the design discharge. For similar reason the correction for downstream apron is not carried out.

Therefore, the final corrected value of the coefficient of discharge for the ogee is

$C = 2.18 * 1.026$

$= 2.23$

From the discharge equation by Polini

$Q = CL'He1.5$

$1410 = 2.23 * L' * 6.91.5$

$L' = 35.00m$

For the provision of round nosed piers ($k_p = 0.01$) at every 8m interval along the ogee
Number of piers required = 4
Pier thickness is 2m
Rounded abutments with headwalls at 90^0 to the direction of flow are used ($k_a = 0.1$)
The effective length of the crest will then be

$L = L' + 2(nkp + ka)H$

$= 35.0 + 2(4*0.01 + 0.1) * 6.9$

$= 36.93 \approx 37.0m$

Adding the pier width the total width of the crest will be

$B = 37.0 + 8$

$= 45.0$ m

The profile of the nape is determined based on the charts available on USBR design of small dams.

$H_a/H_e = 0.9/6.9$

$= 0.13$

For an upstream slope of 1:1
crest position

$X_c/H_e = 0.195$

$X_c = 1.35m$

$Y_c/H_e = 0.07$

$Y_c = 0.49m$

Profile upstream of the crest

$R_1/H_e = 0.465$

$R_1 = 3.21m$

$R_2/H_e = 0.367$

$R_2 = 2.53m$

Down stream of the crest

$Y/H_e = -k(X./H_e)^n$

values of the constants are found(from charts on USBR) to be

$K = 0.52$

$n = 1.763$

$y = -0.119\ x^{1.763}$

Tabulating values for the above equation,
The point of tangency in the downstream for a slope of m= 0.6
The value of a is obtained from table (a = 1.80)

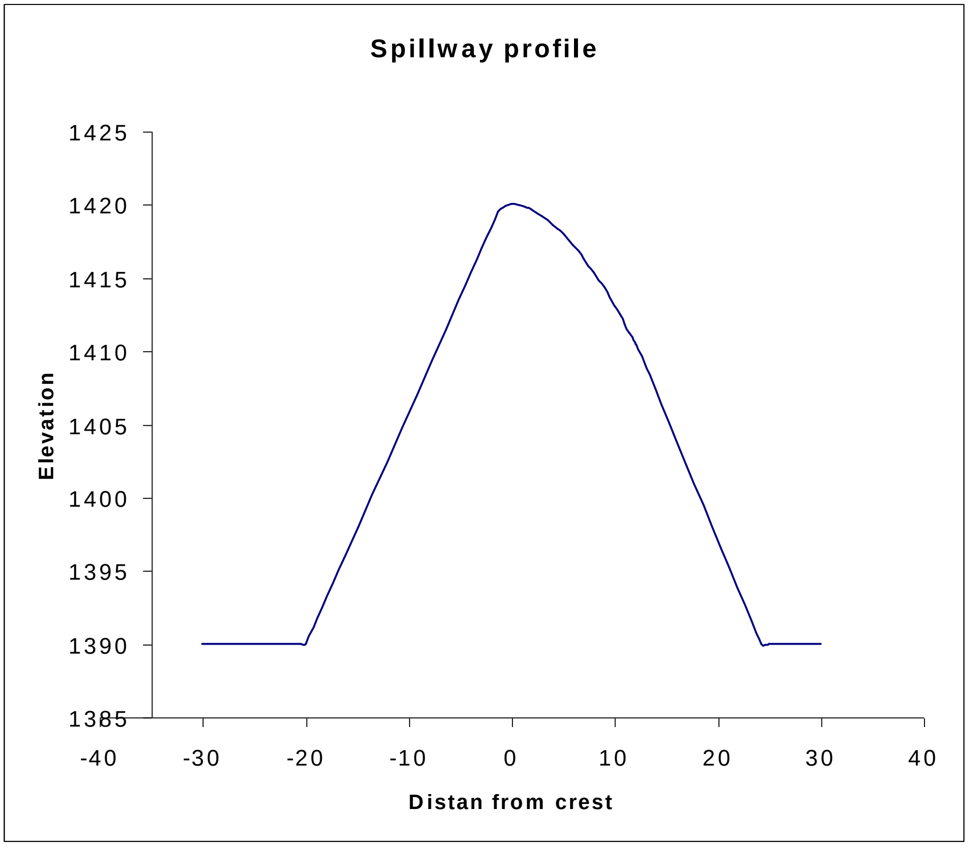
$Y_T = -H_eK(mkn)^{n/(1-a)}$

$= -9.4m$

The coordinate values obtained so far for the ogee nape profile are tabulated and plotted as follows.

Table 1.1 Computation of ogee profile.

X	Elevation
-30	1390
-20.5	1390
-20	1390
-1.35	1419.5
-1	1419.7
0	1420
1.3	1419.8
2	1419.6
2.5	1419.4
3	1419.2
3.5	1418.9
4	1418.6
4.5	1418.3
5	1418
5.5	1417.6
6	1417.2
6.5	1416.8
7	1416.3
7.5	1415.8
8	1415.3
8.5	1414.8
9	1414.3
9.5	1413.7
10	1413.1
10.5	1412.5
11	1411.8
11.5	1411.2
12	1410.5
13	1409.1
24.28	1390
25	1390
30	1390



Alternative II

Siphon spillway

Siphon spillway is a bit complicated to construct and the low elevation difference between the upstream and downstream water level makes it unsuitable for most dams..

For this particular case a site is selected which is suitable for its construction and a tail water elevation necessary for the proper functioning of the spillway is maintained by taking advantage of the topography and building an additional structure.

Design procedure

Siphon cross section at the throat

width (b) 5m

depth (a) 4m

normal water surface elevation is the crest elevation 1420m

design head is as used in the case of the ogee 6m

P_v = 3.595m

The minimum and maximum pressures at the crest elevation

P_{omin} = 88kpa

P_{omax} = 108 kpa

H_s = 1420 m

$$P'_{o\,min} = P_{o\,min}\, e^{(-Hs/6956)}$$

$$= 71.75\text{kpa}$$

$$P'_{o\,max} = P_{o\,max}\, e^{(-Hs/6956)}$$

$$= 88.06\text{kpa}$$

The possible negative head to avoid capitation in the siphon is

$$h_{cp} = \frac{P'_{o\,min} - P_v}{\gamma}$$

$$h_{cp} = \frac{71.75 - 3.56}{9.81}$$

$$h_{cp} = 6.95\,\text{m}$$

Hence the design head for the siphon discharge is taken to be 6.9m

for a concrete hood the roughness coefficient (ε) is taken to be 1.5

the total length of the hood is assumed to be 25m

the hydraulic diameter (D) will be

D = 4A/P

= (4*20)/18

= 4.44 m

ε/D = 0.00034

from Moody's chart for the corresponding value of ε/D, the friction factor(f) for a rough flow behavior (since the Reynolds number is supposed to be very large),

f = 0.0145

Now siphon coefficient (μ) will be determined

$$\mu = \frac{1}{\sqrt{1 + fL / D_H + \Sigma\xi}}$$

where $\Sigma\xi$ is the intake exit and bend loss

$$\mu = \frac{1}{\sqrt{1 + 0.0145 * 25 / 4.44 + 0.4}}$$

$$\mu = 0.82$$

the discharge capacity of the siphon will be

$$Q = ab\mu(2gh)^{1/2}$$
$$= 4*5*0.82 * (2 * 9.81 * 6.9)^{1/2}$$
$$= 190.8\ m^3/s$$

For a siphon hood of crest profile radius 3m and trough radius of 7m the maximum possible discharge will be

$$Q_{maxi} = b\ r_i \ln(1+a/ri)\ [2g(h_{cp}+h)]^{1/2}$$
$$= 5*3*\ln(1 + 4/3)\ [19.62\ (6.9 + 3)]^{1/2}$$
$$= 177.1\ m^3/s$$

$$Q_{maxo} = b(r_i + a)\ln(1+a/r_i)\ [2g(h_{cp} + h - a)]^{1/2}$$
$$= 5(3 + 4)\ \ln(1 + 4/3)\ [19.62\ (6.9 + 4 – 4)]^{1/2}$$
$$= 345\ m^3/s$$

The smaller of the two is used for design purposes, hence Q = 177.1 is taken. The respective maximum head will be

$$H_{max} = \frac{Q^2_{max}}{2g(ab\mu)^2}$$

$$H_{max} = \frac{177.1^2}{19.62(16.4)^2}$$

$$H_{max} = 5.94\ m$$

Hence, the head to be used for the design of the siphon is 5.94m.

Alternative III
Side channel spillway

The topography of the dam site reveals that there is a possibility to construct a side channel spillway. An iterative approach is followed to determine the water surface profile as well as the critical section.The bed slope is the optimum slope obtained after a minimum adjustment is carried out.

Available data

Length	70.00
Design head	6.00
Discharge per length	20.14
Side slope m	0.50
Bottom width	10.00
Manning’s n	0.02
Alpha	1.00
Bed slope	0.30
Crest elevation	1420.00

Critical profile for the given channel condition is simulated for fictious flow depth to be used in obtaining the actual profile through interpolation

Table 3.1. Critical flow depth computation.

Y	**A**	**T**	**D/2**	**V_c**	**Q_c**	**P**	**R_c**
1.00	10.50	11.00	0.48	3.07	32.24	12.24	0.86
1.50	16.13	11.50	0.70	3.71	59.82	13.35	1.21
3.00	34.50	13.00	1.33	5.11	176.30	16.71	2.06
4.00	48.00	14.00	1.71	5.79	277.92	18.94	2.53
5.00	62.50	15.00	2.08	6.39	399.38	21.18	2.95
6.00	78.00	16.00	2.44	6.92	539.76	23.42	3.33
7.00	94.50	17.00	2.78	7.39	698.36	25.65	3.68
8.00	112.00	18.00	3.11	7.81	874.72	27.89	4.02
9.00	130.50	19.00	3.43	8.20	1070.10	30.12	4.33
10.00	150.00	20.00	3.75	8.58	1287.00	32.36	4.64
11.00	170.50	21.00	4.06	8.93	1522.57	34.60	4.93
12.00	192.00	22.00	4.36	9.25	1776.00	36.83	5.21
13.00	214.50	23.00	4.66	9.56	2050.62	39.07	5.49
14.00	238.00	24.00	4.96	9.86	2346.68	41.30	5.76
15.00	262.50	25.00	5.25	10.15	2664.38	43.54	6.03
16.00	288.00	26.00	5.54	10.43	3003.84	45.78	6.29
17.00	314.50	27.00	5.82	10.69	3362.01	48.01	6.55
18.00	342.00	28.00	6.11	10.95	3744.90	50.25	6.81
19.00	370.50	29.00	6.39	11.20	4149.60	52.49	7.06
20.00	400.00	30.00	6.67	11.44	4576.00	54.72	7.31
21.00	430.50	31.00	6.94	11.67	5023.94	56.96	7.56

The critical water surface profile with respect to the critical bed is calculated in Table 2 then the critical depth is transferred to the actual bed slope. An arbitrary elevation (1470 m) is selected and the water surface profile at critical flow is plotted in fig 1.The lowest point of tangency of the actual channel bed with the critical bed profile is taken as the control section. The actual flow profile is then determined by going upstream and downstream from the critical section for subcritical and supercritical flow conditions respectively as shown in Table 3.1.

Table 3.2. Critical flow profile for the given side channel.

X	Δx	Q1	Q1+Q2	Y_c	V_c	V1+V2	ΔQ	ΔV	Y'_m	R_c	h_f	ΔY'	ΣΔY'
0.00													
5.00	5.00	100.71	100.71	2.07	4.20	4.20	100.71	4.20	0.00	1.51	0.02	0.00	0.00
10.00	5.00	201.43	302.14	3.31	5.28	9.48	100.72	1.08	2.05	2.18	0.02	2.07	2.07
15.00	5.00	302.14	503.57	4.26	5.91	11.19	100.71	0.63	1.64	2.61	0.02	1.66	3.73
20.00	5.00	402.86	705.00	5.03	6.40	12.31	100.72	0.49	1.41	2.96	0.02	1.43	5.16
25.00	5.00	503.57	906.43	6.03	6.79	13.19	100.71	0.39	1.25	3.23	0.02	1.27	6.43
30.00	5.00	604.29	1107.86	6.58	7.36	14.15	100.72	0.57	1.34	3.68	0.02	1.36	7.79
35.00	5.00	705.00	1309.29	6.06	7.54	14.90	100.71	0.18	1.01	3.82	0.02	1.03	8.82
40.00	5.00	805.71	1510.71	6.89	7.72	15.26	100.71	0.18	0.93	3.96	0.02	0.95	9.77
45.00	5.00	906.43	1712.14	8.24	7.91	15.63	100.72	0.19	0.88	4.10	0.02	0.90	10.67
50.00	5.00	1007.14	1913.57	9.02	8.09	16.00	100.71	0.18	0.83	4.24	0.02	0.85	11.52
55.00	5.00	1107.86	2115.00	9.27	8.27	16.36	100.72	0.18	0.80	4.38	0.02	0.82	12.34
60.00	5.00	1208.57	2316.43	10.00	8.45	16.72	100.71	0.18	0.77	4.52	0.02	0.79	13.13
65.00	5.00	1309.29	2517.86	10.15	8.61	17.06	100.72	0.16	0.73	4.67	0.02	0.75	13.88
70.00	5.00	1410.00	2719.29	10.82	8.76	17.37	100.71	0.15	0.70	4.79	0.02	0.72	14.60

Table 3.3. Flow profile computation.

X	Δ x	Z_o	Δy	Z	Y	A	Q	V	Q1+Q2	V1+V2	ΔQ	ΔV	$\Delta Y_m'$	R	h_f	ΔY'
subcritical flow profile																
30.00	0.00	1411.00		1418.36	7.36	100.68	604.29	6.00								
25.00	5.00	1412.50	0.86	1419.22	6.72	89.78	503.57	5.61	1107.86	11.61	100.72	0.39	0.86	3.59	0.010	0.87
20.00	5.00	1414.00	0.99	1420.21	6.21	81.38	402.86	4.95	906.43	10.56	100.71	0.66	0.99	3.41	0.009	1.00
15.00	5.00	1415.50	0.99	1421.20	5.70	73.25	302.14	4.12	705.00	9.07	100.72	0.83	0.98	3.22	0.006	0.99
10.00	5.00	1417.00	0.90	1422.10	5.10	64.00	201.43	3.15	503.57	7.27	100.71	0.97	0.90	2.99	0.004	0.90
5.00	5.00	1418.50	0.76	1422.86	4.36	53.10	100.71	1.90	302.14	5.05	100.72	1.25	0.76	2.69	0.002	0.76
0.00	5.00	1420.00	0.37	1423.23	3.23	37.52	0.00	0.00	100.71	1.90	100.71	1.90	0.37	2.18	0.000	0.37
Super critical flow																
30.00	0.00	1411.00		1418.36	7.36	100.68	604.29	6.00								
35.00	5.00	1409.50	0.59	1417.77	8.27	116.90	705.00	6.03	1309.29	12.03	100.71	0.03	0.59	4.10	0.010	0.60
40.00	5.00	1408.00	0.35	1417.42	9.42	138.57	805.71	5.81	1510.71	11.85	100.71	-0.22	0.35	4.46	0.008	0.35
45.00	5.00	1406.50	0.22	1417.20	10.70	164.25	906.43	5.52	1712.14	11.33	100.71	-0.30	0.21	4.84	0.007	0.22
50.00	5.00	1405.00	0.15	1417.05	12.05	193.10	1007.14	5.22	1913.57	10.73	100.71	-0.30	0.14	5.23	0.005	0.15
55.00	5.00	1403.50	0.10	1416.95	13.45	224.95	1107.86	4.92	2115.00	10.14	100.71	-0.29	0.10	5.61	0.004	0.10
60.00	5.00	1402.00	0.08	1416.87	14.87	259.26	1208.57	4.66	2316.43	9.59	100.71	-0.26	0.08	5.99	0.004	0.08
65.00	5.00	1400.50	0.06	1416.81	16.31	296.11	1309.29	4.42	2517.86	9.08	100.71	-0.24	0.06	6.37	0.003	0.06
70.00	5.00	1399.00	0.00	1416.81	17.81	336.70	1410.00	4.19	2719.29	8.61	100.71	-0.23	0.04	6.76	0.002	0.04

Table 3. 4. Flow profile for

X	Drop from EGL	Cr. water level	Bed level	Cr. Bed profile	Actual profile
0.00	1470.00	1420.00	1420.00	1470.00	1423.23
5.00	1470.00	1420.57	1418.50	1467.93	1422.86
10.00	1467.93	1420.31	1417.00	1464.62	1422.10
15.00	1466.27	1419.76	1415.50	1462.01	1421.20
20.00	1464.84	1419.03	1414.00	1459.81	1420.21
25.00	1463.57	1418.53	1412.50	1457.54	1419.22
30.00	1462.21	1417.58	1411.00	1455.63	1417.58
35.00	1461.18	1415.56	1409.50	1455.12	1417.77
40.00	1460.23	1414.89	1408.00	1453.34	1417.42
45.00	1459.33	1414.74	1406.50	1451.09	1417.20
50.00	1458.48	1414.02	1405.00	1449.46	1417.05
55.00	1457.66	1412.77	1403.50	1448.39	1416.95
60.00	1456.87	1412.00	1402.00	1446.87	1416.87
65.00	1456.12	1410.65	1400.50	1445.97	1416.81
70.00	1455.40	1409.82	1399.00	1444.58	1416.81

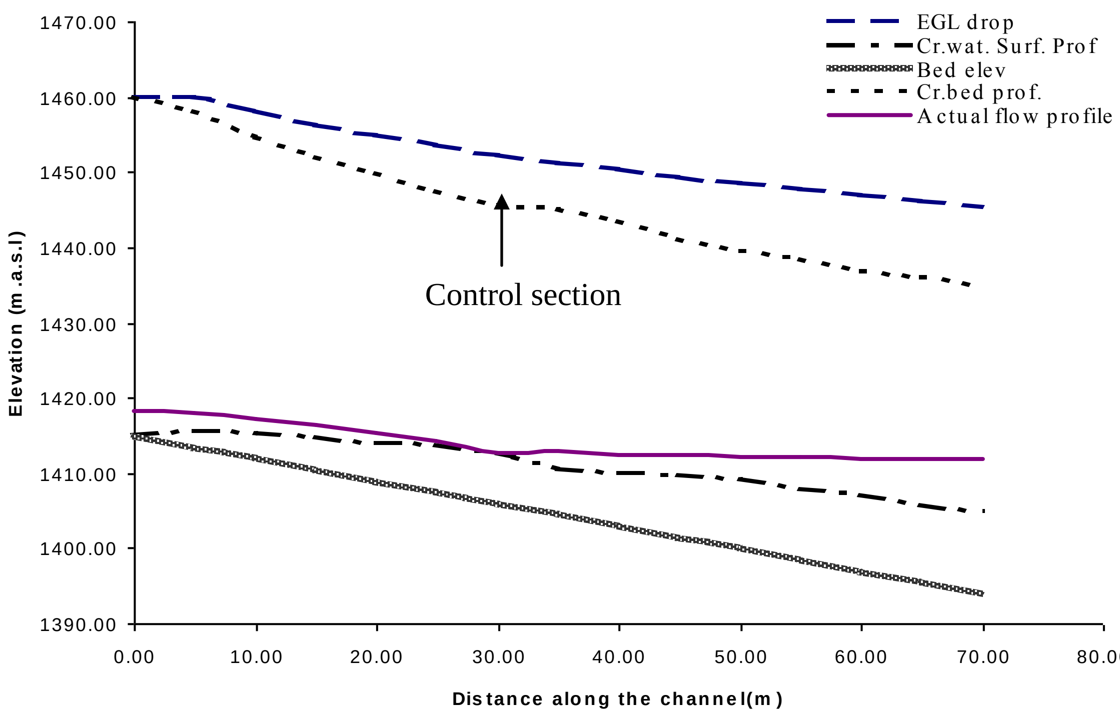


Fig. 1. Computation of flow profile for side channel

Alternative IV

Chute spillway

For the design of the chute spillway three components are considered

1. Design of the approach channel.
2. Design of the control structure.
3. Design of the chute channel.

Spillway crest length is optimized to be 40m.

Round nosed piers of 2m thickness will be used to at every 9m along the spillway.

An approach channel of side slopes 1:1 is suggested to be used to guide the channel to the control structure.

The height of the control ogee is 6m.

Elevation of bottom of the control structure is 1414m.

Approach channel

For the design of the approach channel first the head over the control structure/ogee/ need to be determined. From the equation of discharge

$Q = CL_eH_e^{3/2}$

Assume the value of the discharge coefficient C = 2.13.

The design head hdes is equal to the total head.

$1410 = 2.13 * 32 * h^{1.5}$

h = 7.54

Upstream water surface level = crest elevation + h
= 1420 + 7.54
= 1427.54

bed level of the ogee = 1414m

water depth upstream of ogee = 13.54 m

channel width is = 40m

area of the channel = (40 + 13.54) * 13.54
= 724.9 m2

wetted perimeter P = 40 + 2*1.414* 13.54
= 78.3

hydraulic radius R = A/P
= 9.26 m

v = Q/A
= 1.94

ha = v2/ 2g
0.1923

Correction for the coefficient of discharge

$P/H_e = 0.78$

$C_o = 2.128$

For an upstream slope of 1:1

$C_i/C_o = 1.004$

To remove any submergence effect at the downstream apron position

$h_d+d/H_e = 1.777 > 1.7$

and to maintain supercritical flow

$h_d/H_e = 1.34$

Therefore, the final corrected value of the coefficient of discharge for the ogee is

C = 2.128 *1.004
= 2.13

Design of crest profile

$P/H_e = 0.78$

$h_d/H_e = 0.025$

The upstream profile of the crest is ,therefore, obtained by interpolating for ha/He = 0.025

Table 4.1 coordinates of upstream profile for low ogee weir

X/He	Ha/He			x	Elevation
	0	0.08	0.12		
0	0	0	0	0.00	1220.00
-0.02	0.0004	0.0004	0.0004	-0.15	1420.00
-0.06	0.0036	0.0035	0.0035	-0.46	1420.00
-0.1	0.0103	0.01	0.0099	-0.77	1419.99
-0.12	0.015	0.015	0.0147	-0.93	1419.99
-0.14	0.0207	0.0208	0.0149	-1.08	1419.98
-0.15	0.0239	0.0235	0.0231	-1.16	1419.98
-0.16	0.0275	0.027	0.0265	-1.24	1419.97
-0.175	0.0333	0.0328	0.0325	-1.35	1419.97
-0.19	0.0399	0.0395	0.039	-1.47	1419.96
-0.195	0.0424	0.042		-1.51	1419.96
-0.2	0.045			-1.54	1419.96

The optimum position of the downstream apron is taken as 1414.0m because it is sufficient to maintain critical flow and avoid submergence effects. From the specific energy at the upstream and downstream of the control structure the depth of flow at the toe of the ogee is obtained

U/s E = 6.0 + 7.54 + 0.19

= 13.73m

velocity at the d/s

= q/d

d/s $E = d + (q/d)^2/2g$

equating and solving by trial and error the water depth is 2.3m

For the downstream profile of the ogee the equation obtained from table is

$$Y = -0.115x^{1.75}$$

And the values are calculated for the elevation range of 1420m and 1414m as shown in the following table.

Table 4.2 Downstream profile of ogee

x	Elevation
0	1420.00
0.50	1419.97
1.00	1419.89
2.00	1419.61
3.00	1419.21
4.00	1418.70
5.00	1418.08
6.00	1417.35
7.00	1416.54
8.00	1415.62
9.00	1414.00

Design of the chute.

Critical depth $y_c = (q^2/g)^{1/3}$

$= 5.02m$

The critical depth is much higher than the calculated depth, hence the flow is supercritical. The chute channel should now be given a milder slop for a little distance from toe, keeping the flow in supercritical condition.

Critical velocity = $q/y_c = 7.02$

From Manning's equation

$V = 1/n\ R^{2/3}s^{1/2}$

Inserting the values and calculating for slope

S=0.001

Hence a slope of 1/400 is provided in 40m distance from the toe of the spillway. The bed level at the end of this slope is 1414-0.1 = 1413.9m.

For the reverse curve at the toe a concave curve of radius 2He is provided.

For the remaining reaches of the channel a slope is given based on the prevailing topography formation of white water and keeping the flow supercritical. The calculation is shown in the following table.

Table 4.3 Calculation of water depth on chute channel.

S.N	Distance from start	Length (L)	Drop in bed(Y_m)	Bed level (Z_o)	Depth (d)	Velocity (v)	vel head (h_a)	Sp. energy (E)	Z_o +E	Area (A)	R	h_f	Act TEL	Froud no
Slop 1:6														
1	0			1413.90	2.36	14.94	11.37	13.73	1427.63	23.60	1.60			3.10
2	20	20	3.32	1410.58	2.06	17.11	14.92	16.98	1427.56	20.60	1.46	1.28	1426.35	3.81
	40	20	3.32	1407.26	1.85	19.05	18.50	20.35	1427.61	18.50	1.35	1.76	1425.81	4.47
Slop 1:3														
3	60	20	6.66	1400.60	1.66	21.23	22.98	24.64	1425.24	16.60	1.25	2.43	1425.19	5.26
	80	20	6.66	1393.94	1.54	22.89	26.70	28.24	1422.18	15.40	1.18	3.04	1422.20	5.89
Slop 1:2														
4	100	20	10	1383.94	1.39	25.36	32.78	34.17	1418.11	13.90	1.09	4.15	1418.03	6.87
	120	20	10	1373.94	1.30	27.22	37.76	39.06	1413.00	12.95	1.03	5.15	1412.96	7.64

Convex curves joining the different slopes were then designed as follows.

i. joining slope 1:400 and 1:6
ii. joining 1:6and 1:3
iii. joining slope 1:3 and 1:2

The design is based on the equation

$$y = -x\tan\theta + \frac{x^2}{1.5[4(d+h_v)\cos^2\theta]}$$

where tanθ = the upstream slope(dy/dx)

cos θ = 1

d + hv = the u/s specific energy

the value of x(the limit of the curve) is obtained by differentiating y with respect to x and equating with the upstream slope. Additional points will be obtained by evaluating y at intermediate points.

The detail is shown for the case of curve i.

$$y = -x/400 + \frac{x^2}{1.5[4(13.73)*1]}$$

$$dy/dx = -\tan\theta + \frac{2x}{1.5[4(d+h_v)\cos^2\theta]}$$

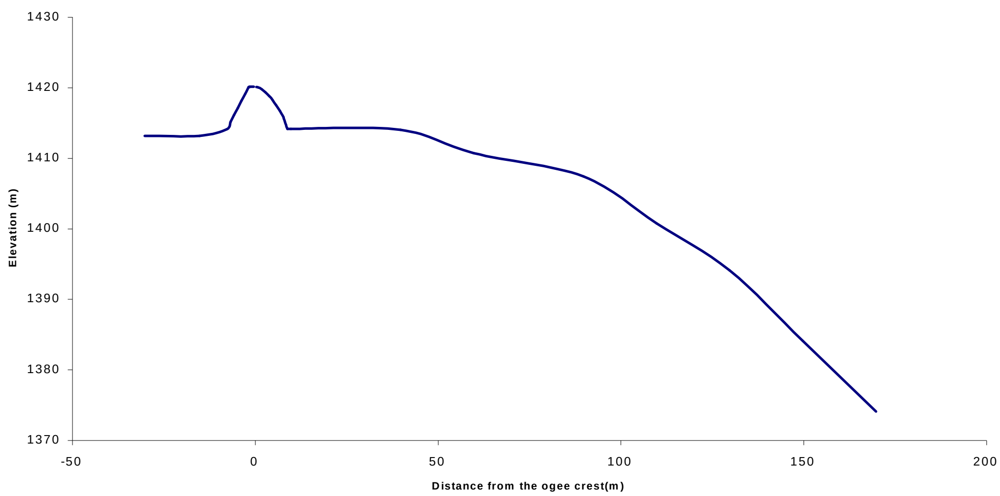
$$dy/dx = -1/400 + \frac{2x}{1.5[4(13.73)]} = 1/6$$

$$x = 26.94$$

$$y = -4.56\,m$$

Table 4.4. Curve terminal points

curves	u/s slope	d/s sloe	u/s E	x	Y
1	0.0025	0.166	13.73	0.00	0.00
				5.00	-0.85
				10.00	-1.69
				15.00	-2.54
				20.00	-3.38
				25.00	-4.23
				26.94	-4.56
2	0.17	0.333	18.50	0.00	0.00
				5.00	-1.67
				10.00	-3.33
				15.00	-5.00
				20.00	-6.67
				25.00	-8.33
				30.00	-10.00
				36.93	-12.31
3	0.333	0.5	28.24	0	0.00
				8	-4.00
				16	-7.99
				24	-11.99
				32	-15.99
				40	-19.99
				48	-23.98
				56.60	-28.28



Profile of the chute spillway

6.4.2 Stilling Basin

A stilling basin is a channel structure of mild slope, placed at the outlet of a spillway, chute or other high velocity flow channel, whose purpose is to confine all or part of the hydraulic jump or other energy reducing action and dissipate some of the high kinetic energy of the flow. It is a structure which is necessary to prevent bed scour and undermining of the structure in situation where high velocity flow is discharged into the downstream channel.

Usually flow entering a stilling basin is at super critical velocity. The stilling basin on the mild slope supports only sub critical flow. The transition from super critical to sub critical flow takes place in the form of a hydraulic jump. The stilling basin is designed to insure that the jump occurs always at such a location that the flow velocities entering the erodible downstream channel are incapable of causing harmful scour.

The design of a particular stilling basin will depend on the magnitude and other characteristics of the flow to be handled, and particularly the Froud number of the approaching flow. Consider the simplest and most common stilling basin, a horizontal rectangular channel as shown in Fig 5-11,

Hydraulic jump equation:

$$\frac{y_2}{y_1} = \frac{1}{2}\left(\sqrt{8F_1^{\,2}+1}-1\right)$$

Jump head loss equation:

$$\frac{\Delta E_j}{y_1} = \frac{\left(\frac{y_2}{y_1}-1\right)^3}{4y_2/y_1} = \frac{\left(\sqrt{8F_1^{\,2}+1}-3\right)^3}{16\,(\sqrt{8F_1^{\,2}+1}-1}$$

Energy dissipation efficiency:

$$\frac{\Delta E_j}{E_1} = \frac{\Delta E_j/y_1}{E_1/y_1} = \frac{\left(\sqrt{8F_1^{\,2}+1}-3\right)^3}{8(\sqrt{8F_1^{\,2}+1}-1)(2+F_1^{\,2}}$$

Jump height:

$$\frac{y_2 - y_1}{y_1} = \frac{1}{2}\sqrt{8F_1^{\ 2} + 1} - \frac{3}{2}$$

The length of jump is a very important factor in stilling basin design. It can be obtained from the curve by USBR.

The longitudinal position of the jump on the apron must be such that the upstream and downstream depth satisfies the jump equation. For more precise jump location a trial-and-error procedure using the flow profiles is necessary.

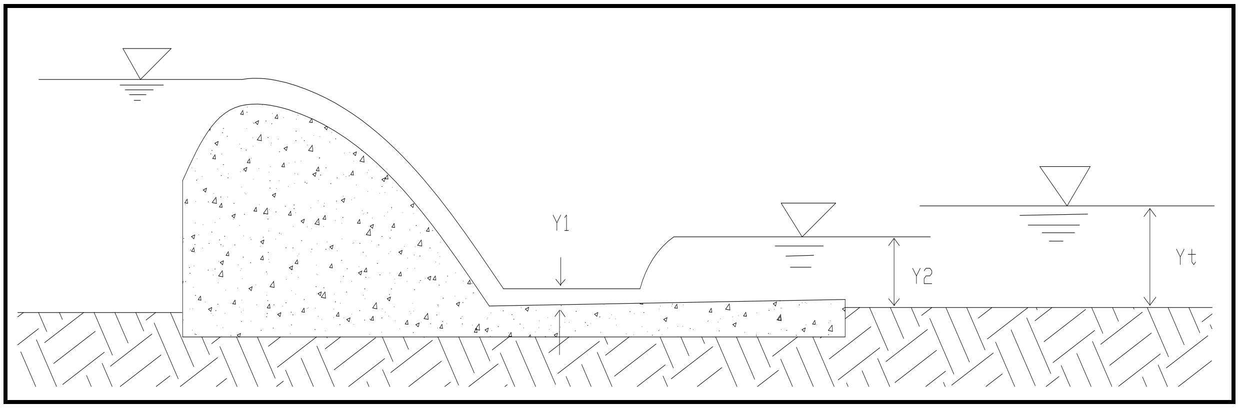


Fig 6-11 Simple Stilling basin

Hence, for a given discharge intensity and given height of spillway, Y_1 is fixed and the thus Y_2 is also fixed. But the availability of a depth equal to Y_2 in the channel on the d/s cannot be guaranteed as it depends upon the tail water level, which depends upon the hydraulic dimensions and slope of the river channel below. For different discharges, the tail water depth is found by actual gauge discharge observations and by hydraulic computation. The post jump depths for all those discharges are also computed from hydraulic jump equation. If a graph is now plotted between q and tail water depth, the curve obtained is known as the Tail water curve. Similarly, if a curve is plotted on the same graph, between q and Y_2, the curve obtained is known as the jump height curve.

If the tail water depth $Y_t = Y_2$, the jump would always form at the toe of the spillway and a short concrete apron would give adequate protection. However, the situation doesn't always exist in reality. The following are usually possible due to the variation of discharge:

Y_2 always lower than Y_t

Y_2 always higher than Y_t

Y_2 partly lower and partly higher than Y_t

Depending upon the relative position of Tail water curve and Y_2 curve, the energy dissipation arrangements can be provided below the spillway. The more common provision of which have been used singly or in combination to modify the jump characteristics in stilling basins are:

Sloping apron

Sill, or small dam, at end of apron

Hydraulic drop

Chute blocks and baffle blocks

Bucket dissipater

Stilling pool

For example, the following energy dissipation mechanism can be provided in cases other than $Y_2 = Y_t$,

When Y_2 is always lower than Y_t, the jump will be submerged and energy dissipation is very low. To obtain a free hydraulic jump and obtain considerable energy dissipation, Y_2 has to be equal to Y_t. This can be achieved by providing a sloping apron above the stream bed level.

In the case of Y_2 higher than Y_t, lowering of the apron below the stream bed level is required.

In the case of Y_2 partly lower and partly higher than Y_t, provide a sloping apron partly above and partly below the stream bed level so that the jump can form where the sequent depth is available.

7 Sediment Transport and Stable Channel Design

7.1 Mechanics of Sediment transport

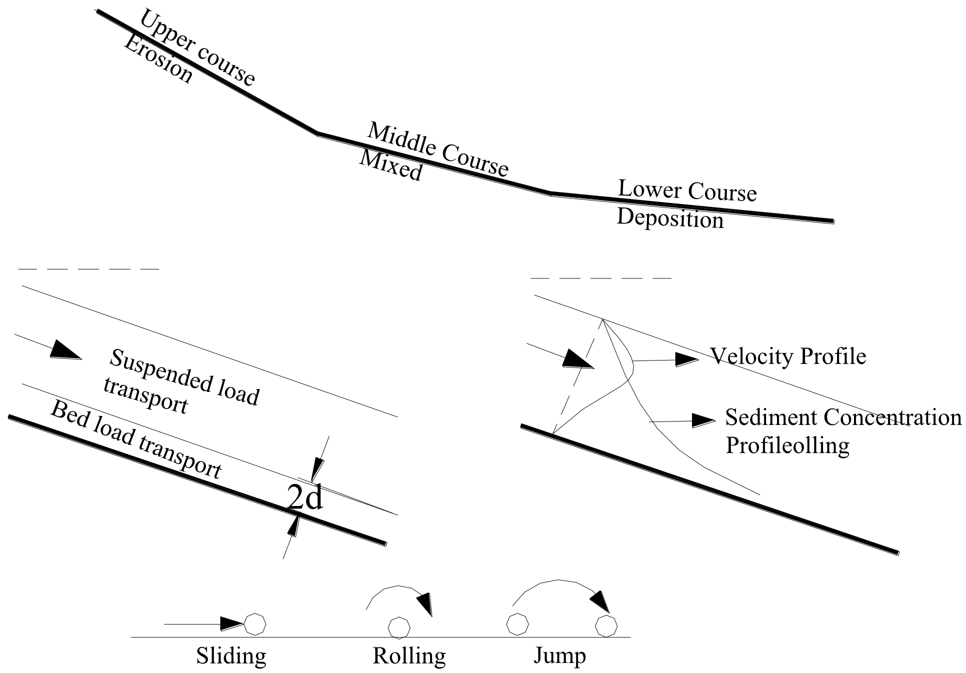


Figure 7-1 River reaches and mode of sediment transport

Channel in enrodible soils carry appreciable silt and sand load. Silt concentration affects the velocity of flow. When the channel water has excess silt load, deposition occurs in the channel. On the contrary when the channel water is silt free erosion of the channels take place.

When deposition takes place, the channel section is reduced; when erosion occurs the crosssection of the channel increases, resulting in decreased water level.

Manning's or chezy's equation do not consider the above effect end they cannot be used in the design of channels in erodible soil.

7.2 Lacey's regime method of stable channel design.

According to Lacy the dimension of a regime channel to cavy a given Q and a given silt lead are fixed by nature.

Regime channel: Channel is said to be in regime when the following conditions are satisfied

i. discharge is constant
ii. the Silt grade and silt load are constant
iii. The channel is flowing in unlimited incoherent alluvium of the same character as that transported.
iv. The channel has freedom to form its own dimension

Lacey's Regime Equation

$$V = \left(\tfrac{2}{5} fR\right)^{\frac{1}{2}} \qquad \text{in metric units}$$

$$Af^{2} = 140\, V^{5}$$

Where f = silt factor

$= 1.76\sqrt{m_r}$

m_r= mean particle diameter of silt (mm)

Lacey's design equations

$$V = \left(\frac{Qf^2}{140}\right)^{\frac{1}{6}} \qquad \text{Velocity discharge relation} \qquad i$$

$Q = AV$ Continuity equation *ii*

P = 4.75√Q Perimeter discharge relation (p-Q relation) *iii*

$$S = \frac{f^{5/3}}{3340\, Q^{\frac{1}{6}}} \qquad \text{Regime slope equation (S-Q-f relation)} \qquad iv$$

Design steps

Given Q and *f* calculate v from [eq. i]

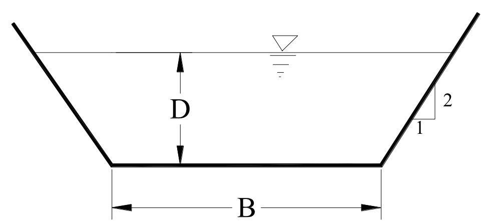
Knowing v determine A from [eq. ii]

Determine P from [eq. iii]

Knowing A and P, determine B and D assuming the side slopes of he channel 1 : 2 (H:V)

$A = BD + 0.5D^2$

$P = BD + D\sqrt{5}$



Solving the above equation simultaneously for B and D

$$D = \frac{[P - \sqrt{P^2 - 6.944\, A}]}{3.472}$$

$B = P - 2.236D$

Check numerical value by calculating R from

$$\left.\begin{array}{l} R = \dfrac{5}{2}\dfrac{V^2}{f} \\ R = \dfrac{A}{P} \end{array}\right\}$$ Both the values of R should be the same. If not check the numerical work (Step 1 - 4)

2. Determine the bed slope of canal by using (eg (iv)

Design example

Using Lacey's equations design an irregular channel for the following data

Q = 20m/s

F = 1.00

Side slope ½ =1

Solution

$$V = \left(\frac{Qf^2}{140}\right)^{\frac{1}{6}} = \left(\frac{20*1}{140}\right)^{\frac{1}{6}} = 0.723\text{m/s}$$

A = Q/V = 20/0.723 = 27.66m^2

P = 4.75√Q = 4.75√20 = 21.24m

$$D = \frac{[P - \sqrt{P^2 - 6.944\, A}]}{3.472} = \frac{[21.24 - \sqrt{21.24^2 - 6.944 * 27.66}]}{3.472} = 1.48$$

$B = P - 2.236D = 21.24 - 2.236*1.48 = 17.43m$

Check on numerical value

$$R = \frac{5}{2}\frac{V^2}{f} = \frac{5}{2}\frac{0.723^2}{1.0} = 1.30\,m$$

$$R = \frac{BD + 0.5D^2}{B + D\sqrt{5}} = \frac{17.93 * 1.48 + 0.5 * 1.48^2}{17.93 + 1.48\sqrt{5}} = 1.30$$

Hence numerical work is OK!

$$S = \frac{f^{5/3}}{3340\,Q^{\frac{1}{6}}} = \frac{1^{5/3}}{3340 * 20^{\frac{1}{6}}} \approx 1/5500$$

Rational Approach stable channel Design

This approach is based on the concept of tractive force average tractive bed shear stress

$$Zo = \gamma Rs$$

The value of tractive shear stress varies along the usetted perimeter p of the channel

For wide channel $\tau_o = Zo = \gamma Ds = const.$

Table 7-1 Values of permissible tractive shear stress corresponds to permissible velocities.

Channel material	Manning	Clear water		Water transprting silt	Colloidal
	n	v	Z_0	v	Z_0
Fine sand , colloidal	0.020	0.46	1.31.3	0.76	3.6
Sandy loam, non colloidal	0.020	0.53	1.8	0.76	3.6
Silt loam	0.020	0.61	2.3	0.91	5.3
Avavial silt	0.020	0.61	2.3	1.07	7.2
Ordinary firm loam	0.020	0.76	3.6	1.07	7.2
Volcanic ash	0.020	0.76	3.6	1.07	7.2
Stiff clay, very colloid al	0.025	1.14	12.5	1.52	22.0
Auuvial silt, colloidal	0.025	1.14	12.5	1.52	22.0
Shales and hard pans	0.025	1.83	32.1	1.83	32.1
Fino gravel	0.03	0.61	3.6	1.52	15.3
Graded loam to cabby, non colloidal	0.03	1.14	18.2	1.52	31.6
Graded silt to cobble, colloidal	0.03	1.23	20.6	1.68	38.3
Coarse gravel non-colloidal	0.025	1.23	14.4	1.83	32.1
Cobbles and shinales	0.025	1.52	43.6	1.68	52.7

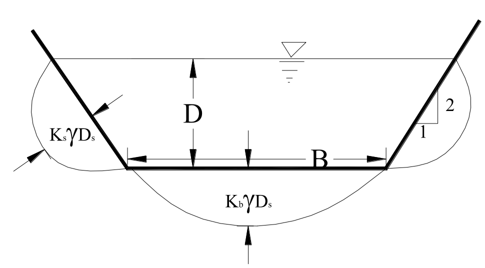
The above value may be computed as follows

$$\tau_o = \gamma RS = \gamma R\left(\frac{nV}{R^{\frac{2}{3}}}\right) = \frac{\gamma}{R^{\frac{1}{6}}}(nV)^2 \approx 9810\left(\frac{nV}{R^{\frac{1}{3}}}\right)$$

If $R = 0.24$m then the values of Z_0 in the table will be found to cause pond approx to the corresponding values of n and v. The tractive force distribution for a trapezoidal channel

$K_s, K_B = f(B/D)$

Ordinary channel sizes, *ks =3/4 and Kb = 1*



7.3 Force on sediment particle

$$R = \sqrt{(W_s \sin\theta)^2 + (\tau_s a_s)^2} = W_s \cos\theta C_f$$

$\sqrt{(W_s \sin\theta)^2 + (\tau_s a_s)^2}$ Acting force

$W_s \cos\theta C_f$ Resisting force

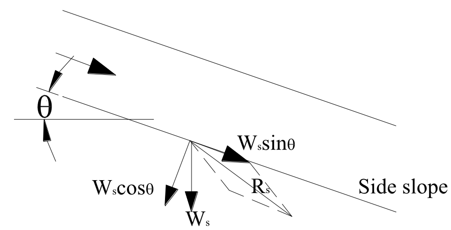
Where C_f = coefficient of frication

Ws = weight of particle

$\tau_s a_s$ = Horizontal tractive force acting on the particle

a_s = effective shear area of particle

Rs = Resultant force on the particle



$$\tau_s = \frac{Ws}{a_s}\cos\theta \tan\phi \sqrt{1 - \frac{\tan^2\theta}{\tan^2\phi}}$$

If $\theta \geq \phi$, $\tau_s = 0$

On the bottom surface $\tau_b = (W_s/a_s) \tan\phi$

The tractive force ration is defined as $K = \tau_s/\tau_b$ $\quad k = \sqrt{1 - \frac{\sin^2\theta}{\sin^2\phi}}$

The required mean depth to prevent erosion will be either

$$D = \frac{\tau_s}{k_s \gamma_s} \quad \text{or} \quad D = \frac{\tau_b}{k_b \gamma_s}$$

Depending on whether the bed shear or side shear controls for a balanced design

$$\frac{\tau_s}{k_s \gamma_s} = \frac{\tau_b}{k_b \gamma_s} \quad \rightarrow \quad \frac{\tau_s}{\tau_b} = \frac{k_s}{k_b} = k$$

Assuming $k_s = 3/4$ and $k_b = 1$

$$k = \frac{k_s}{k_b} = \frac{3}{4} = \sqrt{1 - \frac{\sin^2\theta}{\sin^2\phi}}; \qquad \sin\theta = \sqrt{\frac{7}{10}}\sin\phi$$

When $\sin\theta < \sqrt{7/16}\sin\phi$ bed shear controls and $D = \tau_o/\gamma_s$ otherwise, side shear and $D = \tau_o/\gamma_s(k/3/4)$

Note: Zo is the permissible shear stress as obtained form the previous table.

The above equation applies only to non-cohesive sills.

7.4 Stream Diversion

A Diversion problem exists to some extent to all dam sites except those located off stream, and the selection of the most appropriate scheme for handling the flow of the stream during construction is important to reduce the cost of the dam. The scheme selected ordinarily should represent a compromise between the cost of the diversion facilities and the amount of risk involved.

Factors considered to determine the best diversion scheme:

i. characteristics of stream flow
ii. Size and frequency of diversion flood
iii. Methods of diversion

iv. Specification requirements.

Objective: to select optimum seneme considering practicability cost and the risks involved.

Diversion provisions commonly used singly or in combination:

i. Tunnels driven through the abutments
ii. Conduits through or under the dam
iii. Temporary channels through the dam
iv. Multiple stage diversion over the tops of alternate construction blocks of a concrete dam.

Common problem: Meeting of d/s equipments when the entire flow of the stream is slopped during the closure of the diversion works.

Solution: provide by pumping or through by passes or phons until water is stored in the reservoir to a sufficient level to release by gravity through outlet works.

Tunnels

More feasible for narrow canyons either for a concreter or for an earth fill dam.

Placed in one or both abutments.

If tunnel spillways or tunnel outlet works are provided in the design of the dam. It usually proves economical to use them in the diversion plan.

Temporary auxiliary steam level conduit leading to the intersection of the horizontal portion of the spillway tunnel and the inclined shaft (leading to the spillway gate structure) is provided to by pass the flow.

Shutting of diversion flows; regulation of flow through the diversion tunnel

Example: An arrangement consisting of u/s bulk head with pipes in ht bulk head for regulation of flow can be provided.

Conduits

⇒ Conduit outlet works designed of an earth fill dam may be utilized of diversion.
  - Economical especially if the conduit is adequate to carry the diversion flows.

⇒ Where diversion requirement is greater than the capacity of outlet works, increase the capacity:
  - By delaying the installation of gates valves, pipe and trash racks until the need for diversion is over;
  - By increasing the height of the cofferdam (increasing the head)

⇒ In cases where the intake to outlet works conduit is above the level of the streambed, an auxiliary steam level conduit may be provided to join the lower portion of the permanent conduit. Permanent closure of the auxiliary conduit is accomplished by a concrete plug.

⇒ Diversion conduits at stream level are sometimes provided through a concrete dam. After the requirement for diversion is over, the conduits must be permanently closed throughout their entire length by concrete plugs.

Temporary diversion Channel-Earth fills Dams

⇒ A Temporary channel through the earth fill dam;

⇒ Side slope should be flatter than 4 to 1:
  - To facilitate filling of the gap at the end of the construction period;
  - To decrease the danger of cracking of the embankment due to differential settlements.
  - To secure a good bonding surface b/n the previously constructed embankment and the material to be placed.

⇒ Provided when a division tunnel or conduit is net economical;

⇒ Adaptable to wide sites

- In wider valleys the diversion flows are likely to be too large to he economically period in tunnels and conduits.

⇒ Foundation preparation required for the dam should be completed in the area where temporary opening will be left

⇒ While diversion being carried through the opening foundation and embankment work in the remainder of the streambed is completed.

⇒ The bed lope of temporary channel should be the same as the original streambed to minimized erosion in the channel.

⇒ Closure

- A period when large floods are least likely to occur is selected so that the risk of the rising water surface in the reservoir overtopping the embankment being placed in the closure section does not occur.
- The average rate of embankment placement must be such that the gap can be filled faster than the water rise in the reservoir.

Multiple stage diversion for concrete dams

1st Stage: The flow is restricted to one portion of the stream channel while the dam is constructed to a safe elevation in the remainder of the channel.

2nd stage: The cofferdam is shifted and the stream is carried over low blocks or through division conduits in the constructed section of the dam while work proceeds on the portion not constructed.

Last Stage:- construction is carried out to the ultimate height while diversion is made through the spillway, penstock or permanent outlet.

(ii) Rack losses

$$h_{lr} = k_r (t/b)^{4/3} \frac{V_B^2}{2g} \sin \phi \qquad \text{Kirschmer's Formula}$$

Where kr =f(x-section of bars)

Flow direction -				
Shape				
Kr	2.42	1.83	1.67	0.76

$$h_{lr} = k_r \frac{V^2}{2g}$$

Where k_r = loss coefficient;

= 1.45 -0.45R-R2

V = Velocity through contracted opening

R = Ratio of net area through trash rack bars to gross area of the have and supports.

(iii) Head gate loss: Loss occurs at part gate opening where the gate contracts the flow.

$$Q = C_d A\sqrt{2gh}$$

Where Q= flow in the conduit

A= area of gate opening;

h= difference in head acting on the gate;

C_d = coefficient of discharge;

= $1/\{k+1\}^{1/4}$

k = loss coefficient applied to the velocity at the gate section;

= 1.6 -0.45

The water works experimental station (USA) suggests the following:

C_d =0.73 – 0.80; k=0.87 – 0.56

In addition to the above three major losses and the friction loss there are other conduit losses (losses due to expansion, construction, bends, bifurcation, junctions, valve passages and exit). The expressions for the above may be obtained form any standard text.

References:

1. M.M. Grishin, *Hydraulic Structures*, vol. 1, Mir Publishers, 1987, Moscow.
2. P.N. Moodi, *Irrigation Water Resources and Water Power Engineering*, Standard Book House, 1995, New Delhi.
3. S.K.Garg, *Irrigation Engineering and Hydraulic Structures,*
4. *Design of Small Dams. U.S. Bureau of Reclamation. 4th Ed.*
5. Prof. Bollrich ,G *Manual on Functional Hydraulic Structures for Dams,.*

Sample Questions from Previous Exams

KISAMA AFRICA UNIVERSITY COLLEGE
DEPARTMENT OF CIVIL ENGINEEREING

Mid-Semester Exam **Date Dec. 22, 04**
Hydraulic Structures I (CE : **Time allowed 2 ½ hrs.**

Instruction

2. **Each question has equal marks**
3. **The examination is closed book.**

1. Reply for the letter below as a responsible Civil Engineer based on the request made.

Date December 20, 2004

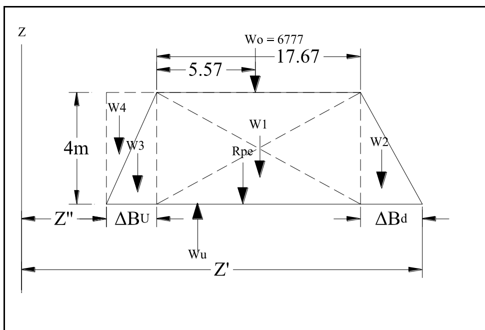
[Address]
Subject: Request for Volunteer Technical Assistance
Dear sir/Madam:
Our community has decided to dedicate all the available resource to improve the current life standard. We are committed to construct a dam across a nearby river that has been flowing for centuries untouched. There is no expert or engineer among ourselves and hence we have sent you a letter requesting your volunteer technical assistance in the reconnaissance and feasibility study.
This is, therefore, to kindly request your assistance on identifying the major concerns and information needed to determine or evaluate

- **i. Necessity of the dam,**
- **ii. Selected dam site,**
- **iii. Reservoir Capacity and**
- **iv. The type of dam.**

Thank you for your invaluable assistance.
Sincerely,
[Signature]
[Name.]

2. i. Discuss at least four advantages and disadvantages of constructing a large dam.
 ii. List three dams built and operational in Ethiopia and write the primary purpose of the reservoir.

4. The following is a section of a dam being designed using the multi-step method of analysis. The section is in zone IV and is the second block 27m below the water surface. Perform the necessary calculation and decide the geometry of the section based on the design standards and criteria of the zone.



Item	Value	Item	Value
H_{max}	60	f	0.75
h_e	3	s_a	560
Tail water	None	s_{sf}	5
γc	24	ζ	0.5
γw	10	C	1
top width	7.5	σ_c'	30
Eq & silt	Ignore	F	6.4
Wave force is 31.25 and is 31.47m from the bottom of section			

Inst.
Shimelis B.
(Ato.)

Good Luck!

Partial Solution

1. Refer to your Hand out on the specific Topics.

i. letter writing **(2 pts)**

ii. Necessity of dam **(2 pts)**

iii. Selected dam site **(3 Pts)**

iv. reservoir capacity **(2 Pts)**

v. type of dam **(1 Pt)**

2. Advantages and Disadvantages of Large dam

Advantages **(4 Pts)**

1. Better utilization of the water resource
2. Improved life standard
3. New Ecosystem
4. Recreational center and Fishery and many more.

Disadvantages **(3 Pts)**

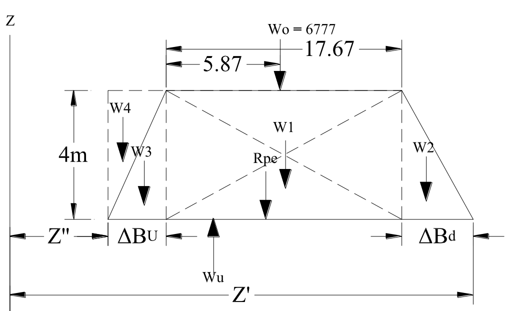
1. Loss of Farm land
2. New pattern of Disease
3. Disturb Natural Ecology
4. Extinction of Plant and Animal Species
5. Resettlement (Social Crisis) and many more.

Major operational dams in Ethiopia **(3 Pts)**

1. Koka Dam for Hydroelectric power generation, Irrigation and Flood control.
2. Gilgel Gibe for Hydroelectric Power Generation.
3. Melkawakena for Hydroelectric Power Generation.

3. Zone IV Block II

ho = 27m Δh = 4m h = 31m Bo = 17.67 m



Line	Item	Description & dimension	Forces: Horizontal	Forces: Vertical	Lever	Moment
1	W0	concreter above h = 27m		6777	8.87	60111.99
2	W1	4*17.67*24		1696.32	11.83	20075.95
	Trial I	ΔB_d = 3.1 **(1 Pt)**				
3	W2	3.1* *24		148.8	21.70	3229.46
4		Total Partial empty **(1 Pt)**		8622.12	9.67	83417.4
		Estimation **(2 Pts)**	Z' =	23.77		
		2B/3=Z' - 8.61 = 23.77 - 8.61=	14.1	B=	21.15	
		DBu =B - (Bo + DBd) =	0.38	Z'' =	2.62	

		Z"+B/3 =	9.67	Z"+2B/3 =	16.72	
5	W3	0.5*0.38*4*24		18.24	2.87	52.35
6		Reservoir Empty		8640.36	9.66	83469.75
7	W4	Water column 0.38*25*10		110.2	2.81	309.662
7	Wu	Uplift: 0.5*B*0.5h*10		-1639.13	9.67	-15850.3
8	Fh	Water Pressure	4805		10.33	49651.67
9	Fwa	Wave action	31.25		31.47	983.44
		Reservoir Full	4836.25	7111.435	16.67	118564.2

The resultant for both reservoir empty and reservoir full case passes with in the middle third of the base. Furthermore, from the line of action of the resultant it can easily be deduced that the section is economical. **(1 Pt)**

Check for Shear Resistance **(1 Pt)**

$$\tan\theta = \frac{\Sigma H}{\Sigma W} = \frac{4836}{7111} = 0.68 < 0.75 = f \qquad \text{Safe!}$$

Check for Stresses

Reservoir Full

$$P_v^{'}, P_v^{''} = \frac{\Sigma W}{B}\left(1 \pm \frac{6e}{B}\right)$$

(1 pt)

$$= \frac{7111}{21.15}\left(1 \pm \frac{6 * 3.48}{21.15}\right) = 668.1, 4.4$$

$$P_i^{'}, P_i^{''} = P_v^{'}.P_v^{''}(1 + \tan^2\phi^{'}, \tan^2\phi'')$$

$$\tan\phi' = \frac{3.1}{4} = 0.775 \qquad \tan\phi'' = \frac{0.38}{4} = 0.095$$

(1 Pt)

$$P_i^{'} = 668.1(1 + 0.775^2) = 1072\ KPa < \sigma_{all} = 5000\ KPa \qquad \text{Safe!}$$

Reservoir empty **(2 pt)**

$$P_v^{'}, P_v^{''} = \frac{\Sigma W}{B}\left(1 \mp \frac{6e}{B}\right)$$

$$= \frac{8640.36}{21.15}\left(1 \mp \frac{6 * 3.53}{21.15}\right) = 20.4, 796.6$$

$$P_i^{''} = P_v^{''}(1 + \tan^2\phi'')$$

$$P_i^{''} = 796.6(1 + 0.095^2) = 803.8\,KPa < \sigma_{all} = 5000\ KPa \qquad \text{Safe!}$$

KISAMA AFRICA UNIVERSITY COLLEGE
DEPARTMENT OF CIVIL ENGINEEREING

Mid-Semester Exam **Date Dec. 22, 04**
Hydraulic Structure I: **Time allowed 2 ½ hrs.**
The examination is Open book.

Part I Multiple Choice

Among the four alternative answers choose the best answer that fits to each question. Each question has 2 marks.

1. Earthen dams are:
   a. rigid dams c. overflow dams
   b. non rigid dams d. diversion dams

2. when sand and gravel foundation strata is available at a proposed dam site of moderate height, the dam may be of the type:
   a. Earthen dam or rock fills dam c. double arch dam
   b. masonry gravity dam d. concrete gravity dam
3. Yield of a reservoir represents:
   a. The inflow into the reservoir c. the outflow demand on the reservoir
   b. The capacity of the reservoir d. none of the above

4. The capacity of a storage reservoir can be decided by using:
   a. The mass curve of inflow c. by both (a) and (b)
   b. The mass curve of outflow d. none of the above.

5. According to thin cylinder theory the volume of concrete required for an arch dam would be minimum if the central angle is:
   a. $150^0 34'$ c. $136^0 34'$
   b. $133^0 34'$ d. $140^0 34'$

6. The slope of the upstream face of a flat-slab type buttress dam
   a. Is always less than 35^0 c. usually varies from 35^0 to 45^0
   b. Is always more than 45^0 d. none of the above
7. When seepage takes place through the body of an earthen dam, it leads to:
   a. development of pore pressures in the dam body c. reduction in the shear stresses in the dam
   b. reduction in the shear strength of the dam d. both (a) and (b)

8. When the water level standing against an earthen embankment, suddenly falls down, then there is an imminent risk of sliding failure, to the:
   a. upstream slope c. both (a) and (b)
   b. downstream slope d. none of the above

9. During the maintenance of an earthen dam, the apparent seepage through the foundation of the dam is best taken care of , by providing:
   a. A chimney drain c. a drain trench along the d/s toe
   b. A rock toe d. an u/s impervious cutoff

10. Which one of the following spillways is least suited to earthen dams?
   a. Ogee spillway c. chute spillway
   b. Side channel spillway d. shaft spillway

Part I Multiple choice Best answers

Question no.	1	2	3	4	5	6	7	8	9	10
Best choice	B	A	C	C	B	C	D	A	C	A

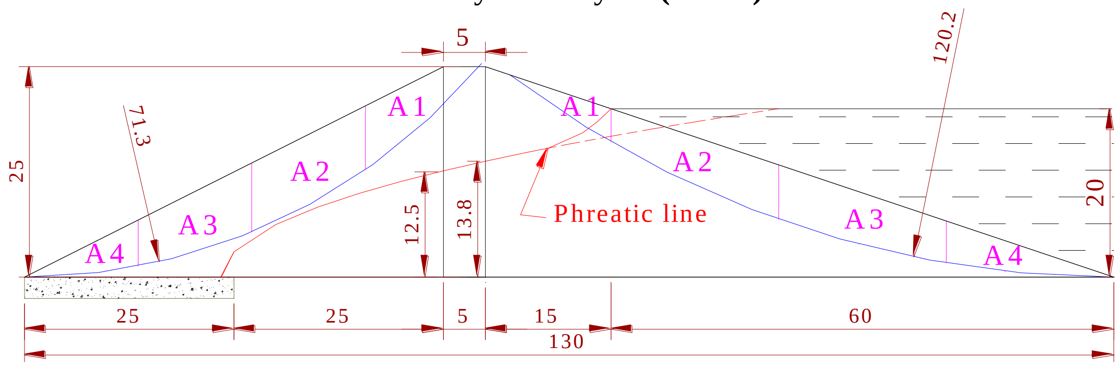
Part II Design Problem

1.Design the embankment dam shown below. Show all the necessary procedures and important consideration in the process.

a. Draw the seepage line inside the dam when the reservoir is full. **(10 Pts)**

b. Check the overall stability and the safety of the dam against sliding. **(10 Pts)**

c. Analyze the stability of the slopes using Swedish Slip Circle method. Show how you obtain the locus of center of failure surfaces and consider toe failure case and the slices shown below your analysis. **(10 Pts)**



Geometric properties of slices.

	Upstream				Downstream		
Slice	Area(m^2)	α (deg)	h_w (m)	L (m)	Area (m^2)	α (deg)	L (m)
A1	25.97	32	0	14.5	68.58	40	18
A2	111.68	24	6.1	22	111.48	28	15.4
A3	120.83	15	6.2	20.7	96.1	17	14.2
A4	55.52	4	3.6	20.1	40.11	7	13.7

Area in relation with phreatic line.

Area	Dam	U/s shoulder	D/s shoulder

under seepage line	1102.08	838.1	221
Dry portion	584.92	99.4	404
Total	1687	937.5	625

Part II Design Solution

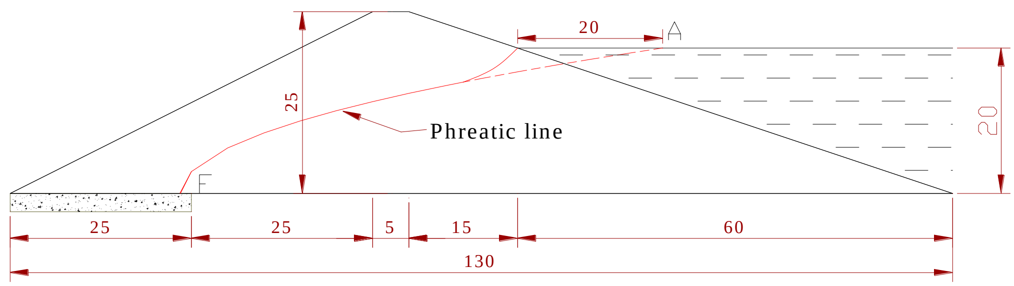
Determination of phreatic line

For the origin of the Cartesian co-ordinate system at the face of the filter (point F), the equation of the parabola of the seepage line can be expressed as:

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

At point A, x = 65m, and y = 20m. Inserting into the parabola equation, S = 3.07m.
Working out a few more points from the equation, the parabola can be easily drawn and corrected for the curve at the upstream face of the dam, so as to get the seepage line.

x	-1.51	0	10	15	25	30	40	45	55	65
y^2	0	9.06	69.26	99.36	159.56	189.66	249.86	279.96	340.16	400.36
y	0	3.01	8.32	9.97	12.63	13.77	15.81	16.73	18.44	20.01



To assess the overall stability of the dam considering 1m length,

	Dam		U/s shoulder		D/s shoulder	
Item	Area	Weight	Area	Weight	Area	Weight
Under seepage line	1102.08	13224.96	838.1	10057.2	221	2652
Dry portion	584.92	10528.56	99.4	1789.2	404	7272
Total	1687	23753.52	937.5	11846.4	625	9924

Shear resistance of the dam at the base(R) = C + W tanΦ

Where: C = total cohesive resistance of the soil at the base
= c*B*1 = 24 * 130 * 1
= 3120 kN

$W\tan\phi = 23753 * \tan 25^0$
= 11076 kN

R = 3120 + 11076
= 14196 kN

Horizontal force due to hydrostatic pressure of water

$P = ½\, \gamma_w\, h^2 = ½ * 10 * 20^2$
= 2000kN

Factor of safety against failure due to horizontal shear at the base = R/P = 7.1 > 1.5 Safe!

Check stresses in the foundation

t = 8m < L/10 = 130/10 =1.3,

Hence,

$\tau_{max} = \gamma_f H_s t/L$
$= 18.3 * 10 * 8/130$
$= 11.26 kN/m^2$

Shear strength = Ss = c + σ tanΦ
$= c + W/L \tan\phi$
$= 54 + 11076/130 * \tan 12^0$
$= 72 kN/m^2$

The factor of safety against overstress is

$FS = Ss/\tau_{max} = 72/11.26 = 6.4 > 1.5$ Safe!

Stability of u/s and d/s slopes against sliding for average shear[§].

Upstream slope (under sudden drawdown):

Considering unit length of the dam, the horizontal force P_u is

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

$$= \frac{12 * 13.8 + 18(25 - 13.8)}{25}$$

$$= 14.7$$

$$P_u = \left[\frac{\gamma_1 h^2}{2}\tan^2\left(45 - \frac{\phi}{2}\right) + \gamma_w \frac{h_1^2}{2}\right] = \left[\frac{14.7 * 25^2}{2}\tan^2\left(45 - \frac{25}{2}\right) + 10\frac{13.8^2}{2}\right] = 2816.6$$

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

$$R_u = cB_u + W\tan\phi = 54 * 75 + 11846.4 * 0.47 = 9574.1$$

Where; W is the weight of the u/s triangular shoulder of the dam

The factor of safety against can be easily calculated, using

$$FS = \frac{R_u}{P_u} = \frac{9574.1}{2816.6} = 3.4 > 1.5$$ Safe!

For the downstream shoulder:

$$\gamma_1 = \frac{12 * 12.5 + 18(25 - 12.5)}{25} = 15$$

$$P_u = \left[\frac{15 * 25^2}{2}\tan^2\left(45 - \frac{25}{2}\right) + 10\frac{12.5^2}{2}\right] = 2683.7$$

$$R_u = cB_u + W\tan\phi = 54 * 60 + 9924 * 0.47 = 7904.3$$

$$FS = \frac{R_u}{P_u} = \frac{7904.3}{2683.7} = 2.9 > 1.5$$ Safe!

[§] One can check the stresses at selected sections where the shear can is locally maximum, in addition to the average shear consideration done for this particular case.

Analysis of upstream and downstream slopes by Swedish Circle method:

Slice	Area	α	W	T	N	U	L	UL	N'=N-ul	tanΦ	N'tanΦ	Cl	N'tanΦ+cl
A1	25.97	32	467.46	247.72	396.43	0	14.5	0	396.43	0.47	186.32	348	534.32
A2	111.68	24	1340.16	545.09	1224.3	61	22	1342	-117.7	0.47	-55.32	528	472.68
A3	120.83	15	1449.96	375.28	1400.55	62	20.7	1283.4	117.15	0.47	55.06	496.8	551.861
A4	55.52	4	666.24	46.47	664.62	36	20.1	723.6	-58.98	0.47	-27.72	482.4	454.68
			Σ T	1214.56							Σ (N'tanΦ+cl)		2013.54

$$FS = \frac{\sum c_i L_i + \sum N' \tan \phi}{\sum T'} = \frac{2013.54}{1214.56} = 1.66 > 1.5$$

Safe!

Slice	Area	α	W	T	N	l	N'=N-ul	tanΦ	N'tanΦ	Cl	N'tanΦ+cl
A1	68.58	40	1234.44	793.48	945.64	18	945.64	0.47	444.45	432	876.45
A2	111.48	28	2006.64	942.06	1771.76	15.4	1771.76	0.47	832.73	369.6	1202.33
A3	96.1	17	1729.8	505.74	1654.22	14.2	1654.22	0.47	777.48	340.8	1118.28
A4	40.11	7	721.98	87.99	716.6	13.7	716.6	0.47	336.8	328.8	665.6
				2329.27				Σ (N'tanΦ+cl)			3862.66

$$FS = \frac{3862.66}{2329.27} = 1.66 > 1.5$$

Safe!

DANDI BORU UNIVERSITY COLLEGE
DEPARTMENT OF CIVIL ENGINEEREING

Mid-Semester Exam **Date 14 April 2005**
Hydraulic Structure **Time allowed 2 hrs.**

Instruction

5. **Each question has equal marks**
6. **The examination is closed book.**

1. Explain with illustrative sketch how you determine the capacity of reservoir to avoid drought.
2. How will you find the dead and live storage of a reservoir?
3. Discuss at least four advantages and disadvantages of constructing a large dam.
4. List three dams built and operational in Ethiopia and write the primary purpose of the reservoir.
5. List down and discuss briefly the various ways of controlling sedimentation of reservoirs
6. How are dams classified? Discuss in detail.
7. What factors affect the selection of site for a dam? Discuss them briefly.
8. Compare and contrast the advantages and disadvantages of earth foundation and rock foundations.
9. Briefly discuss the possible challenges for both earth and rock foundation
10. Describe various ways of treating coarse grained earth foundations against seepage flow and danger of piping.

DANDI BORU UNIVERSITY COLLEGE
DEPARTMENT OF CIVIL ENGINEEREING

Final Exam **Date 23 June 2005**
Hydraulic Structure **Time allowed 2 hrs.**

The examination is Open book.

Part I Multiple Choice

Among the four alternative answers choose the best answer that fits to each question. Each question has 2 marks.

11. Earthen dams are:
 a. rigid dams c. overflow dams
 b. non rigid dams d. diversion dams

12. when sand and gravel foundation strata is available at a proposed dam site of moderate height, the dam may be of the type:
 a. Earthen dam or rock fills dam c. double arch dam
 b. masonry gravity dam d. concrete gravity dam

13. Yield of a reservoir represents:
 a. The inflow into the reservoir c. the outflow demand on the reservoir
 b. The capacity of the reservoir d. none of the above

14. The capacity of a storage reservoir can be decided by using:
 a. The mass curve of inflow c. by both (a) and (b)
 b. The mass curve of outflow d. none of the above.

15. When seepage takes place through the body of an earthen dam, it leads to:
 a. development of pore pressures in the dam body c. reduction in the shear stresses in the dam
 b. reduction in the shear strength of the dam d. both (a) and (b)

16. When the water level standing against an earthen embankment, suddenly falls down, then there is an imminent risk of sliding failure, to the:
 a. upstream slope c. both (a) and (b)
 b. downstream slope d. none of the above

17. During the maintenance of an earthen dam, the apparent seepage through the foundation of the dam is best taken care of , by providing:
 a. A chimney drain c. a drain trench along the d/s toe
 b. A rock toe d. an u/s impervious cutoff

18. Which one of the following spillways is least suited to earthen dams?
 a. Ogee spillway c. chute spillway
 b. Side channel spillway d. shaft spillway

19. The spillway, which can be called as an "overflow spillway", is essentially
 a. Ogee spillway c. chute spillway
 b. Side channel spillway d. shaft spillway

20. The gated regulator, which is constructed in the parent canal near the site of an offtaking canal, is
 a. Canal head regulator c. cross regulator
 b. Distributory head regulator d. none of the above

Part II

1. a. What are spillways and what is their necessity? **(5 Pts)**
   b. Enumerate the different types of spillways which are used in dam construction. **(5 Pts)**
   c. Discuss briefly the design principles involved in the design of ogee spillway. **(5 Pts)**

2. a. Draw a neat sketch of a typical earth dam and and describe the basic elements. **(5 Pts)**
   b. What are the causes of failures of earth dam? **(5 Pts)**
   c. Explain briefly how the stability of earthen slopes is checked by slip circle method. **(5 Pts)**

3. a. What are the functions of head regulator? **(5 Pts)**
   b. How does “scouring sluice” differ from those of “Head sluices”? **(5 Pts)**

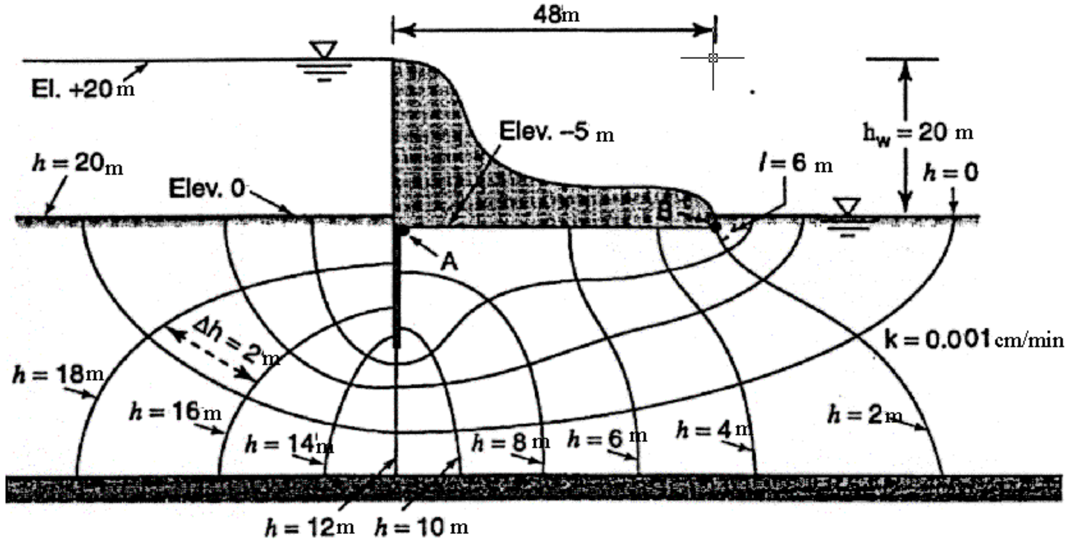
ADDIS ABABA UNIVERSITY
FACULTY OF TECHNOLOGY
DEPARTMENT OF CIVIL ENGINEEREING

Ac. Year 2004/2005
Mid-Semester Exam **Date June 4, 05**
Hydraulic Structures I (CE 418) **Time allowed 2 ½ hrs.**
Instruction: 1. Each question has equal marks
2. The examination is closed book.

7. Describe how you would do a policy analysis to decide on whether or not to remove a dam already constructed on a large reservoir. What kinds of factors would you need to include in your analysis? What are some of the potential harms and benefits of dam removal? What kinds of scientific data should we collect? (7.5 pts)

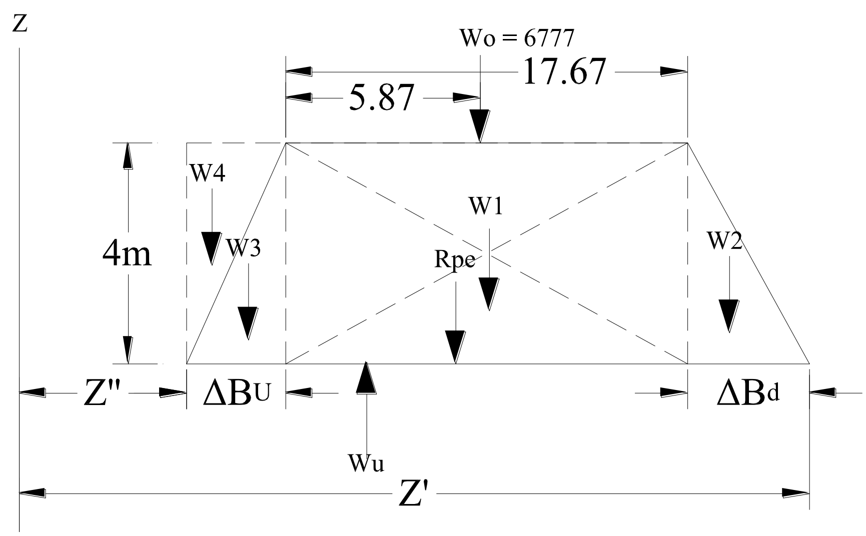
8. Describe exhaustively the possible reasons for provision of free board of dams.(7.5 pts)

9. For the masonry dam with sheetpiling cutoff shown below: (7.5 Pts)
   a. Compute the seepage in m^3/min.; and
   b. Calculate the uplift force acting on the base of the dam.



10. The following is a section of a dam being designed using the multi-step method of analysis. The section is in zone IV and is the second block 27m below the water surface. Perform the necessary calculation and decide the geometry of the section based on the design standards and criteria of the zone. (7.5 Pts)

Item	Value	Item	Value
H_{max}	60 m	f	0.75
h_e	3 m	s_a	5.6MPa
Tail water	None	s_{sf}	5
γc	24 kN/m³	ζ	0.5
γw	10KN/m³	C	1
top width	7.5 m	σ_c'	30 MPa
Eq & silt	Ignore	F	6.4 Km
Wave force is 31.25 and is 31.47m from the bottom of section			



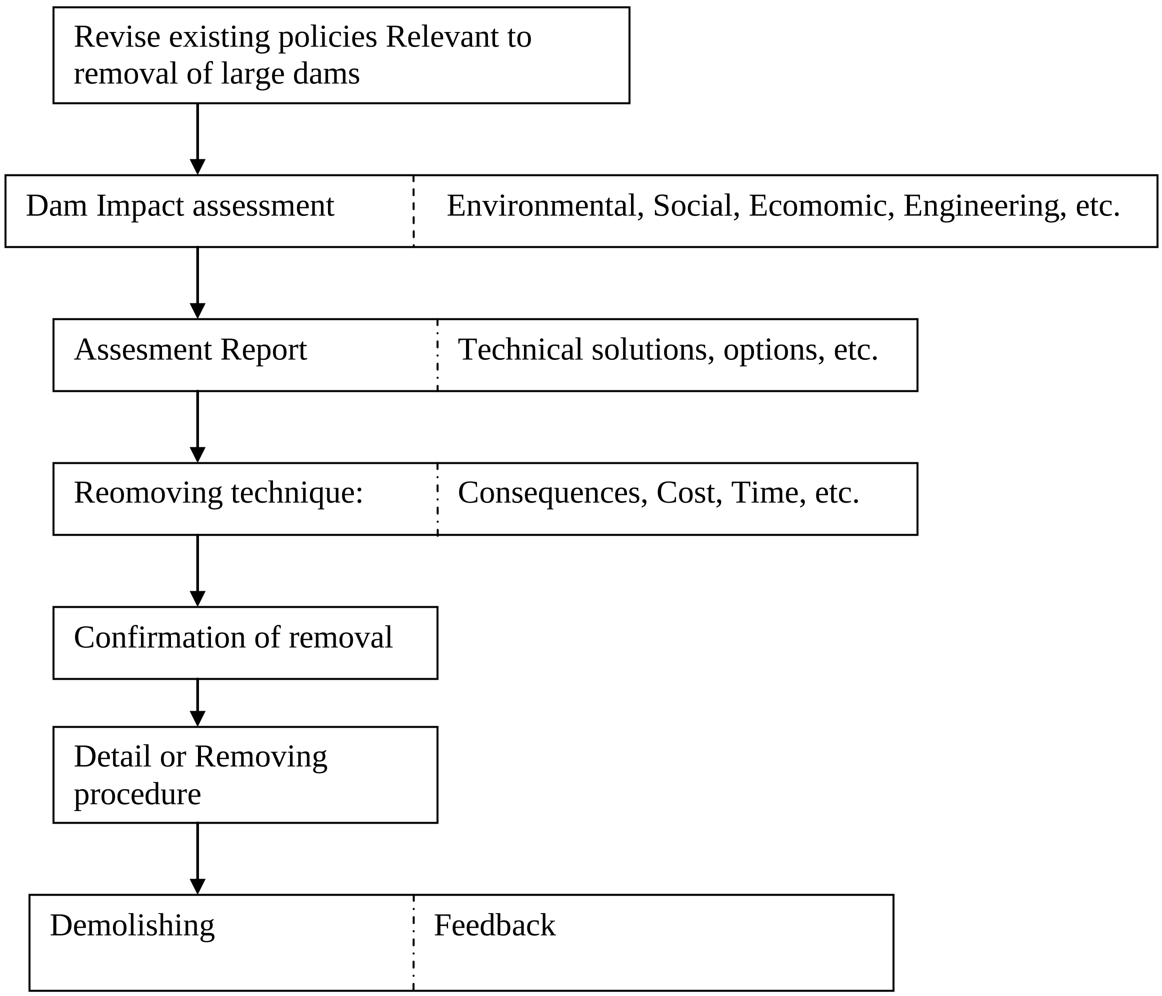
Inst. Shimelis B. (Ato.)
Good Luck!

Partial Solution for Hydraulic Structure I Mid-Semester Exam.

1.

(2 Pts) In order to construct a new dam or remove an existing one, the analysis of policies related to water resources development should first be done. Such policies intermingle different disciplines. Hence a good understanding of every aspect is the corner stone for the final decision.

For removal of an existing large dam, there has to be a sound reason as it leads to a lose of huge sum and social crises. The removal of the dam need to produce a tangible benefit either in social perspective, or economic terms. One can use the following flow chart can be used as a starting guideline to relate the removal with the existing policies at every stage of the assessment.



Here are some of the major factors to be considered in the policy analysis **(2 Pts)**

1. Does the dam negatively affect the welfare of the society?
2. Does the dam created a political instability or conflict between beneficiaries?
3. Is there a new technology that could produce better economic advantage than the existing one?
4. Is the dam causing environmental problems that outweigh its benefit?
5. Will the removal affect the ecological balance of the area?
6. Is there any discovery of precious mineral with in the reservoire site?

7. Is the dam susceptible to failure due to structural defects or geologic reasons that are not observed during design and construction?

Among the potential harms due to removal of large dams **(1 Pt)**
- Flooding of downstream areas
- Lose of money on investments related to the dam such as irrigation schemes, hydropower, etc.
- lose of job for the workers who make their living on activites related to the dam
- ecological disturbance
- Cost of demolishing

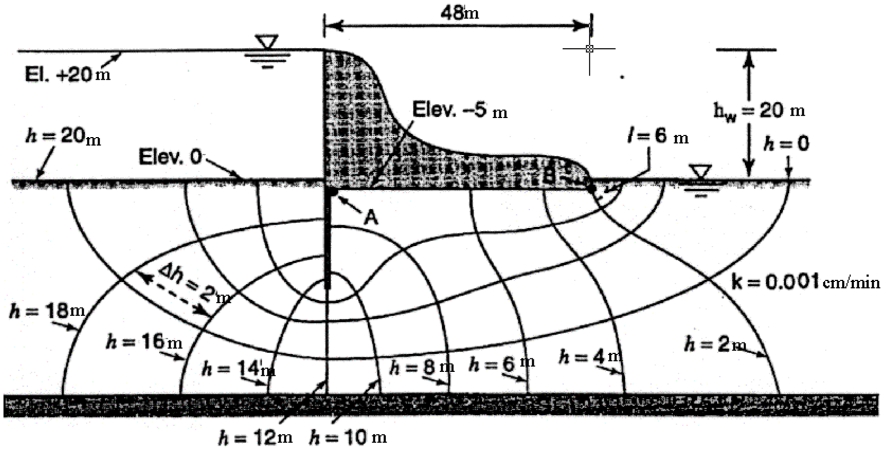
Among the benefits from removal of large dam **(1 Pt)**
- Restoration of the natural water course
- More land if the reservoire is suitable for agriculture or urbanization
- Avoidance of the risk that lead to the removal of the dam, (eg. Failure, disease, political or social reasons, etc.)
- Better utilization using state of the art technology could be possible
- Lesson for future water resource projects

For the analysis various scientific data should be collected **(1.5 Pts)**
Socio-ecomoic data
- Market value of products from the dam (amount of power, irrigation output, etc.)
- The impact of the dam on the neigboring areas (# of deaths, extent of different dangers imposed due to the presence of the dam)
- Need of resettlement if there are areas prone for flooding

Technical data
- The total cost of dam construction, dimensions, unique features and the size of the dam
- Hydrologic data
- Capacity of the valley
- Maxium discharge to be released
- Appropriate time for demolishing
- Cost of demolishing
- Manpower for demolishing etc.

2. The provision of free board for dams is necessary for the following reasons **(7.5 Pts)**
1. To provide space for wave rise of water in the reservoire
2. To account for settlement of either the foundation or dam or both during the life of the dam
3. To reduce progressive deterioration of the dam crest due to impact with floating objects
4. To provide dry and accessible crest of the dam that will not be affected by overflow of water
5. To put confidence on the layman that excess flood could be taken care of in the reservoir (psycology)
6. Facilitate spillage of water by damping momentary peak flood
7. To install water level reading staff gauge

3. Seepage flow



To compute the seepage

Alternative I.

Δh = 2m

k = 0.00001m/min

L[structure] = 48m

I = Δh/l = 2/6 = 0.33 **(1 Pt)**

q = KIA [Darcy's Equation] **(1.5 Pt)**

= 0.00001 * 0.33 * 24

= 8.0 x 10^{-5} m^3/min Per meter length of structure **(1 Pt)**

Alternative II

H = 20m is the head differential. **(0.5 Pt)**

N_f = 4 is number of stream lines. **(0.5 Pt)**

N_d = 10 is number of Equipotential lines. **(0.5 Pt)**

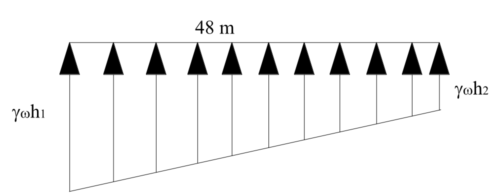
$$q = kH \frac{N_f}{N_d}$$ **(1.0 Pt)**

$$q = kH \frac{N_f}{N_d} = 0.00001\ m/\min * \ 20 * \frac{4}{10} = 8.0 * 10^{-5} m^3/\min$$ **(1.0 Pt)**

Uplift force on the dam

At point A h1 ≈ 9m **(0.5 Pt)**

At point B h2 = 2m **(0.5 Pt)**

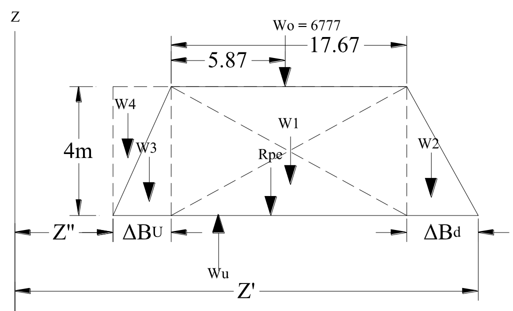


Uplift Pressure $P_u = \gamma_w \left[\frac{h1 + h2}{2}\right] = 10\left[\frac{9+2}{2}\right] = 55\ KPa$ **(2 Pts)**

Uplift force FU = Pu x A = 55 * 48 = 2640 kN per meter length of the structure. **(1 Pt)**

4. Zone IV Block II

ho = 27m   Δh = 4m   h = 31m   Bo = 17.67 m



Line	Item	Description & dimension	Forces Horizontal	Forces Vertical	Lever	Moment
1	W0	concreter above h = 27m		6777	8.87	60111.99
2	W1	4*17.67*24		1696.32	11.83	20075.95
	Trial I	ΔB_d = 3.1 **(0.5 Pt)**				
3	W2	3.1* *24		148.8	21.70	3229.46
4		Total Partial empty **(0.5 Pt)**		8622.12	9.67	83417.4
		Estimation **(1.5 Pts)**	Z' =	23.77		
		2B/3=Z' - 8.61 = 23.77 - 8.61=	14.1	B=	21.15	
		DBu =B - (Bo + DBd) =	0.38	Z" =	2.62	
		Z"+B/3 =	9.67	Z"+2B/3 =	16.72	
5	W3	0.5*0.38*4*24		18.24	2.87	52.35
6		Reservoir Empty		8640.36	9.66	83469.75
7	W4	Water column 0.38*25*10		110.2	2.81	309.662
7	Wu	Uplift: 0.5*B*0.5h*10		-1639.13	9.67	-15850.3
8	Fh	Water Pressure	4805		10.33	49651.67
9	Fwa	Wave action	31.25		31.47	983.44
		Reservoir Full	4836.25	7111.435	16.67	118564.2

The resultant for both reservoir empty and reservoir full case passes with in the middle third of the base. Furthermore, from the line of action of the resultant it can easily be deduced that the section is economical. **(1 Pt)**

Check for Shear Resistance **(1 Pt)**

$$\tan\theta = \frac{\Sigma H}{\Sigma W} = \frac{4836}{7111} = 0.68 < 0.75 = f$$ Safe!

Check for Stresses

Reservoir Full

$$P_v^{'}, P_v^{''} = \frac{\Sigma W}{B}\left(1 \pm \frac{6e}{B}\right)$$

(1 pt)

$$= \frac{7111}{21.15}\left(1 \pm \frac{6 * 3.48}{21.15}\right) = 668.1, 4.4$$

$$P_i^{'}, P_i^{''} = P_v^{'}.P_v^{''}(1 + \tan^2\phi^{'}, \tan^2\phi'')$$

$$\tan\phi' = \frac{3.1}{4} = 0.775 \qquad \tan\phi'' = \frac{0.38}{4} = 0.095$$ **(1 Pt)**

$$P_i^{'} = 668.1(1 + 0.775^2) = 1072\ KPa < \sigma_{\text{all}} = 5000\ KPa$$ Safe!

Reservoir empty **(1 pt)**

$$P_v^{'}, P_v^{''} = \frac{\Sigma W}{B}\left(1 \mp \frac{6e}{B}\right)$$

$$= \frac{8640.36}{21.15}\left(1 \mp \frac{6 * 3.53}{21.15}\right) = 20.4, 796.6$$

$$P_i^{''} = P_v^{''}(1 + \tan^2\phi'')$$

$$P_i^{''} = 796.6(1 + 0.095^2) = 803.8KPa < \sigma_{\text{all}} = 5000\ KPa$$ Safe!

Addis Ababa University

Faculty of Technology

Department of Civil Engineering

Ac. Year 2004/05 **July 11.2005**
Final Exam **Time allowed 2hrs**
Hydraulic Structures (CEng 408)
Note: Examination is OPEN book

1. With reference to the hydrologic cycle and its component stages discuss how the structural intervention of human being is essential to adjust the spatial variation and temporal variation of water is adjusted in a way suitable for development. (Take illustrative hydraulic structure to elaborate your discussion) **(10 pts)**

2. Design a 55m high constant angle arch dam, by the thin cylinder theory, for a valley 12m wide at base and 68m at a height of 55m. Draw to scale the plan and section on the centerline of the dam. Take σ_{all} 200t/m^2. **(15 pts)**

3. A section of a homogeneous earth dam is shown in
4. Figure 2. Determine the phreatic line and the seepage discharge per meter length through the body of the dam. The coefficient of permeability of the dam material may be taken as 5 x 10-6 m/sec. **(15 pts)**

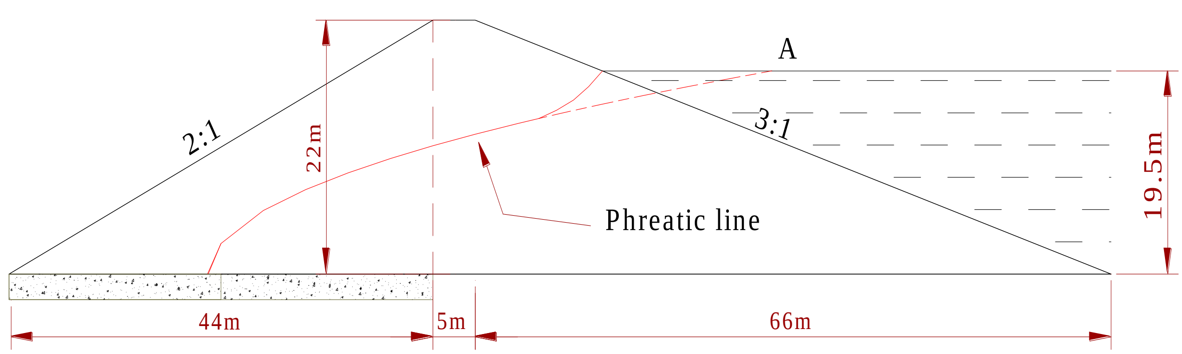


Figure 2 Body of homogeneous earth dam

5. Describe the following terms with the context of hydraulic structures. Use diagrams that reinforce your discussion. **(10 pts)**

   a. Spillways
   b. Outlet structures
   c. Intake structures
   d. Stilling basin
   e. Hydraulic jump

Final-Semester Exam Partial Solution

1. *With reference to the hydrologic cycle and its component stages discuss how the structural intervention of human being is essential to adjust the spatial variation and temporal variation of water in a way suitable for development. (Take illustrative hydraulic structure to elaborate your discussion)* ***(10 pts)***

Water is one of the essential prerequisites of life. A country's water resources include all the water in rivers, lakes, seas, and groundwater which are stages of the hydrologic cycle. The distribution of water in nature in space and time, however, is such that it is scarce at some locations and at particular times and excess at other locations (and at another time at same location). Rainfall, which is the main water input to our ecosystem, is variable in space and time. This is then reflected, for instance, in river flow, groundwater and lake-levels. Some areas get more or less uniform and good amount of rainfall most of the year (like areas in Southwest Ethiopia), whereas other places get their rainfall concentrated in few months (the wet season). Still there are places that get very scanty rainfall. On the other hand the society's demand is not in general synchronised with the availability of water. In fact, some needs, such as irrigation water requirements are high during periods of no, or less rainfall. Hence man is faced with the task of developing the available water resources to meet his needs.

With water needs for domestic use and that for food production being the basic requirements, water needs of a society, both in quantity and quality depend upon its level of development. Water is needed for energy production (hydropower), industrial use, recreation, and navigation (waterways), to mention the most common and traditional ones. Thus projects are designed and implemented to meet all or some of these needs.

The very water that is essential for life may also threaten life. Floods cause from time to time great losses to human life and property. Thus settlements and developments on banks of rivers should be protected from occurring floods, high flows in streams should not cause damage to bridges, etc., for instance by building dykes, In such cases the water has to be controlled so that its harmful consequences are minimised, if not totally prevented.

Consider building of a storage reservoir as a case

The absence of natural storage of adequate capacities necessitates construction of some artificial storage works. Development of natural storages may also be included in this category sometimes (Cherecherea weir at Lake Tana). In rainy season there is excess flow down the valley in a river. An impounding reservoir(human intervention) can be constructed in the valley to store this excess water which will meet the demand in dry periods.

Ctd….

2. *Design a 55m high constant angle arch dam, by the thin cylinder theory, for a valley 12m wide at base and 68m at a height of 55m. Draw to scale the plan and section on the centerline of the dam. Take σ_{all} = 200t/m^2.* ***(15 pts)***

Taking θ = 133.440 [1pt]

$r_i = 0.544B$ **[1pt]**

$$t = \frac{\gamma_w h r_i}{\sigma_{all} - \gamma_w h}$$ **[1pt]**

$r_e = r_i + t$

$\sigma_{all} = 200t/m^2 = 2Mpa$

h	B	r_i	P	P^*r_i	$\sigma_{all} - p$	t	r_e
		[2pts]			[1pt]	[2pts]	[2pts]
0	68.0	37.0	0.0	0.0	2000	0	37.0
5	62.9	34.2	50.0	1711.1	1950	0.88	35.1
10	57.8	31.5	100.0	3145.3	1900	1.66	33.1
15	52.7	28.7	150.0	4302.5	1850	2.33	31.0
20	47.6	25.9	200.0	5182.8	1800	2.88	28.8
25	42.5	23.1	250.0	5786.2	1750	3.31	26.5
30	37.5	20.4	300.0	6112.6	1700	3.6	24.0
35	32.4	17.6	350.0	6162.0	1650	3.73	21.3
40	27.3	14.8	400.0	5934.5	1600	3.71	18.5
45	22.2	12.1	450.0	5430.1	1550	3.5	15.6
50	17.1	9.3	500.0	4648.7	1500	3.1	12.4
55	12.0	6.5	550.0	3590.4	1450	2.48	9.0

Note: Provide a nominal thickness of 1.5 m when necessary. [1pt]

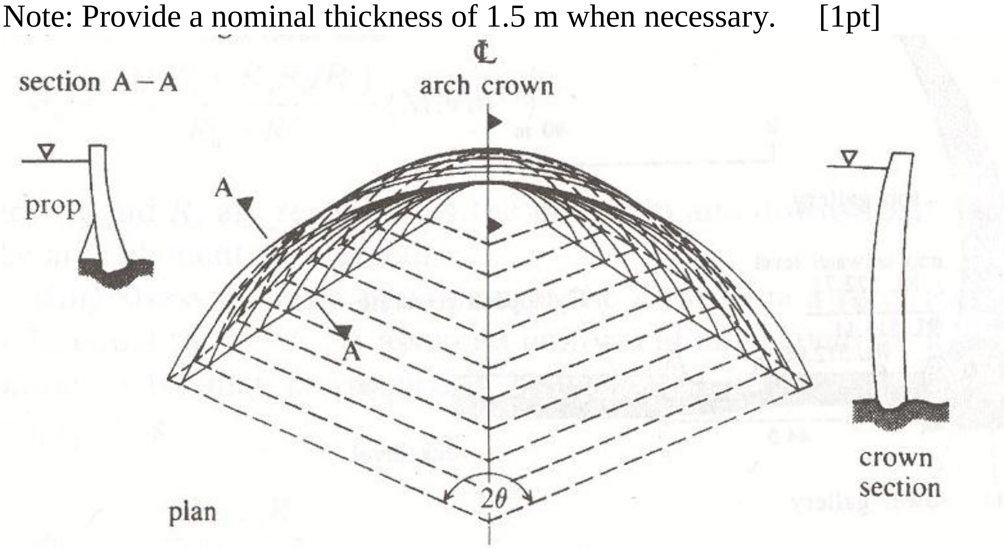


Fig. Section and elevation of constant angle arch dam **[4pts]**

3. *A section of a homogeneous earth dam is shown in*

4. ***Figure 2****. Determine the phreatic line and the seepage discharge per meter length through the body of the dam. The coefficient of permeability of the dam material may be taken as 5 x 10^{-6} m/sec.* ***(10 pts)***

Determination of Phreatic Line

The equation of parabola for seepage through a homogenous earth dam with a horizontal filter at the toe is given by:

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

At Point A x = 30.05, y = 19.5m and S = 5.77m. **[5pts]**
Working out a few more points from the equation, the parabola can be easily drawn and corrected for the curve at the upstream face of the dam, so as to get the seepage line

x	-2.885	0	5	10	12.5	15	20	25	30.5
y2	0	33.29	90.99	148.69	177.54	206.39	264.09	321.79	385.26
Y **[3pts]**	0	5.77	9.54	12.19	13.32	14.37	16.25	17.94	19.63

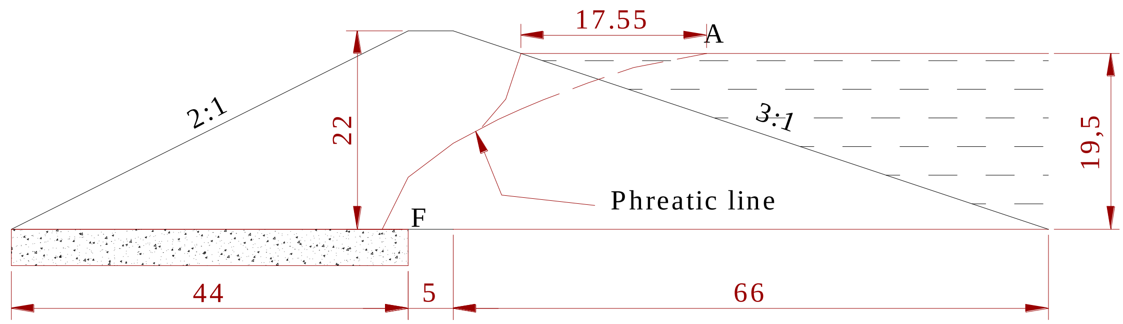


Figure 1 Body of homogeneous earth dam [3pts]

The seepage flow through the dam per unit length is given by

Q = kS **[2pts]**
= 5 x 10^{-6} * 5.77
= 28.9 m^3/sec per meter length. **[2pts]**

Addis Ababa University
Faculty of Technology, Department of Civil Engineering

Ac. Year 2004/05 **July 11.2005**
Final Exam **Time allowed 2hrs**
Hydraulic Structures (CED 355)
Note: Examination is OPEN book

1. *Briefly discuss the checks that are required to be made to investigate the stability of an earthen dam.*

The major types of failures of embankment dam are
Hydraulic failure
Seepage failure
Structural failure
Stability though mainly taken care of through structural analysis the cause could also be attributed to seepage or/and hydraulic failures. Basically stability of a earthen dam is assured through the following three considerations govern the design of an earth embankment.

1. side slopes must be stable;
2. Dimensions must be sufficient to control seepage;
3. Base width must be long enough to distribute weight of dam over sufficient area to prevent overstress in the foundation.

Therefore the checks needed to investigate stability will be **[1pt each total 5 pts]**

a. Stability of upstream slope during sudden drawdown
b. Stability of upstream portion of the dam, during sudden drawdown, from the consideration of horizontal shear developed at base under the upstream slope of the dam
c. Stability of downstream slope under steady seepage from the consideration of horizontal shear at base under the downstream slope of the dam
d. Stability of the foundation against shear
e. Overall stability of the dam section as a whole

2. *Explain how the following parameters affect design of an earth dam*:
   a. *Optimum moisture content (1.5 pts)*

Optimum moisture content is the moisture content at which the weight of soil grains obtained in unit volume of the compacted soil mass is maximum. The compaction of soil in the fill, particularly high earth dam, is to obtain high density of the soil to reduce settlements, to reduce percolation through the fill, and to increase its shear resistance. All this factors lead to increased safety and stability of the embankment dam.

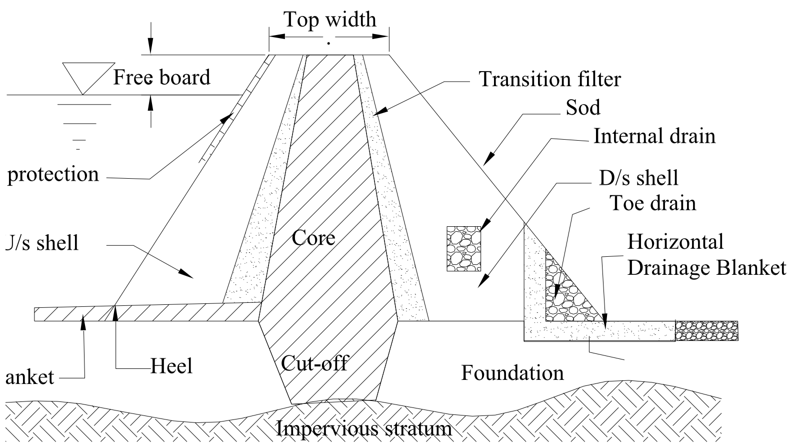
   b. *C(cohesion) and ϕ(internal friction) value of soil; permeability of soil (2 pts)*

Both parameters determine the shear strength of embankement soil

$$\tau = C + \sigma \tan \phi$$

   c. *Sudden draw-down of the reservoir **(1.5 pts)***

3. *Illustrate with neat sketch* ***(1 pt each)*** *the following parts of an earthen dam and state their function* ***(1 pt each)*** *briefly;*
    a. *Toe drain*
    b. *Horizontal drainage blanket*
    c. *Cut-off*
    d. *Rip-rap*
    e. *Top width*

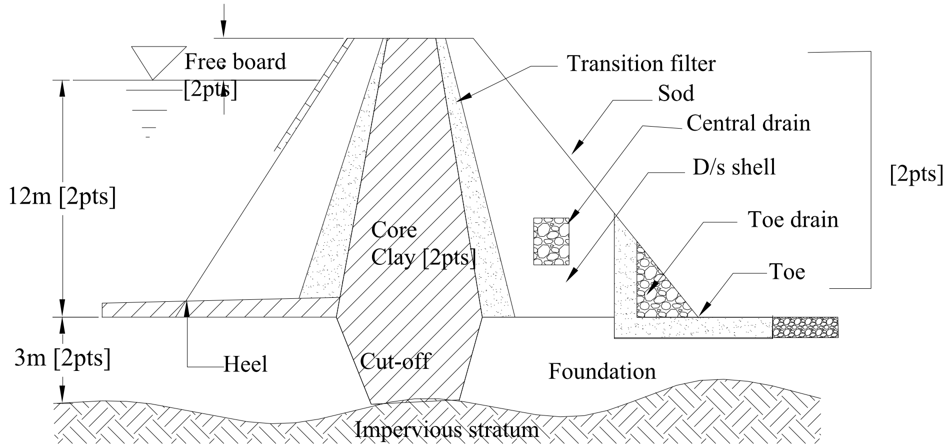


4. *Explain briefly how the stability of earthen slopes is checked by slip circle method.* ***(10 pts)***

Sketchof the slip circle 2pts
Discussion about the method [2pts]
How to get the critical slip circle [2pts]
Free body diagram of interacting forces [2pts]
Expressing moments to determine factor of safety [2pts]

Pls refer to the steps in your handout.

5. *An earthen dam has to be constructed to store a maximum depth of 12m of water over river bed consisting of coarse sand and gravel up to a depth of 3m below river bed followed thereafter by hard and sound rock. Clay soil is available in plenty in the vicinity of the river. Draw and detail a suitable section of the dam at the river bed.* ***(10 pts)***



6. *Describe the following terms with the context of hydraulic structures. Use diagrams that reinforce your discussion.* ***(2 pts each)***
    a. *Spillways*
    b. *Outlet structures*
    c. *Intake structures*
    d. *Stilling basin*
    e. *Hydraulic jump*

[1pt] for each description and **[1pt]** for the respective diagrams.

a) Spillway is a dam component which serves to release excess flood from a reservoir efficiently and safely.
b) Outlet structures are structures that serve to regulate or release water impounded by a dam. The release could be at a retarded rate (detention dams), diversion into a canal or pipeline (diversion dam), as dictated by downstream needs or to empty reservoir for inspection.
c) Intake structures are situated at the entrance of canals, tunnels or pipes through which water is conveyed to where it is needed.
d) Stilling basin is a channel structure of mild slope, placed at the outlet of a spillway, chute or other high velocity flow channel, whose purpose is to confine all or part of the hydraulic jump or other energy reducing action and dissipate some of the high kinetic energy of the flow. It is a structure which is necessary to prevent bed scour and undermining of the structure in situation where high velocity flow is discharged into the downstream channel.
e) Hydraulic Jump is the jump of a flowing water that takes place when the supercritical flow changes into a subcritical flow. When water falls over a spillway or a vertical fall, it acquires a lot of momentum and velocity. This high velocity, if not checked, will cause large scale erosion and scouring of the downstream soil. The hydraulic jump can be used with great advantage to dissipate this excess kinetic energy of the water.

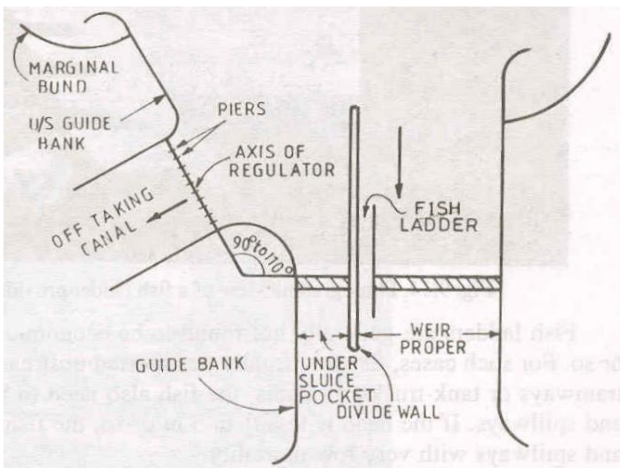
7. a. *What are the functions of canal head regulator?* ***(5 pts)***

Canal Head Regulator is provided at the head of an off-taking canal. Its functions are:

- To regulate the supply of water entering the canal
- To control the entry of silt in the canal
- To prevent the river flood water from entering the canal

b. How does "Scouring sluice" differ from those of "Head Sluices"? ***(5 pts)***

These are openings provided in the body wall of the weir, at the **river bed level** [head sluices at the head regulator] and in the portion close to the head regulator. The main function of scouring sluices is to allow the silt-ladden lower portion of the water **to pass out to the river [head sluices to the off take canal]**and hence reducing silt entry into the canal.



KISAMA AFRICA UNIVERSITY COLLEGE
DEPARTMENT OF CIVIL ENGINEEREING

Mid-Semester Exam **Date Dec. 22, 04**
Hydraulic Structure I: **Time allowed 2 ½ hrs.**
The examination is Open book.

Part I Multiple Choice

Among the four alternative answers choose the best answer that fits to each question. Each question has 2 marks.

21. Earthen dams are:
   a. rigid dams c. overflow dams
   b. non rigid dams d. diversion dams

22. when sand and gravel foundation strata is available at a proposed dam site of moderate height, the dam may be of the type:
   a. Earthen dam or rock fills dam c. double arch dam
   b. masonry gravity dam d. concrete gravity dam

23. Yield of a reservoir represents:
   a. The inflow into the reservoir c. the outflow demand on the reservoir
   b. The capacity of the reservoir d. none of the above

24. The capacity of a storage reservoir can be decided by using:
   a. The mass curve of inflow c. by both (a) and (b)
   b. The mass curve of outflow d. none of the above.

25. According to thin cylinder theory the volume of concrete required for an arch dam would be minimum if the central angle is:
   a. $150^0 34'$ c. $136^0 34'$
   b. $133^0 34'$ d. $140^0 34'$

26. The slope of the upstream face of a flat-slab type buttress dam
   a. Is always less than 35^0 c. usually varies from 35^0 to 45^0
   b. Is always more than 45^0 d. none of the above

27. When seepage takes place through the body of an earthen dam, it leads to:
   a. development of pore pressures in the dam body c. reduction in the shear stresses in the dam
   b. reduction in the shear strength of the dam d. both (a) and (b)

28. When the water level standing against an earthen embankment, suddenly falls down, then there is an imminent risk of sliding failure, to the:
   a. upstream slope c. both (a) and (b)
   b. downstream slope d. none of the above

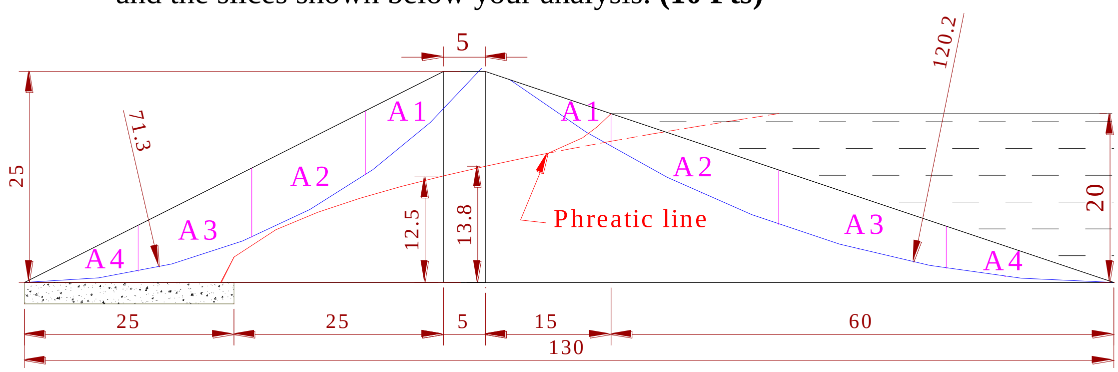
29. During the maintenance of an earthen dam, the apparent seepage through the foundation of the dam is best taken care of , by providing:
   a. A chimney drain c. a drain trench along the d/s toe
   b. A rock toe d. an u/s impervious cutoff

30. Which one of the following spillways is least suited to earthen dams?
   a. Ogee spillway c. chute spillway
   b. Side channel spillway d. shaft spillway

Part II Design Problem

2.Design the embankment dam shown below. Show all the necessary procedures and important consideration in the process.

a. Draw the seepage line inside the dam when the reservoir is full. **(10 Pts)**
b. Check the overall stability and the safety of the dam against sliding. **(10 Pts)**
c. Analyze the stability of the slopes using Swedish Slip Circle method. Show how you obtain the locus of center of failure surfaces and consider toe failure case and the slices shown below your analysis. **(10 Pts)**



Geometric properties of slices.

	Upstream				Downstream		
Slice	Area(m^2)	α (deg)	h_w (m)	L (m)	Area (m^2)	α (deg)	L (m)
A1	25.97	32	0	14.5	68.58	40	18
A2	111.68	24	6.1	22	111.48	28	15.4
A3	120.83	15	6.2	20.7	96.1	17	14.2
A4	55.52	4	3.6	20.1	40.11	7	13.7

Area in relation with phreatic line.

Area	Dam	U/s shoulder	D/s shoulder
under seepage line	1102.08	838.1	221
Dry portion	584.92	99.4	404
Total	1687	937.5	625

Shimelis Behailu (Ato)

KISAMA AFRICA UNIVERSITY COLLEGE
DEPARTMENT OF CIVIL ENGINEEREING

Final-Semester Exam Partial Solution **Date April 8, 2013**
Hydraulic Structure I: **Time allowed 2 hrs.**
Note: The examination is Open book.
Justify any of your assumption and use of missing information.
The exam is marked out of 50%

1. *Describe how you would do a policy analysis to decide on whether or not to remove a dam already constructed on a large reservoir. What kind of factors would you need to include in your analysis? What are some of the potential harms and benefits of dam removal? What kinds of scientific data should we collect? (**10 pts**)*

For removal of an existing large dam, there has to be a sound reason as it leads to a lose of huge sum and social crises. The removal of any dam need to produce a tangible benefit either in social perspective, or economic terms. Tthe following flow chart can be used as a starting guideline to relate the removal with the existing policies at every stage of the assessment.

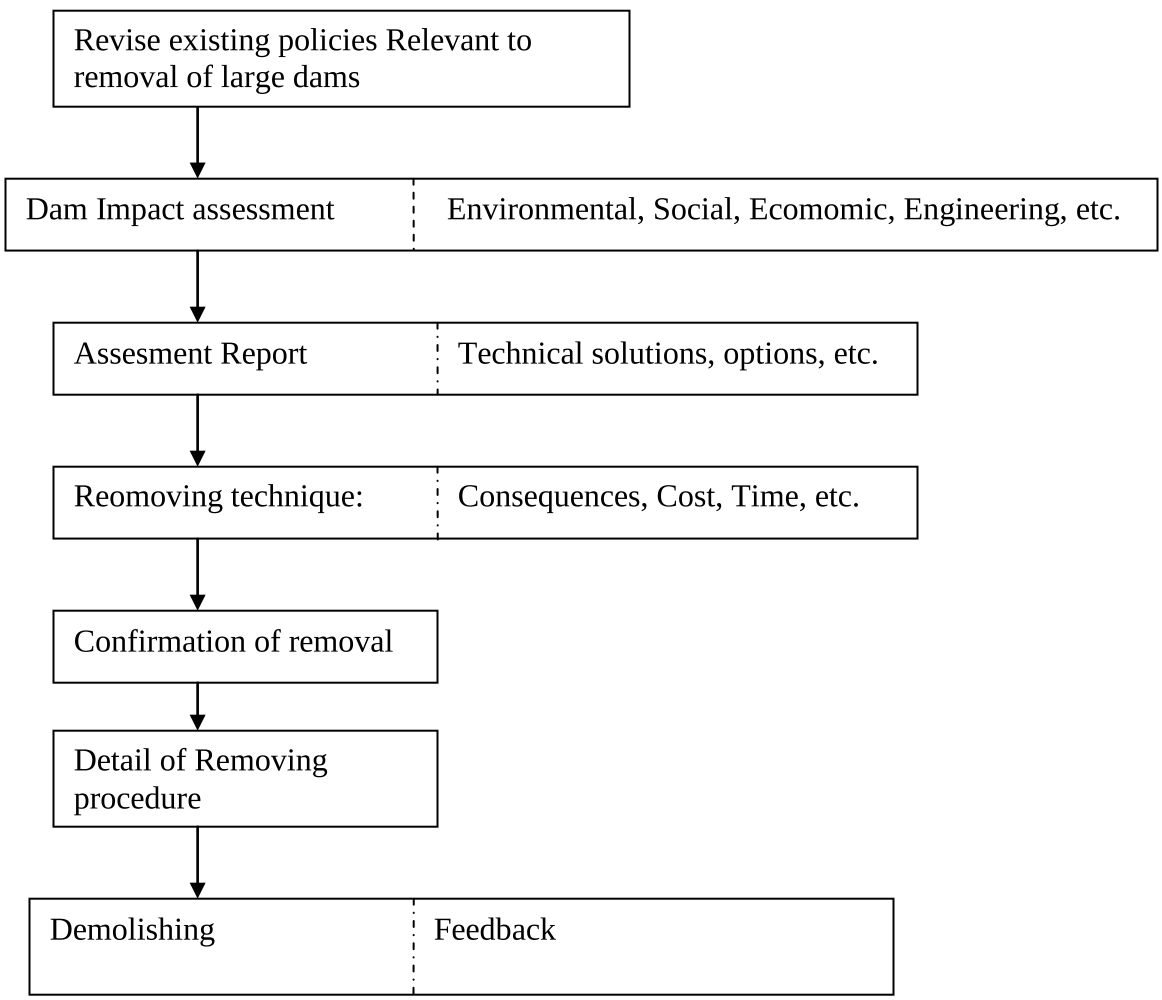


Fig. 1. Stages for demolishing of dam**(4.0 Pts)**

Here are some of the major factors to be considered in the policy analysis **(2.0 Pts)**

8. Does the dam negatively affect the welfare of the society?
9. Does the dam created a political instability or conflict between beneficiaries?
10. Is there a new technology that could produce better economic advantage than the existing one?
11. Is the dam causing environmental problems that outweigh its benefit?
12. Will the removal affect the ecological balance of the area?
13. Is there any discovery of precious mineral with in the reservoire site?
14. Is the dam susceptible to failure due to structural defects or geologic reasons that are not observed during design and construction?

Among the potential harms due to removal of large dams **(1.0 Pt)**

- Flooding of downstream areas
- Lose of money on investments related to the dam such as irrigation schemes, hydropower, etc.
- lose of job for the workers who make their living on activites related to the dam
- ecological disturbance
- Cost of demolishing

Among the benefits from removal of large dam **(1.0 Pt)**

- Restoration of the natural water course
- More land if the reservoire is suitable for agriculture or urbanization
- Avoidance of the risk that lead to the removal of the dam, (eg. Failure, disease, political or social reasons, etc.)
- Better utilization using state of the art technology could be possible
- Lesson for future water resource projects

For the analysis various scientific data should be collected **(1.0 Pts)**

Socio-ecomoic data

- Market value of products from the dam (amount of power, irrigation output, etc.)
- The impact of the dam on the neigboring areas (# of casualities, extent of various damages due to the presence of the dam)
- Need of resettlement if there are areas prone for flooding

Technical data

- The total cost of dam construction, dimensions, unique features and the size of the dam
- Hydrologic data
- Capacity of the valley
- Maxium discharge to be released
- Appropriate time for demolishing
- Cost of demolishing
- Manpower for demolishing etc.

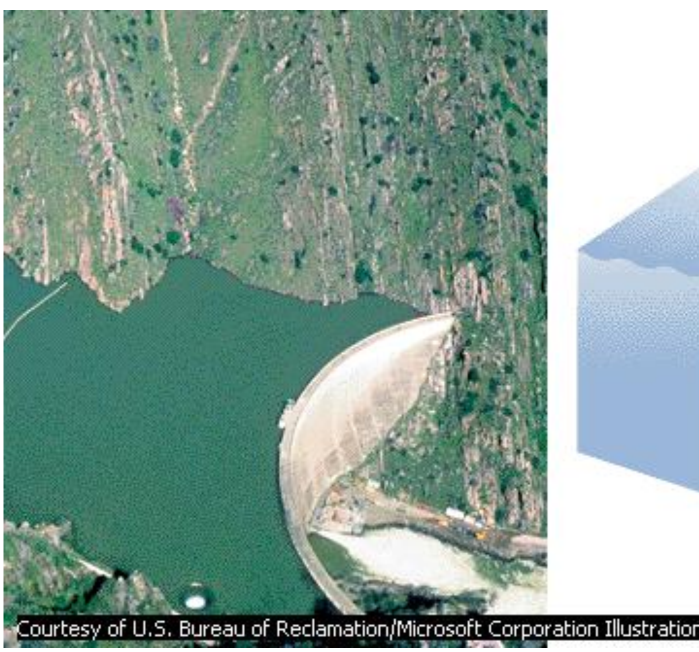
2. *List down exhaustively the similarity and difference between arch dam and buttress dam (use illustrative sketch to reinforce your discussion).* ***(10 pts)***

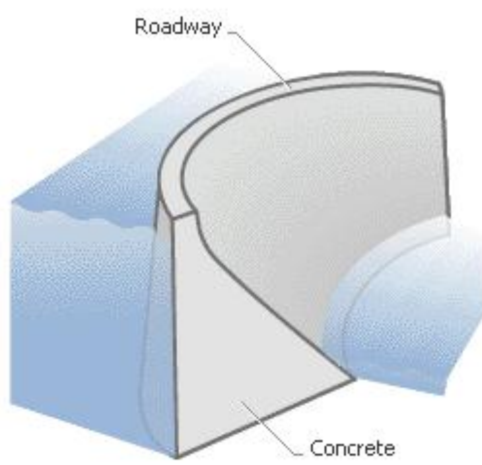
Similarity [4.0 pts]

i. Material: Concrete dams
ii. Section: Thin concrete sections
iii. Uplift pressure: Small/insignificant
iv. Concrete Volume: small compared to gravity dam
v. Foundation: strong or weak foundation
vi. Design procedure: complex
vii. Construction: needs skilled man power, etc.
viii. Susceptible to sabotage

Difference [4.0 pts]

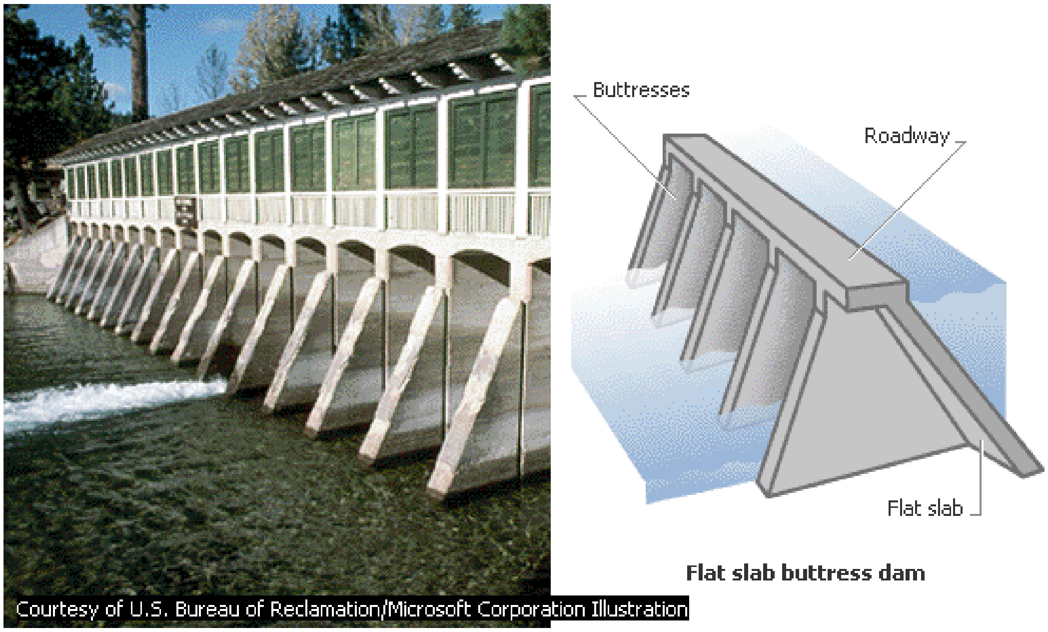
Feature	**Arch dam**	***Buttress dam***
Load transfer	Arch action	*Gravity*
Support	Strong abutment	*Suitable bed*
Valley	Narrow valley	*More flexible*
Shape	*Considerable curvature*	*Straight/small curvature*
Spillway	*Separate*	*Possible to integrate*
Structural element	*Curved member*	*Deck and Buttress*
Upstream face	*Straight/small curvature*	*Considerably inlined*





Arch dam

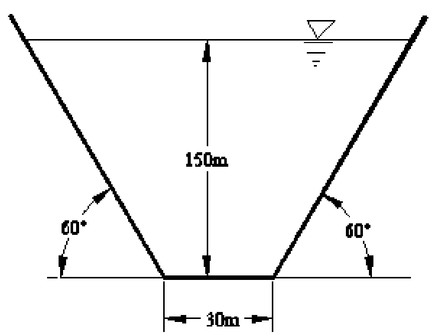
Monticello Dam impounds Putah Creek west of Sacramento, California. The solid concrete structure stands 93 m tall. The dam's arched upstream face transfers some of the pressure from its reservoir, Lake Berryessa, onto the walls of the canyon. This design enables an arch dam to be much less massive than an equivalent gravity dam, which relies solely on the force of its weight to hold back the water in a reservoir. While Monticello Dam measures 30 m at its base, an equivalent gravity dam might be more than five times as thick at the base.



Lake Tahoe Dam impounds the Truckee River in northern California. Like all flat slab buttress dams, it has a flat slab upstream face supported by a series of buttresses on the downstream side. Lake Tahoe Dam measures 5.5 m tall and 33 m long. It was completed in 1913 to raise the water level in Lake Tahoe, a natural lake, to provide additional water for crop irrigation.

Fig. 2. Arch dam and Flat Slab Buttress Dam **[2.0 pts]** *(Microsoft ® Encarta ® Reference Library 2005. © 1993-2004 Microsoft Corporation. All rights reserved.)*

3. *Design a* constant *angle arch dam by the thin cylinder theory for a valley 30m wide at base and sides rising at* 60^0 *to the horizontal on both sides. Height of the dam is 150m and the safe stress is* $210t/m^2$ ***(15 pts)***



Taking θ = 133.44^0

$r_i = 0.544B$

$$t = \frac{\gamma_w h r_i}{\sigma_{all} - \gamma_w h}$$

$$r_e = r_i + t$$

Efficient central angle, sketch of a section and design formulae **[3.0 pts]**

Table 1. **[10 pts]**

h	B	r_i	P = γh	P*r_i	σ_{all} –p	t	r_e
0	203.2	110.5	0	0	2100	0	110.5
10	191.7	104.3	100	10430	2000	5.2	109.5
20	180.1	98	200	19600	1900	10.3	108.3
30	168.6	91.7	300	27510	1800	15.3	107
40	157	85.4	400	34160	1700	20.1	105.5
50	145.5	79.2	500	39600	1600	24.8	104
60	133.9	72.8	600	43680	1500	29.1	101.9
70	122.4	66.6	700	46620	1400	33.3	99.9
80	110.8	60.3	800	48240	1300	37.1	97.4
90	99.3	54	900	48600	1200	40.5	94.5
100	87.7	47.7	1000	47700	1100	43.4	91.1
110	76.2	41.5	1100	45650	1000	45.7	87.2
120	64.6	35.1	1200	42120	900	46.8	81.9
130	53.1	28.9	1300	37570	800	47	75.9
140	41.5	22.6	1400	31640	700	45.2	67.8
150	30	16.3	1500	24450	600	40.8	57.1

Provide a nominal thickness of 2.0m. **[2.0 pts]**

4. *a. A section of a homogenous earth dam with 8m long downstream filter layer is shown below. Draw the profile of the phreatic line and calculate the seepage discharge per meter length through the body of the dam. The coefficient of permeability of the dam material is 8 x 10^{-5} m/sec.* ***(7 pts)***

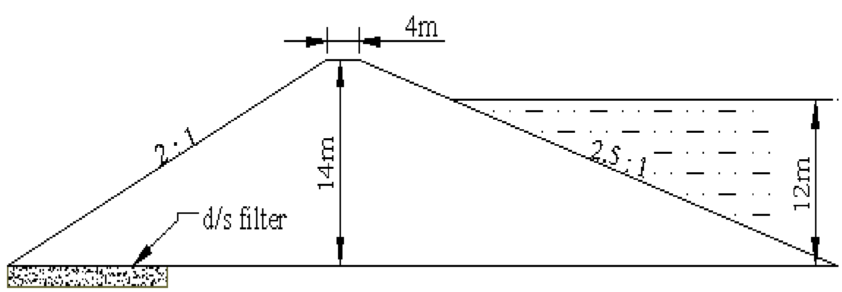


Fig. 3. Section of homogeneous earth dam

For the origin of the Cartesian co-ordinate system at the face of the filter, the equation of the parabola of the seepage line can be expressed as:

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

At point A, x = 39m, and y = 12m. Inserting into the parabola equation, S = 1.80m. **[1.5 pt]**Working out a few more points from the equation, the parabola can be easily drawn and corrected for the curve at the upstream face of the dam, so as to get the seepage line. **[2.5 pts]**

x	-0.9	0	5	10	15	20	29	39
y^2	0	3.24	21.24	39.24	57.24	75.24	107.64	143.64
y	0	1.8	4.61	6.26	7.57	8.67	10.37	11.98

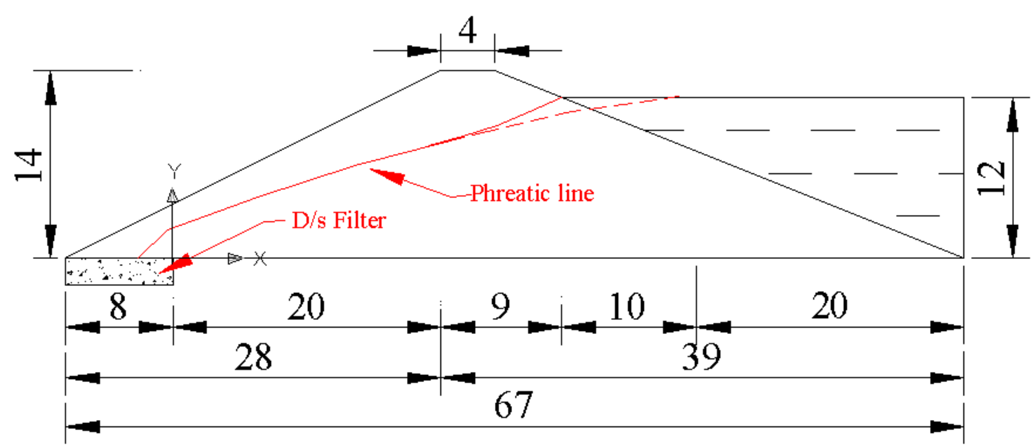


Fig. 4 Profile of phreatic line for an earth dam **[2.0 pts]**

The amount of seepage flow is **[2.0 pts]**

$$Q = kS$$
$$= 8 * 10^{-5} * 1.80$$
$$= 14.40 * 10^{-5} m^3/\text{sec per meter width of dam}$$

a. *Briefly discuss the major types of failures of an earth dam and the checks/measures that are required to be done so as to avoid these failures.**(8 pts)***

The major types of failures of embankment dam are **[0.5 pt each total 1.5 pts]**

i. Hydraulic failure
ii. Seepage failure
iii. Structural failure

Stability though mainly taken care of through structural analysis the cause could also be attributed to seepage or/and hydraulic failures. Basically stability of a earthen dam is assured through the following three considerations govern the design of an earth embankment. **[0.5 pt each total 1.5 pts]**

4. side slopes must be stable;
5. Dimensions must be sufficient to control seepage;
6. Base width must be long enough to distribute weight of dam over sufficient area to prevent overstress in the foundation.

Therefore the checks needed to investigate stability will be **[1pt each total 5 pts]**

a. Stability of upstream slope during sudden drawdown
b. Stability of upstream portion of the dam, during sudden drawdown, from the consideration of horizontal shear developed at base under the upstream slope of the dam
c. Stability of downstream slope for full reservoir
d. Stability of downstream slope under steady seepage from the consideration of horizontal shear at base under the downstream slope of the dam
e. Stability of the foundation against shear
f. Overall stability of the dam section.

KISAMA AFRICA UNIVERSITY COLLEGE
DEPARTMENT OF CIVIL ENGINEEREING

Mid-Semester Exam **Date April 8, 2013**
Hydraulic Structure I: **Time allowed 2 hrs.**

Note: The examination is Open book.
Justify any of your assumption and use of missing information.
The exam is marked out of 30%

5. *Among water preservation and storage techniques one is construction of underground dam that stores water with in the pore spaces of the soil/aquifer. Enumerate the basic necessary engineering investigations that would help for site selection, planning design and implementation of such projects.* ***(5 Pts)***

Among the basic Engineering Investigations:

- Surface and subsurface hydrology of the ground water basin
- The permeability and porosity of the ground aquifer
- The geology of the area and the dam site
- The strength of the rock/soil for penetration
- The areal extent of the aquifer
- Relevant criteria among the surface dam site investigation.
- Similar informations as for the surface dams on the demand analysis and location of the area to be seved.

6. *Describe in brief general construction procedures and quality control techniques that you may follow in gravity dam construction.* ***(5 Pts)***

The Construction Process

1. *Dry construction area:* Before construction can begin on any dam, the water in the streambed must be diverted or stopped from flowing through the site. As in the case of fill dams, a coffer-dam (a temporary structure to impound the water) must be built or the water must be diverted into another channel or area down-stream from the dam site. For large projects, this construction may be done several seasons before building of the dam begins. The flow of water is closed off at the very last moment.
2. *Foundation:* The foundation area for any concrete dam must be immaculate before the first concrete for the dam is placed. As for fill dams, this is a detailed process of excavating, cleaning, and repairing the rock throughout the foundation "footprint" and on both abutments (the sides of the canyon that form the ends of the dam). Sites immediately downstream of the dam for any powerplant, stilling basin, or other structure must also be prepared.

   At some sites, extensive work may be required. If the rock in the foundation or abutments is prone to fracturing because of the load imposed by the dam and its reservoir, earthquake activity, or the properties of the rock, it may be necessary to install extensive systems of rock bolts or anchor bolts that are grouted into the rock through potential fracture zones. On the abutments above the dam, systems of rock bolts and netting may be required to keep large rock fragments from falling onto the dam. Instruments to monitor groundwater levels, joint movement, potential seepage, slope movements, and seismic activity are installed beginning during the early stages of foundation preparation through completion of the dam.

A cutoff wall may be excavated deep into rock or holes may be drilled in the foundation for the installation of reinforcing steel, called rebars, that extend up into the dam and will be tied to the steel inside the first lifts of the dam. The idea is to build a reservoir that, like a bowl, is equally sound around its perimeter. The water is deepest and heaviest at the dam (when the reservoir is near capacity) so the dam and its foundation cannot be a weak point in that perimeter.

3. *Formwork and concrete casting:* Forms made of wood or steel are constructed along the edges of each section of the dam. Rebar is placed inside the forms and tied to any adjacent rebar that was previously installed. The concrete is then poured or pumped in. The height of each lift of concrete is typically only 1.5-3 m and the length and width of each dam section to be poured as a unit is only about 15 m. Construction continues in this way as the dam is raised section by section and lift by lift. Some major dams are built in sections called blocks with keys or inter-locks that link adjacent blocks as well as structural steel connections.

   The process is much like constructing a building except that the dam has far less internal space; surprisingly, however, major concrete dams have observation galleries at various levels so the condition of the inside of the dam can be observed for seepage and movement. Inlet and outlet tunnels or other structures also pass through concrete dams, making them very different from fill dams that have as few structures penetrating the mass of the dam as possible.

4. *Early dam performance:* As soon as a significant portion of the dam is built, the process of filling the reservoir may begin. This is done in a highly controlled manner to evaluate the stresses on the dam and observe its early performance. A temporary emergency spillway is constructed if dam building takes more than one construction season; lengthy construction is usually done in phases called stages, but each stage is fully complete in itself and is an operational dam. The upstream cofferdam may be left in place as a temporary precaution, but it is not usually designed to hold more than minimal stream flows and rainfall and will be dismantled as soon as practical. Depending on design, some dams are not filled until construction is essentially complete.
5. *Appurtenances:* The other structures that make the dam operational are added as soon as the elevation of their location is reached as the dam rises. The final components are erosion protection on the upstream (water) side of the dam (and sometimes downstream at the bases of outlet structures), instruments along the crest (top) of the dam, and roads, side-walks, streetlights, and retaining walls. A major dam like Hoover Dam has a full-fledged roadway along its crest; small dams will have maintenance roads that allow single-file access of vehicles only.

   Away from the dam itself, the powerhouse, instrument buildings, and even homes for resident operators of the dam are also finished. Initial tests of all the facilities of the dam are performed.

6. *Completion:* The final details of constructions are wrapped up as the dam is put into service. The beginning of the dam's working life was also carefully scheduled as a design item, so that water is available in the reservoir as soon as the supply system is ready to pump and pipe it downstream, for example. A program of operations, routine maintenance, rehabilitation, safety checks, instrument monitoring, and detailed observation will continue and is mandated by law as long as the dam exists.

Quality Control

There is no dam construction without intensive quality control. The process of building alone involves heavy equipment and dangerous conditions for construction workers as well as the public. The population living downstream of the dam has to be protected over the structure itself; the professionals who design and construct these projects should absolutely be committed to safety, and they are monitored by local, regional, and federal agencies.

7. *List down exhaustively the strength and weaknesses of Bligh's, Lane's and Khosla's seepage theories.*

8. Design a non-overflow gravity dam by the Single-step method using the following data. (Ignore earthquake and silt pressure) **(15 pts)**

Item	**Value**	**Item**	**Value**
H_{max} (depth of headwater)	45 m	*f* (friction factor)	0.75
h_e (spillway crest to MWL)	3 m	s_a *(Shear strength)*	4.5 MPa
Tail water	None	s_{sf} (Shear safety factor)	5
Top width	7.5	ζ (Uplift factor)	0.5
Hs (depth of silt-water mixture)	4 m	C (uplift area factor)	1
Ss(Specific gravity of silt)	1.5	σ_c' (concrete ultimate strength)	30 MPa
γ_{ss} (for horizontal silt water presure	14 kN/m^2	F (Fetch length)	5 km
α (earthquake factor)	0.12	V (Wind Velocity)	128 km/hr
T (period of EQ vibration)	0.80 sec	γ_c (concrete Specific unit weight)	22 MPa
		γ_w(water specific unit weight)	10 MPa

Partial design solution

Determine the wave height by the empirical equations

$$h_w = 0.763 + 0.032\sqrt{vf} - 0.271 f^{1/4} \qquad \text{for } f < 32\,km$$

$$h_w = 0.763 + 0.032\sqrt{128 * 5} - 0.271 * 5.0^{1/4}$$

$$= 1.17 \text{ m}$$

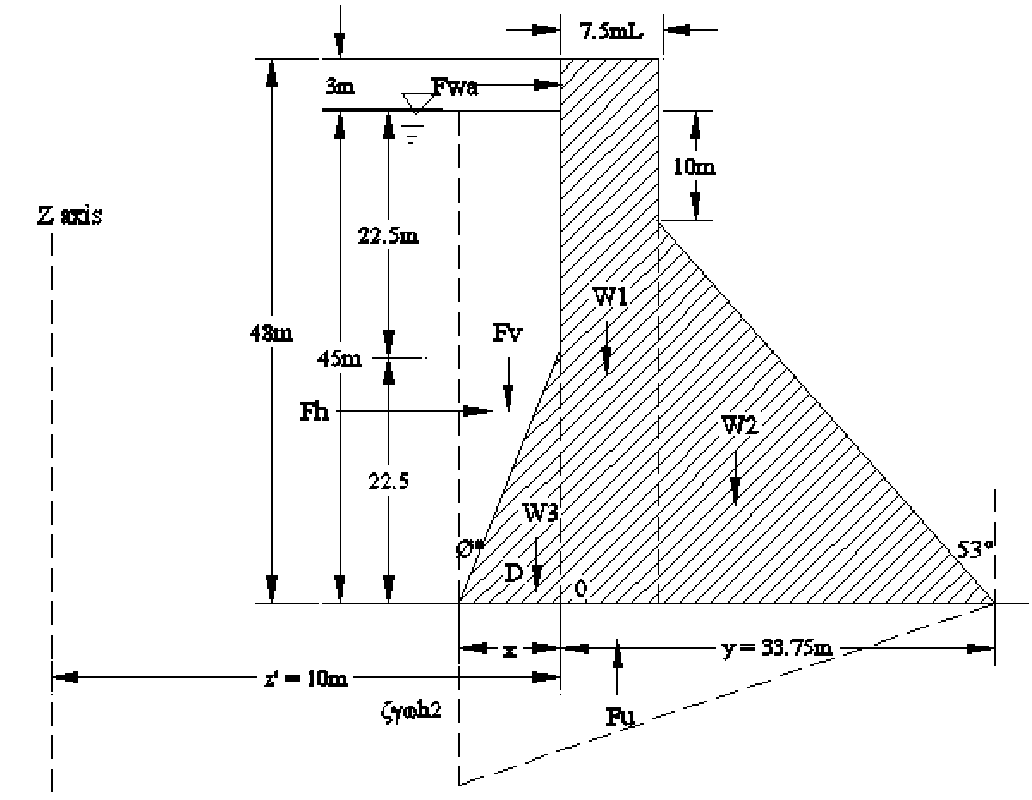
Rise of water wave $= 1.33h_w$

$= 1.56$ m;

With an allowance of 0.14 m, free board = 1.70m

$$F_{wv} = 2.0\gamma_w h^2_w$$
$$= 2.0 * 10 * 1.17^2$$
$$= 27.40 \text{ kN/m}$$

Point of application = 3/8 x 1.17 =0.44m above still water level.



Line	Item	Description & dimension	Forces: Horizontal	Forces: Vertical	Lever	Moment	Remark
1	W0	7.5*46.7*22		7705.5	13.75	105950.6	
2	W1	0.5*35*26.25*22		10106.25	26.25	265289.1	
		Total Partial empty		**17811.75**	**20.84**	**371239.7**	
Trial I		Estimation of x	Z'=	43.75			
		2B/3=Z' - 25.26 =	22.91	B=	34.37		
		x =	**0.75**	Z" =	9.25		
		Z"+B/3 =	20.70667	Z"+2B/3 =	32.16333		
3	W3	0.5*22.5*0.8*22		185.625	9.75	1809.84	
		Reservoir Empty		**17997.38**	**[20.73]**	**373049.5**	**Ok!**
4	Fv	Water column 0.3*25*10		253.125	9.63	2437.594	
5	Fu	Uplift: 0.5*B*0.5h*10		-3866.63	20.71	-80077.8	
6	Fh	Water Pressure	10125		15	151875	
7	Fwa	Wave action	27.4		45.44	1245.06	
		Reservoir Full	**10152.4**	**14383.88**	**[31.18]**	**448529.4**	**Ok!**

Check for sliding

$$\frac{\Sigma H}{\Sigma V} = \tan\theta = \frac{10152.4}{14383.88} = 0.71 < 0.75$$ Friction alone is sufficient. (Safe !)

Check for Stresses

Reservoir empty

$$P_V^{'}, P_V^{''} = \frac{\Sigma W}{B}\left(1 \mp \frac{6e}{B}\right) = \frac{17997.38}{34.37}\left(1 \mp \frac{6 * 5.71}{34.37}\right) = 1045.1, 0$$

$$P_V^{'} = 1045.1 \qquad P_V^{''} = 0$$

$$P_i^{'} = p_v^{'}(1 + \tan^2\phi^{'}) = 1045.1(1 + 0.0.033^2) = 1046.3kPa$$

$$P_V^{''}, P_i^{'} < \sigma_{rock,all} = 4000\text{kPa}$$ (safe!)

Reservoir full

$$P_V^{'}, P_V^{''} = \frac{\Sigma W}{B}\left(1 \pm \frac{6e}{B}\right) = \frac{14383.88}{34.37}\left(1 \pm \frac{6 * 4.75}{34.37}\right) = 757.7, 70.88$$

$$P_V^{'} = 757.7 \text{ kPa} \qquad P_V^{''} = 70.88\,kPa$$

$$P_i^{'} = p_v^{'}(1 + \tan^2\phi^{'}) = 757.7(1 + 1.327^2) = 2092.0kPa$$

$$P_V^{''}, P_i^{'} < \sigma_{rock,all} = 4000\ kPa$$ (safe!)

ADDIS ABABA UNIVERSITY
FACULTY OF TECHNOLOGY
DEPARTMENT OF CIVIL ENGINEEREING

Mid-Semester Exam Partial solution (2005/2006 Ac. Yr.) **Date April 8, 2013**
Hydraulic Structures I (CEng 408) **Time allowed 2.0 hrs.**

Note: 1. Each question has equal marks [Total 30%]
2. Make valid assumptions whenever necessary.
3. The examination is OPEN book.

11. A reservoir is contemplated on a stream which has an annual average runoff 500 million m^3. Measurements indicate that the average sediment inflow is 2 x 10^9 N/year. Assuming that a cubic meter of settled sediment will dry out to a density of 12000 N/m^3. The original capacity of the reservoir is 24 x 10^6 m^3. The percentage of the inflowing sediment retained in the reservoir, Trap efficiency = *100[1- (1/(1 + 65Z))]*^{2}, where *Z = capacity - inflow ratio*. Determine the design life of reservoir based on sedimentation of 65% of the original capacity.

Solution

Input information/data ***[1.0 pts]***

Original Reservoir Capacity	*24,000,000*	*M^3*
Average Annual Inflow	*500,000,000*	*m^3/year*
Average annual Sediment load	*2,000,000,000*	*N/year*
Specific Weight of Sediment	*12,000*	*N/m^3*
Maximum %age of Sediment deposit	*65*	
a	*65*	
n	*2*	
Trap efficiency	*0.573*	*fraction*

Trap efficiency of the reservoir varies with the remaining/residual capacity of the reservoir. Hence the amount of sediment filling the reservoir each year is varies which then affects the capacity inflow ratio, z, accordingly. The usual approach is, therefore, to divide the % fill into five zones and determine the average capacity inflow ratio as shown in the table below.

Steps	*Res. Capacity*	*Inflow*	*Z*	*Trap Eff.*	*Sed. %*	*Sed. Vol*
0	*24,000,000*	*500,000,000*	*0.048*		*0*	
1	*20,880,000*	*500,000,000*	*0.042*	*0.536*	*13*	*3120000*
2	*17,760,000*	*500,000,000*	*0.036*	*0.491*	*26*	*6240000*
3	*14,640,000*	*500,000,000*	*0.029*	*0.427*	*39*	*9360000*
4	*11,520,000*	*500,000,000*	*0.023*	*0.359*	*52*	*12480000*
5	*8,400,000*	*500,000,000*	*0.017*	*0.276*	*65*	*15600000*
Average capacity inflow ratio is 0.418 **[2.0 pts]**						

For an average capacity-inflow ratio of different capacities the number of years in which the dam will have a water storage capacity of more than 35% is determined as

0.65 x *original capacity = # of years* x *average trap efficiency* x *sediment yield / specific unit weight of sediment.* ***[2.5 pts]***

0.65 x *24* x *10^6 m3 = # of years* x *0.418* x *2* x *10^9 N/year / 12000 N/m3*

Hence design period of the dam will be about 224 years. ***[2.0 pts]***

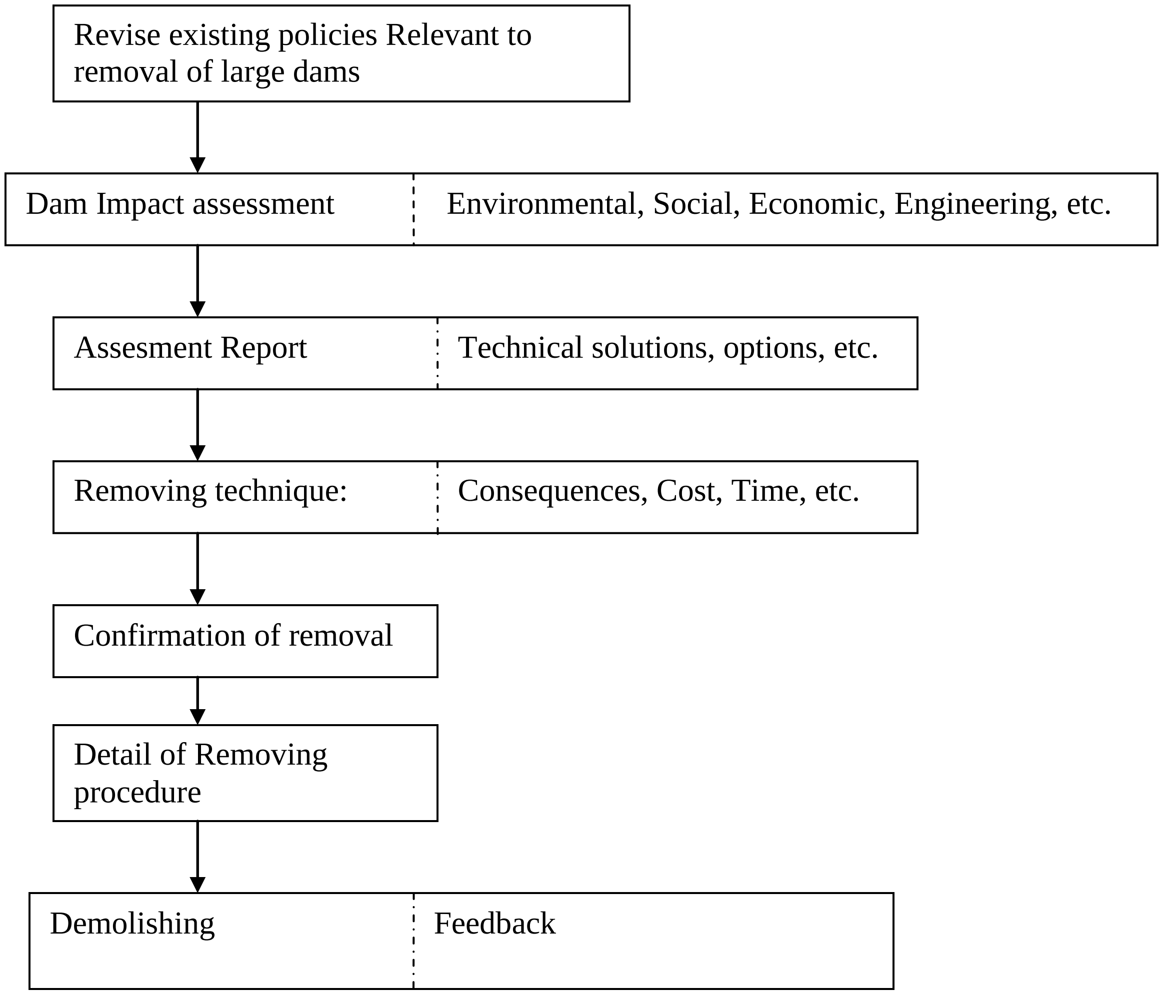
12. Assume you are assigned as a technical and course of action advisor in a decision process to demolish a dam already constructed on a large reservoir.

a. Describe how you would perform the strategic analysis to decide on whether or not to takeout the structure.
b. Detail factors that, in your opinion, are significant and need to be addressed in the analysis.
c. Enumerate potential harms and benefits of the removal.
d. Specify the key data types you may collect from technical (engineering) perspective.

Solution

a. **[2.0 pts]** *In* order *to construct a new dam or remove an existing one, the analysis of relevant policies of water resources development in general and construction of dams and reservoirs in general should first be done. Such policies intermingle different disciplines; hence, a good understanding of every aspect is the corner stone for the final decision.*

For removal of an existing large dam, there has to be a sound reason as it leads to a lose of huge sum and social crises otherwise. The removal, therefore, needs to produce a tangible benefit either in social perspective, political stability, reduced hazard or economic terms. One can use the following flow chart as a starting guideline to weigh up the removal with the existing policies at each and every stage of the appraisal.


Revise existing policies Relevant to removal of large dams
Dam Impact assessment
Environmental, Social, Economic, Engineering, etc.
Assesment Report
Technical solutions, options, etc.
Removing technique:
Consequences, Cost, Time, etc.
Confirmation of removal
Detail of Removing procedure
Demolishing
Feedback

b. ***[2.0 pts]*** *Here are some of the major factors to be considered in the policy analysis*

15. *Does the dam negatively affect the welfare of the society?*
16. *Does the dam created a political instability or conflict between beneficiaries?*
17. *Is there a new technology that could produce better economic advantage than the existing one?*
18. *Is the land value of the reservoir/dam more productive if allotted for other purposes?*
19. *Is the dam causing environmental problems that outweigh its benefit?*
20. *Will the removal affect the ecological balance of the area?*
21. *Is there any discovery of precious mineral with in the reservoir site?*
22. *Is the dam susceptible to failure due to structural defects or geologic reasons that were not observed during design and construction? etc.*

c. ***[1.0 pt]*** *Among the potential harms due to removal of large dams*

- *Flooding of downstream areas,*
- *Economic lose of investments relying on the dam/reservoir such as irrigation schemes, hydropower, etc.*
- *lose of job for the workers who make their living on activities related to the dam/reservoir,*
- *ecologic disturbance,*
- *Cost of demolishing, etc.*

[1.0 pt] *Among the benefits from removal of large dam*

- *natural water course and flow will be restored,*
- *More land if the reservoir area is suitable for agriculture or urbanization,*
- *Relief of the risk that led to the removal of the dam, (eg. Failure, disease, political or social crisis, etc.),*
- *Better utilization using state of the art technology could be possible,*
- *Lesson for future water resource development projects, etc.*

d. ***[1.5 pts]*** *For the analysis various types of data should be collected*

Socio-economic data

- *Market value of products from the dam (amount of power, irrigation output, etc.)*
- *The impact of the dam on the neighborhood (# of deaths, extent of hazards imposed by the presence of the dam)*
- *Need of resettlement if there are areas prone to flooding, etc.*

Technical data

- *The total cost of dam construction, dimensions, unique features and the size of the dam and reservoir,*
- *Hydrologic data,*
- *Capacity of the valley/channel,*
- *Maximum discharge to be released,*
- *Technical/technological capacity for demolishing,*
- *Appropriate time for demolishing and schedule,*

- *Cost of demolishing and material/equipment requirement,*
- *Manpower for demolishing and many more.*

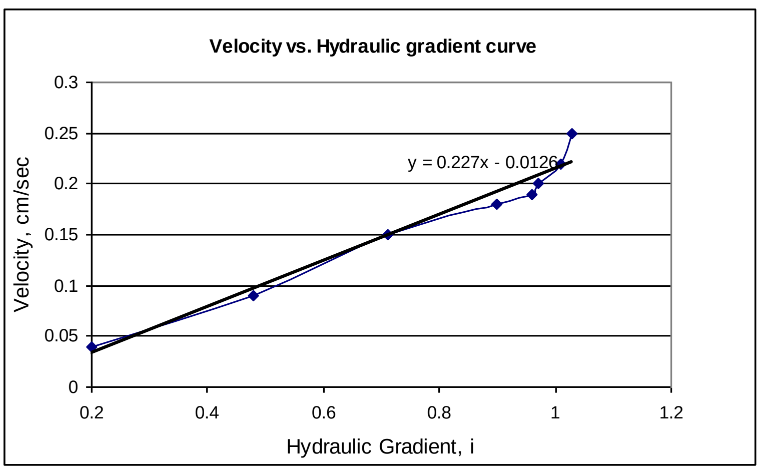
13. A 1040 gm of undisturbed dry foundation soil sample was taken from a proposed dam site and put in a constant head permeameter for a test. The sample occupied a height of 14.5 cm in a 7.6 cm diameter cylinder. Flow measurements were made at a number of different heads until piping commenced. From the readings given below plot the velocity of flow against hydraulic gradient and determine the coefficient of permeability of the soil. Also compare the observed and theoretical values of the hydraulic gradient for piping and discuss their application in the investigation of the foundation for the dam. Specific gravity of Soil grains is 2.7.

Head, cm	2.9	6.9	10.3	13.1	13.9	14.1	14.6	14.9
Volume, cc	176	191	184	187	178	181	192	175
Time, sec	93.8	45	27.8	22.6	20.2	20.2	19.6	15.2

Solution

[1.0 pt]

head cm	*Volume CC*	*Time sec*	*Discharge cc/sec*	*i*	*velocity*	*k*
2.9	*176*	*93.8*	*1.88*	*0.2*	*0.04*	*0.21*
6.9	*191*	*45*	*4.24*	*0.48*	*0.09*	*0.19*
10.3	*184*	*27.8*	*6.62*	*0.71*	*0.15*	*0.21*
13.1	*187*	*22.6*	*8.27*	*0.9*	*0.18*	*0.2*
13.9	*178*	*20.2*	*8.81*	*0.96*	*0.19*	*0.2*
14.1	*181*	*20.2*	*8.96*	*0.97*	*0.2*	*0.2*
14.6	*192*	*19.6*	*9.8*	*1.01*	*0.22*	*0.21*
14.9	*175*	*15.2*	*11.51*	*1.03*	*0.25*	*0.25*



[2.0 pts]

From the graph the hydraulic conductivity of the foundation soil is the slope of the fitted trend line, 0.23cm/sec **[1.0 pt]**

The observed hydraulic gradient for piping is 1.03 **[1.0 pt]**

The theoretical hydraulic gradient is

$$e = \frac{Vv}{V_s} = \frac{0.25 \times \pi \times 7.6^2 \times 14.5 - 1040 \ / 2.7}{1040 \ / 2.7} = \frac{272.6}{385.2} = 0.7$$

$$i = \frac{G-1}{1+e} = \frac{2.7-1}{1+0.7} = 1.0$$ **[2.0 pts]**

Discussion of applications of the above gradients ***[0.5 pt]***

14. Design a non-overflow concrete gravity dam by the single-step method using the following data. Allowable compressive strength of foundation rock, $\sigma_{all.,rock}$ = 4.0 MPa.

Item	Value	Item	Value
H_{max} (depth of headwater)	60 m	*f* (friction Coefficient)	0.75
h_e (spillway crest to MWL)	3 m	s_a (Ultimate Shear strength)	560 kPa
Tail water	None	s_{sf} (Shear safety factor)	5
γ_c (concrete Specific unit weight)	24 kN/m3	ζ (Uplift factor)	0.5
γ_w(water specific unit weight)	10 kN/m3	C (uplift area factor)	1
Earthquake	Ignore	σ_c' (concrete ultimate strength)	30 MPa
silt pressure	Ignore	F (Fetch length)	6.4 km
		V (Wind Velocity)	128 km/hr

Solution

Determine the wave height by the empirical equations ***[1.0 pt]***

$$h_w = 0.763 + 0.032\sqrt{vf} - 0.271 f^{1/4} \quad ;\text{for } f < 32\,km$$

$$h_w = 0.763 + 0.032\sqrt{128 * 6.4} - 0.271 * 6.4^{1/4}$$

$$= 1.25\text{m}$$

Rise of water wave $= 1.33h_w = 1.66$ *m;*

With an allowance of 0.14 m, free board = 1.8m

$$F_{wa} = 2.0\gamma_w h^2_w \quad = 2.0 * 10 * 1.25^2 = 31.25\ kN/m$$

*Point of application = 3/8 *1.25 =0.47m above still water level.*

The dam designed by single step method has a straight downstream face. When extended it intersects upstream face at the headwater surface. The recommended dimensions are: ***[1.0 pt]***

L = 10-15% of h_1

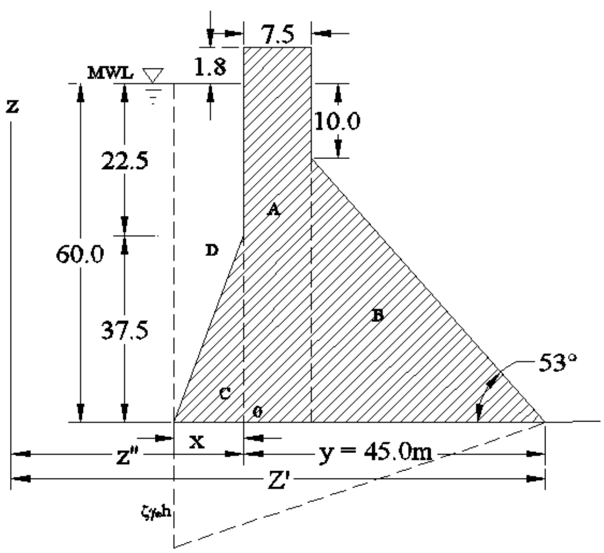
$\Rightarrow$ *L = 6.0 – 9.0m ;*

Take L = 7.5m considering accessibility.

H_{10} *= 3L (when earthquake is not considered)*

= 3 x 7.5m = 22.5m

H_6 *= 1.33L = 1.33 x 7.5 = 10.0m*



In designing (analyzing) a dam in the single step method, the dam is considered as a single block; and dam dimensions are determined in such a way that rules of Zone IV [in multiple step method] are satisfied. The detail calculation of the forces and analysis of stability is shown in the Table below ***[2.5 pts]***

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line	Item	Description & dimension	Forces: Horizontal	Forces: Vertical	Lever	Moment	Remark
1	W_A	61.8*7.5*24		11124	8.75	97335	
2	W_B	0.5*50*37.5*24		22500	25	562500	
		Total Partial empty		**33624**	**[19.62]**	**659835**	
		Estimation of X	Z'	50			
		2B/3 = Z' – 32.97 =	30.38	B=	45.57		
		X = B – Y =	0.57m	; Provide nominal X = 0.80m			
		Therefore,	Z" =	4.20	**B=**	**45.80**	
		Z"+B/3 =	19.47,	Z"+2B/3 =	34.73		
3	W_C	0.5*0.8*37.5*24		360	4.73	1702.8	
		Reservoir Empty		**33984**	**[19.47]**	**661537.8**	**OK!**
4	W_D	Water column		330	4.6	1518	
5	W_u	Uplift: 0.5*B*0.5h*10		-6835.5	19.39	-132540	
6	F_h	Water Pressure	18000		20	360000	
7	F_{wa}	Wave action	31.25		61.8	1931.25	
		Reservoir Full	**18031.25**	**27478.5**	**[32.48]**	**892446.7**	**OK!**

Check for sliding $\frac{\Sigma H}{\Sigma V} = \tan\theta = \frac{18031.25}{27478.5} = 0.66 < 0.75$ *Friction alone is sufficient!* ***[1.0 pt]***

Check Stresses for Reservoir empty ***[1.0 pt]***

$$P_V^{'}, P_V^{''} = \frac{\Sigma W}{B}\left(1 \mp \frac{6e}{B}\right) = \frac{33984}{45.8}\left(1 \mp \frac{6*7.5}{45.80}\right) = 7.36, 1484.14$$

$$P_V^{''} = 1484.14$$

$$P_i^{''} = p_v^{''}(1+\tan^2\phi^{'}) = 1484.14(1+0.021^2) = 1484.82\,kPa$$

$$P_V^{''}, P_i^{'} < \sigma_{rock,all} = 4000\text{kPa} \qquad \text{(safe!)}$$

Check Stresses for Reservoir full ***[1.0 pt]***

$$P_V^{'}, P_V^{''} = \frac{\Sigma W}{B}\left(1 \pm \frac{6e}{B}\right) = \frac{27478.5}{45.80}\left(1 \pm \frac{6*5.5}{45.80}\right) = 1039.66, 166.33$$

$$P_i^{'} = p_v^{'}(1+\tan^2\phi^{'}) = 1039.66(1+0.753^2) = 1629.16\,kPa$$

$$P_V^{''}, P_i^{'} < \sigma_{rock,all} = 4000\ kPa \qquad \text{(safe!)}$$

KISAMA AFRICA UNIVERSITY COLLEGE
DEPARTMENT OF CIVIL ENGINEEREING

Ac. Year 2005/2006
Mid-Semester Exam **Date April 8, 2013**
Hydraulic Structures I (CEng 408) **Time allowed 2 ½ hrs.**

Instruction: 1. Each question has equal marks[Total 30%]
2. Make valid assumptions whenever necessary
3. The examination is OPEN book.

1. a. Explain with illustrative sketch how you determine the capacity of a reservoir to be used for mitigation of water shortage in an area suffered from successive drought .

b. Discuss at least four advantages and disadvantages of constructing a large dam.

Advantages	Disadvantages
Dependable Water Source	Extensive reservoir area
Reduced downstream flood hazard	Huge capital investment
	Disturbs the Ecosystem

c. List three dams built and operational in Ethiopia and write the primary purpose of the reservoir.

4. *Koka Dam for Hydroelectric power generation, Irrigation and Flood control.*
5. *Gilgel Gibe for Hydroelectric Power Generation.*
6. *Melkawakena for Hydroelectric Power Generation.*

2. With reference to the hydrologic cycle and its component stages discuss how the structural intervention of human being on the natural cycle is essential to adjust the spatial variation and temporal variation of water in a way that brings sustainable development in harmony with the ecosystem and environment. (Take an illustrative hydraulic structure to elaborate your discussion if necessary)

Water is one of the essential prerequisites of life. A country's water resources include all the water in rivers, lakes, seas, and groundwater which are stages of the hydrologic cycle. The distribution of water in nature in space and time, however, is such that it is scarce at some locations and at particular times and excess at other locations (and at another time at same location). Rainfall, which is the main water input to our ecosystem, is variable in space and time. This is then reflected, for instance, in river flow, groundwater and lake-levels. Some areas get more or less uniform and good amount of rainfall most of the year (like areas in Southwest Ethiopia), whereas other places get their rainfall concentrated in few months (the wet season). Still there are places that get very scanty rainfall. On the other hand the society's demand is not in general synchronised with the availability of water. In fact, some needs, such as irrigation water requirements are high during periods of no, or less rainfall. Hence man is faced with the task of developing the available water resources to meet his needs.

With water needs for domestic use and that for food production being the basic requirements, water needs of a society, both in quantity and quality depend upon its level of development. Water is needed for energy production (hydropower), industrial use, recreation, and navigation (waterways), to mention the most common and traditional ones. Thus projects are designed and implemented to meet all or some of these needs.

The very water that is essential for life may also threaten life. Floods cause from time to time great losses to human life and property. Thus settlements and developments on banks of rivers should be protected from occurring floods, high flows in streams should not cause damage to bridges, etc., for instance by building dykes, In such cases the water has to be controlled so that its harmful consequences are minimised, if not totally prevented.

Consider building of a storage reservoir as a case

The absence of natural storage of adequate capacities necessitates construction of some artificial storage works. Development of natural storages may also be included in this category sometimes (Cherecherea weir at Lake Tana). In rainy season there is excess flow down the valley in a river. An impounding reservoir(human intervention) can be constructed in the valley to store this excess water which will meet the demand in dry periods.

Ctd….

3. A 1040 gm of undisturbed dry foundation soil sample was taken from a proposed dam site and put in a constant head permeameter for a test. The sample occupied a height of 14.5 cm in a 7.6 cm diameter cylinder. Flow measurements were made at a number of different heads until piping commenced. From the readings given below plot the velocity of flow against hydraulic gradient and determine the coefficient of permeability of the soil. Also compare the observed and theoretical values of the hydraulic gradient for piping and discuss their application in the investigation of the foundation for the dam. Specific gravity of Soil grains is 2.7.

Head, cm	2.9	6.9	10.3	13.1	13.9	14.1	14.6	14.9
Volume, cc	176	191	184	187	178	181	192	175
Time, sec	93.8	45	27.8	22.6	20.2	20.2	19.6	15.2

Solution

[1.0 pt]

head cm	*Volume CC*	*Time sec*	*Discharge cc/sec*	*i*	*velocity*	*k*
2.9	*176*	*93.8*	*1.88*	*0.2*	*0.04*	*0.21*
6.9	*191*	*45*	*4.24*	*0.48*	*0.09*	*0.19*
10.3	*184*	*27.8*	*6.62*	*0.71*	*0.15*	*0.21*
13.1	*187*	*22.6*	*8.27*	*0.9*	*0.18*	*0.2*
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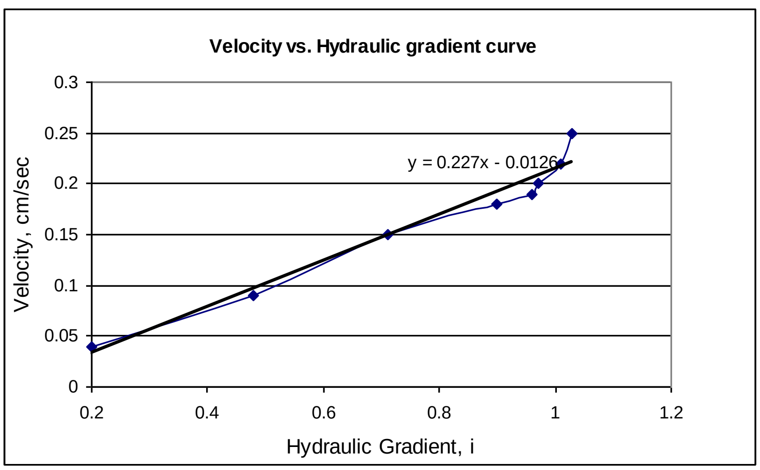
14.9	*175*	*15.2*	*11.51*	*1.03*	*0.25*	*0.25*



[2.0 pts]

From the graph the hydraulic conductivity of the foundation soil is the slope of the fitted trend line, 0.23cm/sec **[1.0 pt]**

The observed hydraulic gradient for piping is 1.03 **[1.0 pt]**

The theoretical hydraulic gradient is

$$e = \frac{Vv}{V_s} = \frac{0.25\times\pi\times7.6^2\times14.5-1040\ \ /2.7}{1040\ \ /2.7} = \frac{272.6}{385.2} = 0.7$$

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Discussion of applications of the above gradients **[0.5 pt]**

3. Design a non-overflow concrete gravity dam by the single-step method using the following data. Allowable compressive strength of foundation rock, $\sigma_{all.,rock}$ = 4.0 MPa.

Item	Value	Item	Value
H_{max} (depth of headwater)	60 m	*f* (friction Coefficient)	0.75
h_e (spillway crest to MWL)	3 m	s_a (Ultimate Shear strength)	560 kPa
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γ_c (concrete Specific unit weight)	24 kN/m^3	ζ (Uplift factor)	0.5
γ_w(water specific unit weight)	10 kN/m^3	C (uplift area factor)	1
Earthquake	Ignore	σ_c' (concrete ultimate strength)	30 MPa
silt pressure	Ignore	F (Fetch length)	6.4 km
		V (Wind Velocity)	128 km/hr

Solution

Determine the wave height by the empirical equations ***[1.0 pt]***

$$h_w = 0.763 + 0.032\sqrt{vf} - 0.271 f^{1/4} \quad ;\text{for } f < 32\,km$$

$$h_w = 0.763 + 0.032\sqrt{128 * 6.4} - 0.271 * 6.4^{1/4}$$

$$= 1.25\text{m}$$

Rise of water wave $= 1.33h_w = 1.66$ *m;*

With an allowance of 0.14 m, free board = 1.8m

$$F_{wa} = 2.0\gamma_w h^2_w \quad = 2.0 * 10 * 1.25^2 = 31.25\ kN/m$$

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The dam designed by single step method has a straight downstream face. When extended it intersects upstream face at the headwater surface. The recommended dimensions are: ***[1.0 pt]***

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$\Rightarrow$ *L = 6.0 – 9.0m ;*

Take L = 7.5m considering accessibility.

H_{10} *= 3L (when earthquake is not considered)*

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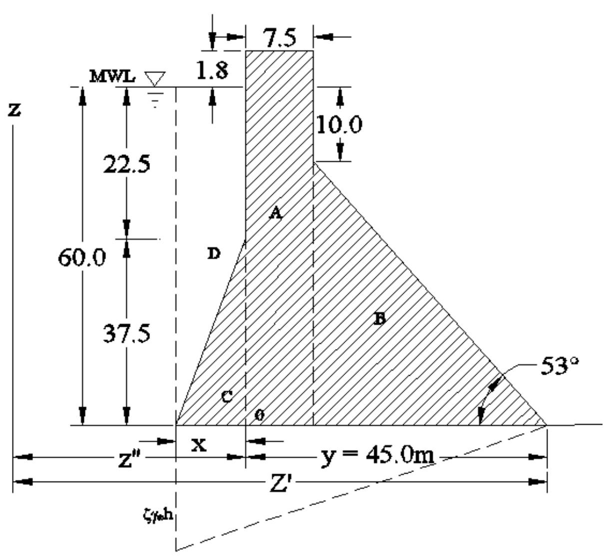

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.

line	Item	Description & dimension	Forces Horizontal	Forces Vertical	Lever	Moment	Remark
1	W_A	61.8*7.5*24		11124	8.75	97335	
2	W_B	0.5*50*37.5*24		22500	25	562500	
		Total Partial empty		**33624**	**[19.62]**	**659835**	
		Estimation of X	Z'	50			
		2B/3 = Z' – 32.97 =	30.38	B=	45.57		
		X = B – Y =	0.57m	; Provide nominal X = 0.80m			
		Therefore,	Z" =	4.20	**B=**	**45.80**	
		Z"+B/3 =	19.47,	Z"+2B/3 =	34.73		
3	W_C	0.5*0.8*37.5*24		360	4.73	1702.8	
		Reservoir Empty		**33984**	**[19.47]**	**661537.8**	**OK!**
4	W_D	Water column		330	4.6	1518	
5	W_u	Uplift: 0.5*B*0.5h*10		-6835.5	19.39	-132540	
6	F_h	Water Pressure	18000		20	360000	
7	F_{wa}	Wave action	31.25		61.8	1931.25	
		Reservoir Full	**18031.25**	**27478.5**	**[32.48]**	**892446.7**	**OK!**

Check for sliding $\frac{\Sigma H}{\Sigma V} = \tan\theta = \frac{18031.25}{27478.5} = 0.66 < 0.75$ *Friction alone is sufficient!* ***[1.0 pt]***

Check Stresses for Reservoir empty ***[1.0 pt]***

$$P_V^{'}, P_V^{''} = \frac{\Sigma W}{B}\left(1 \mp \frac{6e}{B}\right) = \frac{33984}{45.8}\left(1 \mp \frac{6*7.5}{45.80}\right) = 7.36, 1484.14$$

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$$P_V^{''}, P_i^{'} < \sigma_{rock,all} = 4000\text{kPa} \qquad \text{(safe!)}$$

Check Stresses for Reservoir full ***[1.0 pt]***

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Appendix 1 Complete list of Greek Letters

Symbol		Name
Capital	**Small**	
Α	α	Alpha
Β	β	Beta
Γ	γ	Gamma
Δ	δ	Delta
Ε	ε	Epsilon
Ζ	ζ	Zeta
Η	η	Eta
Θ	θ	Theta
Ι	ι	Iota
Κ	κ	Kappa
Λ	λ	Lambda
Μ	μ	Mu
Ν	ν	Nu
Ξ	ξ	Xi
Ο	ο	Omicron
Π	π	Pi
Ρ	ρ	Rho
Σ	σ	Sigma
Τ	τ	Tau
Υ	υ	Upsilon
Φ	φ	Phi
Χ	χ	Chi
Ψ	ψ	Psi
Ω	ω	Omega