PHILIPPINE NATIONAL STANDARD

PNS/BAFS/PAES 225:2017
ICS 65.060.35

Rainwater and Runoff Management – Small Water Impounding System

DEPARTMENT OF AGRICULTURE
1898
DEPARTMENT OF
AGRICULTURE
PHILIPPINES

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Foreword

The formulation of this national standard was initiated by the Agricultural Machinery Testing and Evaluation Center (AMTEC) under the project entitled "Enhancement of Nutrient and Water Use Efficiency Through Standardization of Engineering Support Systems for Precision Farming" funded by the Philippine Council for Agriculture, Aquaculture and Forestry and Natural Resources Research and Development - Department of Science and Technology (PCAARRD - DOST).

As provided by the Republic Act 10601 also known as the Agricultural and Fisheries Mechanization Law (AFMech Law of 2013), the Bureau of Agriculture and Fisheries Standards (BAFS) is mandated to develop standard specifications and test procedures for agricultural and fisheries machinery and equipment. Consistent with its standards development process, BAFS has endorsed this standard for the approval of the DA Secretary through the Bureau of Agricultural and Fisheries Engineering (BAFE) and to the Bureau of Philippine Standards (BPS) for appropriate numbering and inclusion to the Philippine National Standard (PNS) repository.

This standard has been technically prepared in accordance with BPS Directives Part 3:2003 – Rules for the Structure and Drafting of International Standards.

The word "shall" is used to indicate mandatory requirements to conform to the standard.

The word "should" is used to indicate that among several possibilities one is recommended as particularly suitable without mentioning or excluding others.

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ANNEXES

1 Scope

This standard specifies the minimum design requirements of a small water impounding system.

A small water impounding system shall be defined as an earth fill structure built across a narrow depression or valley to harvest and store rainfall and runoff for immediate and multiple use. It has a height of 5 m to a maximum of 15 m and service area of 25 ha to 150 ha.

2 References

The following normative documents contain provisions through which reference in this text constitute provisions in this National Standard:

PNS/BAFS/PAES 217:2017	Determination of Irrigation Water Requirements
PNS/BAFS/PAES 218:2017	Open Channels – Design of Main Canals, Laterals and Farm Ditches
PNS/BAFS/PAES 221:2017	Design of Canal Structures – Road Crossing, Drop, Siphon and Elevated Flume

3 Definition

For the purpose of this standard, the following definitions shall apply:

3.1
active storage
volume of water stored in reservoir between the minimum water level and normal water level (see Figure 4a)

3.2
dam
any barrier constructed to store water

3.3
dam height
vertical distance from lowest point of the ground line to the dam crest

3.4
dead storage
volume below the intake structure computed as

$$V = 20900 \times A^{0.687}$$

where:

A is the drainage area, (km^2)
sediment volume based on 25 years of accumulation in the reservoir (see Figure 4a)

3.5
filter drain
dam component which prevents migration of small particles and screen off fine materials that flow with seepage water and prevent piping (Figure 1)

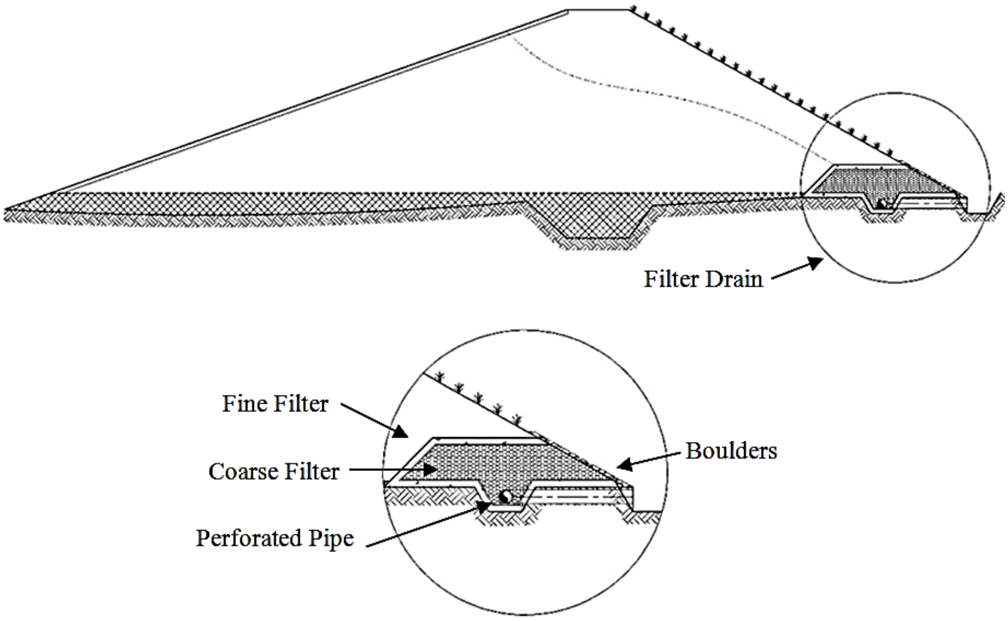

Figure 1. Filter Drain

3.6
homogeneous embankment
dam composed of a single kind of embankment material exclusive for slope protection

3.7
Karst topography
geological formation shaped by the dissolution of a layer or layers of soluble bedrock, usually carbonate rocks such as limestone or dolomite

3.8
natural spillway
spillway which is not excavated such as natural draw, saddle or drainage way

3.9
normal storage elevation
maximum elevation the water surface which can be attained by the dam or reservoir without flow in the spillway (see Figure 4a)

3.10
reservoir
part of the system that impounds the runoff

3.11
seepage line
phreatic line
line with no filter arrangements where seepage occurs

3.12
spillway
channel which releases surplus or flood water which cannot be contained in the active storage space of the reservoir

3.13
storage capacity
total capacity at normal water surface elevation

3.14
structural height
vertical distance measured from the top of the dam down to the bedrock

3.15
upstream face
side of the embankment wetted by the impounded water

3.16
watershed
area which contributes runoff or drains water into the reservoir (see Figure 3)

3.17
water right
privilege granted by the government to use and appropriate water

3.18
well-protected reservoir
reservoir where the upper reaches of the basin is shielded by high mountain barriers

3.19
zoned embankment
dam consisting a central impervious core flanked between zones of more pervious materials

4 Main Components

The main components of a small water impounding system are shown in Figure 2.

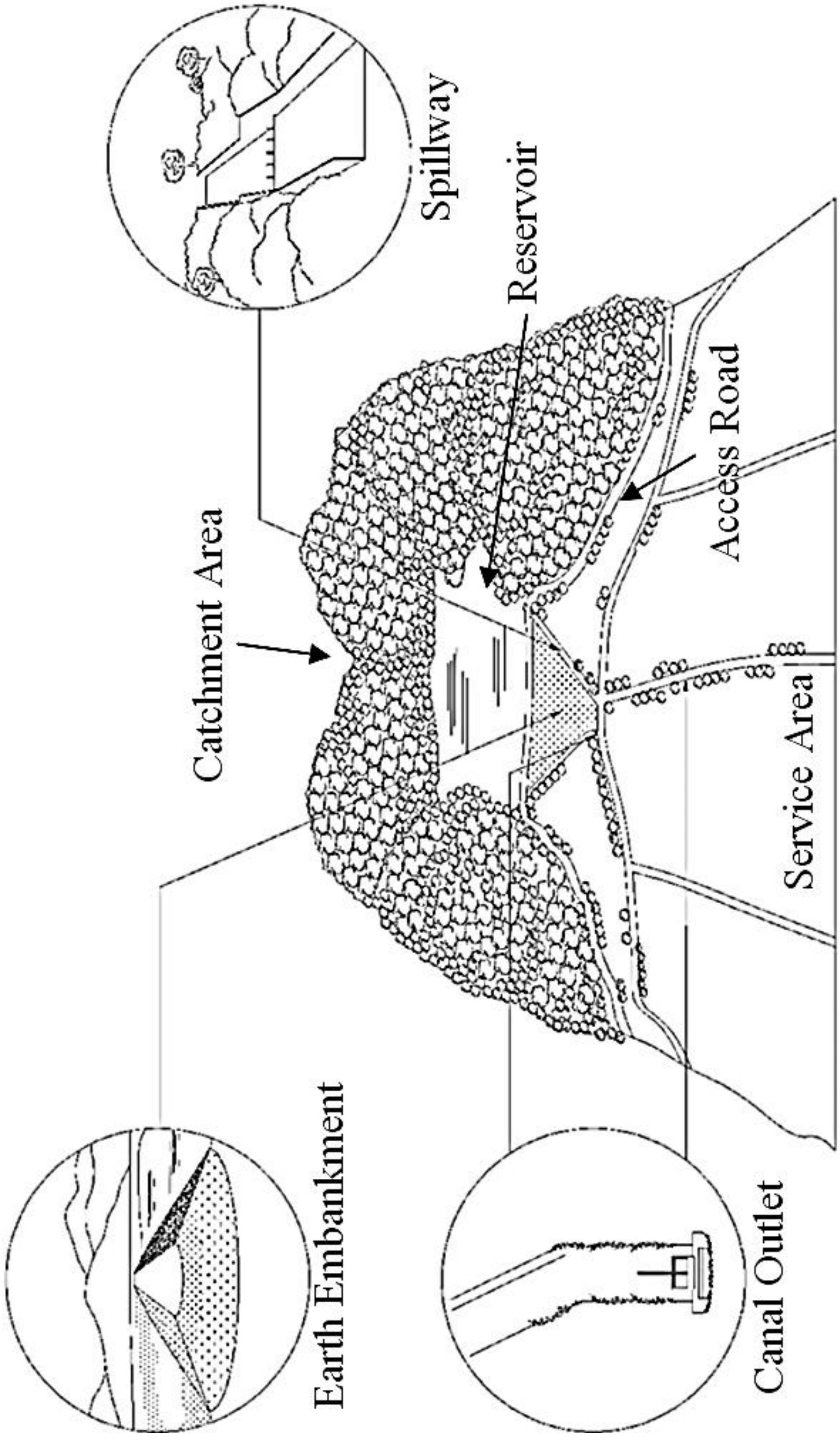

Figure 2. Main Components of a Small Water Impounding System

5 Site Selection

5.1 Physical Condition

The ideal site for the dam should be a natural depression or streambed of which slope is at minimum and then widens out behind some natural constriction as this condition will require the minimum inputs. Figure 3 shows a general map of a SWIS.

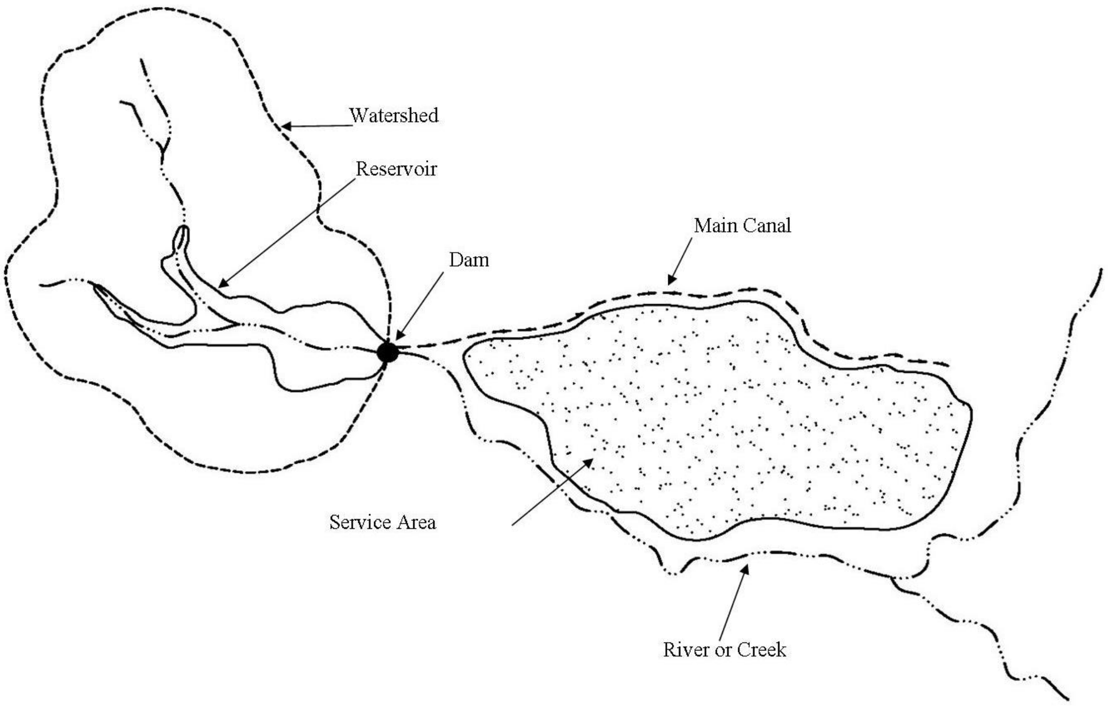

Figure 3. General Map of a Small Water Impounding System
SOURCE: PCAARRD-DOST. 1986. The Philippine Recommends for Small Water Impounding Projects

5.1.1 The earth dam should be located at the narrowest section of the depression or gully such that the width is the shortest possible.

5.1.2 A horseshoe, U-shape or tank dam shall be used in the absence of natural basin.

5.1.3 Seeps, springs, slides and rock outcrops shall be avoided.

5.1.4 Embankment materials such as clay to sandy clay borrows should be available at a reasonable proximity and adequate quantity.

5.1.5 Soil shall be well-graded and shall contain at least 30% clay.

5.1.6 The dam site shall have a sound foundation to ensure stability, preferably rock to avoid excessive leakage.

5.1.7 Spillway shall be located on a consolidated natural formation

5.1.8 Natural spillways should be preferred than cut spillways.

5.1.9 Spillway shall be capable of discharging excess water during flood periods and the outfall must be secured against erosion.

5.1.10 Reservoir inflows shall be dependable and adequate to sustain its purpose.
(see Annex A for the computation of values for dependable and adequate reservoir inflow)

5.1.11 Reservoir shall be at least 1 ha at normal water level.

5.1.12 Land area to be submerged shall be of relatively low value.

5.1.13 Site shall be accessible by vehicle.

5.1.14 Dam and reservoir sites with the following conditions shall be avoided:

- sink holes
- faults (clearance)
- alluvial deposits
- ant hills
- gravelly areas
- shale and limestone (and other water-soluble sedimentary rock) parent materials
- mud deposits
- other formations that will allow excessive leakages or loss of impoundment
- mining areas located above the reservoir
- sites on or near a known active earthquake fault
- Karst topography

5.1.15 The subsurface geology of the site shall be determined through a representative of a subsurface geology of the area not necessarily found in the dam site itself but in some distant location, roadside cut, outcrops and riverbanks.

5.2 Watershed Condition

5.2.1 The watershed shall be able to provide adequate yield for the reservoir.

5.2.2 It shall have good vegetation cover.

5.2.3 It shall have a maximum slope of 18%.

5.3 Service Area

5.3.1 It shall be located as near as possible to the reservoir site.

5.3.2 It shall commensurate to the expected active storage/capacity of the reservoir.

5.3.3 It shall be contiguous and has developed paddies.

5.3.4 It shall have the provision for access road between the dam site and nearest existing road.

5.4 Socio-economic Consideration

The project shall be economically viable and socially acceptable.

6 Preliminary Design Activities

6.1 Soil and Topographic Surveys

6.1.1 The types of soil in the dam sites, reservoir, watershed and service area as well as their extent shall be determined.

6.1.2 The soil formation shall be characterized. Borings should be done along the axis of the dam down to the impervious layer or a depth equal to the proposed height of the dam.

6.1.3 Types and extent of existing land uses in service and watershed areas shall be determined.

6.1.4 A soil survey report which includes the soil and topographic survey with contour interval of 0.5 m and scale of 1:2000 shall be prepared.

6.1.5 The delineation of the boundaries of the contributing watersheds and service areas shall be presented in the topographic map or in a digital elevation model. The proposed location of the dam, access roads and service canals shall also be indicated.

6.1.6 A profile cross-section of the stream/depression at the selected site 20 m upstream and 20 m downstream shall also be included.

6.2 Dam Site Characterization

6.2.1 The depth of overburden that must be removed to reach an acceptable foundation for the dam wall shall be determined through seismic refraction or other geographical method.

6.2.2 The rock types which make up the foundation and the extent of the effects of surface weathering shall be specified.

6.2.3 Engineering properties of the foundation rock types such as strength, deformability and durability shall be specified.

6.2.4 A full description of the geological structure of the foundation and its defects such as jointing, faulting and folding of the rock strata shall be specified. The defect pattern in the rock mass shall also be identified including orientation, spacing, extent or persistence and aperture or openness.

6.3 Agrohydrologic Analyses

Agrohydrologic parameters shall be determined prior to the design of the structures and components involved in the project. There are three major parameters considered:

6.3.1 Run-off and inflow hydrograph

6.3.2 Field water balance

6.3.3 Reservoir inflow

The process of determining each parameter is detailed in Annex A.

7 Design Considerations

7.1 Dam

The dam shall be structurally stable under all conditions and shall be sufficiently water tight. Components and profile of an earth-fill dam is given in Figure 4.

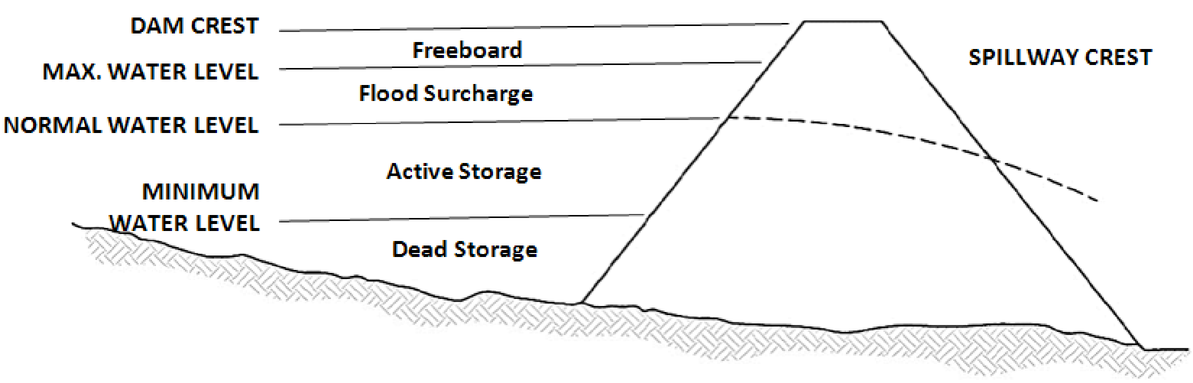

Figure 4a. Vertical Components of an Earth-fill Dam

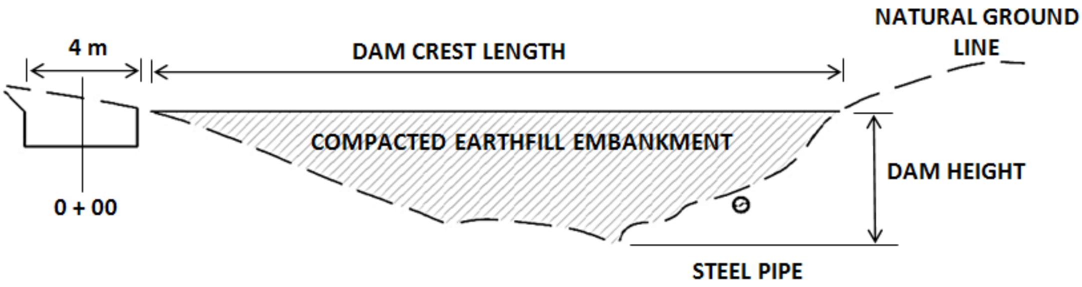

Figure 4b. Profile along Dam Axis

7.1.1 Dam height shall be determined based on the vertical storage requirements: dead storage, active storage, flood surcharge and freeboard as shown in Figure 4a. Section A.6 of Annex A shows an outline for dam height computation.

- Dead Storage - unless amended later, the sediment volume shall be computed based on 25 years of accumulation in the reservoir.

- Active Storage - this shall be determined based on reservoir operation studies (see section A.4 of Annex A)

- Flood Surcharge - this shall be determined by flood routing (see section A.5 of Annex A)

- Freeboard - this shall be computed based on the wave run-up and embankment settlement shown in the formula below

$$Fb_1 = 1.5\left(0.032\sqrt{FV} + 0.763 - 0.271\sqrt[4]{F}\right)$$
$$Fb_2 = 2\%\ to\ 5\%\ of\ dam\ height$$
$$Fb = Fb_1 + Fb_2$$

where:

Fb_1	is the freeboard due to wave run-up (m)
F	is the reservoir effective fetch (km)
V	is the wind velocity (km/h)
Fb_2	is the freeboard due to embankment settlement (m)
Fb	is the total freeboard (m)

7.1.2 The dam crest width shall be computed based on the following criteria. The largest computed dimension shall be adapted as the dam crest width.

7.1.2.1 The minimum width for maintenance purposes is 4 m.

7.1.2.2 $W_1 = 5/3\sqrt{H_1}$ where W_1 = width of the dam crest, m; H_1 = dam height, m

7.1.2.3 $W_2 = \frac{H_2}{5} + 10$ where W_2 = width of the dam crest, ft; H_2 = dam height, ft

7.1.3 The type of suitable dam shall be selected based on the availability and excavation costs of the materials for construction.

7.1.3.1 **Homogeneous/Modified Homogeneous Type -**This type shall be considered if the supply of materials of low permeability such as sandy or silty clay and other clayey material is abundant.

7.1.3.1.1 Composed of a single kind of embankment material exclusive of slope protection

7.1.3.1.2 The embankment material must be sufficiently impervious to provide adequate water barrier

7.1.3.1.3 The slopes must be stable under critical loading condition

7.1.3.1.4 Modified by using carefully placed pervious material which help to control seepage flow and pore pressure development

7.1.3.1.5 Advantages:

- Allows the use of steep slopes by lowering the phreatic level within the embankment
- The flowing of fine particle with the seepage water is screened off which prevents piping

7.1.3.1.6 Provisions for Drainage:

- Rockfill Toe - a filter shall be constructed between the embankment proper and the rockfill toe.
- Horizontal Blanket - shall extend from the downstream toe deep into the embankment to a distance not greater than 1/3 of the base of the dam but not so far upstream as to shorten the seepage path too much to critical extent.
- Chimney Drain - shall be provided for better collection capability

7.1.3.2 Zoned Type - This type shall be considered if sufficient quantities of both pervious and impervious materials are available.

7.1.3.2.1 Components:

- Core - central impervious portion
- Transition zone – filter
- Upstream shell – made of more pervious material which provides stability against rapid drawdown

- Downstream shell – made of more pervious material which serves asdrain to control seepage

7.1.3.2.2 Advantages:
- Steeper slopes could be adopted with consequent reduction in total volume of embankment materials.
- A wide variety of materials could be utilized hence utilization of materials excavated structure could be maximized.
- Better stability against earthquake, cracking and settlement could be provided.

7.1.4 The embankment slopes shall be stable against the most adverse conditions which they can be subjected.

- **Upstream Face –** The slope shall be checked against drawdown condition. The stable slope for the upstream face is 2.75:1.

- **Downstream Face–** The slope shall be checked against steady seepage with full reservoir capacity. The stable slope for the upstream face is 2.5:1.

Other recommended values of slopes for homogeneous and zoned dams are presented in Table 1 and Table 2.

Table 1. Recommended Slopes for Small Homogeneous Earth Fill Dams on Stable Foundation

Case	Purpose	Drawdown Condition	Embankment Classification[2]	Slopes	
				Upstream	Downstream
Homogeneous or Modified Homogeneous	Detention or Storage	Slow	GC,GM,SC	2.5:1	2:1
			SM,CL,ML,CH	3:1	2.5:1
			MH	3.5:1	2.5:1
Modified Homogeneous	Storage	Rapid[1]	GC,GM,SC	3:1	2:1
			SM,CL,ML,CH	3.5:1	2.5:1
			MH	4:1	2.5:1

NOTE From the "Design of Small Dams", US Bureau of Reclamation

[1]Drawdown rates of 0.15 m/day following prolonged storage at high level

[2] Definitions:
GC – Clayey, poorly graded gravel-sand-clay mixture
GM – Silty gravel, poorly graded sand-silt mixture
SC – Clayey sand, poorly graded sand-silt mixture
SM – Silty sand, poorly graded sand-silt mixture
CL – Inorganic clay of low to medium plasticity, gravely clay, sandy clay, silty clay, lean
ML – Inorganic silt and very fine sand, rock flour, silty or clayey fine sand with slight plasticity
CH – Inorganic clay of high plasticity, fat clay
MH – Inorganic silt, micaceous or diatomaceous fine sandy or silty soil, elastic silt

Table 2. Recommended Slopes for Small Zoned Earth Fill Dams on Stable Foundation

Case	Purpose	Drawdown Condition	Shell Material	Core Material	Slopes	
					Upstream	Downstream
Zoned with minimum core[1]	Any	Not Critical	Rockfill	GC,GM	2:1	2:1
			GW,SP	SM,CL		
			SW or SP	ML,CH,MH		
Zoned with maximum core[1]	Detention or Storage	Slow	Rockfill	GC,GM	2:1	2:1
			GW,SP	SC,SM	2.25:1	2.25:1
			SW or SP	CL,ML	2.5:1	2.5:1
				CH,MH	3:1	3:1
	Storage	Rapid[2]	Rockfill	GC,GM	2.5:1	2:1
			GW,GP	SC,SM	2.5:1	2.25:1
			SW or SP	CL,ML	3:1	2.5:1
				CH,MH	3.5:1	3:1
NOTE From the "Design of Small Dams", US Bureau of Reclamation						
[1]Minimum and maximum size of cores are shown in Figure 5.						
[2] Drawdown rates of 0.15 m/day following prolonged storage at high level						
[3] Definitions: CW – Well-graded, gravel-sand mixture, little or no fine GP – Poorly gravel, gravel-sand mixture, little or no fine SW – Well graded sand, gravely sand, little or no fine SP – Poorly graded sand, gravely sand, little or no fine						

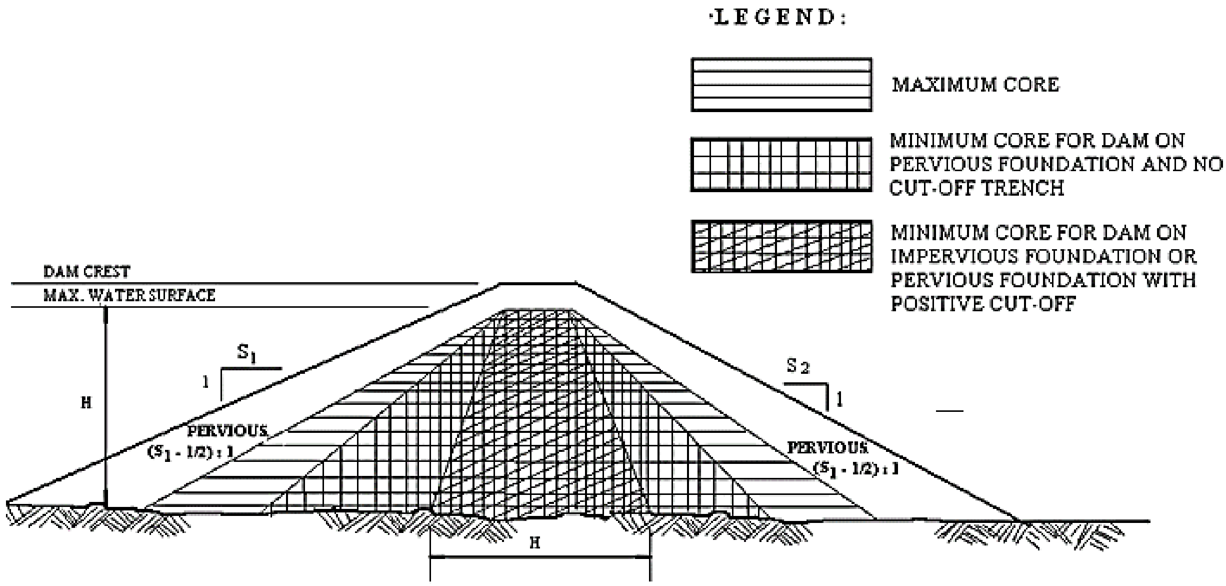

Figure 5. Size of Impervious Core for Zoned Dam

7.1.5 The embankment slopes, upstream and downstream shall be protected against wave action and erosion, respectively. Sizing of materials for embankment slope protection is detailed in Annex B.

7.1.5.1 If rock or stone or riprap is available near the site, these materials shall be the priority of use for the upstream face. Boulder riprap underlain with gravel blanket or dumped gravel shall be used. The minimum thickness shall be 20 cm.

7.1.5.2 The downstream face shall be protected by using grass sodding.

7.1.5.3 For a well-protected reservoir, plain gravel shall be used to protect the upstream face of the dam.

7.1.5.4 For unprotected reservoir, rock shall be used to protect the upstream face. Concrete pavement shall be used only in extreme cases.

7.1.5.5 Free flow of water from the upstream face shall be prevented. Embankment shall be compacted by layers of 20 cm with proctor density of 95%. Sections along outlets, conduits and joints with concrete sections shall be compacted thoroughly

7.1.5.6 For zoned dams where the downstream outer shell consists of rock or cobbles, no special treatment of the slope shall be used.

7.1.5.7 For homogeneous/modified homogeneous dams, a layer of cobbles, sodding or interceptor canals shall be used to protect the downstream slope.

7.1.5.8 A gutter made of grouted rock or cobbles shall be provided to control the development of unsightly gullies at the contact between the embankment and the abutments.

7.1.5.9 The foundation shear stress shall be smaller than the shear strength to provide a suitable margin of safety. Foundation with silt or quicksand shall not be used.

7.1.6 The seepage line shall be well within the downstream face of the dam. The downstream face of the dam shall be provided with rock toe drain. The height of which depends on the height of water at normal water level. The rock toe has usually a height of 1/3 the hydraulic head.

7.1.7 Core trench along the centerline of dam axis shall be provided to cut off seepage across the foundation.

7.1.8 To prevent the migration of small particles and to screen off fine materials that flow with seepage water through the embankment, the filter shall satisfy the following requirements:

- The graduation shall be able to prevent the soil particles from entering the filter and clogging it.
- The capacity of the filter shall adequately handle total seepage flow.
- The filter shall be permeable enough to provide easy access of seepage water to reduce the uplift forces.

7.1.9 Multi-layer of filters although more effective must be avoided in general since these are costly. If sufficient quantities of filter material are available at reasonable cost, it would be more economical to provide thick layers rather than process material to meet exact requirements for a thin filter design.

Determination of the thickness of the filter drain is presented in section B.2 of Annex B.

7.2 Spillway

7.2.1 It shall be hydraulically and structurally adequate and shall provide sufficient capacity. A spillway diagram is shown in Figure 6.

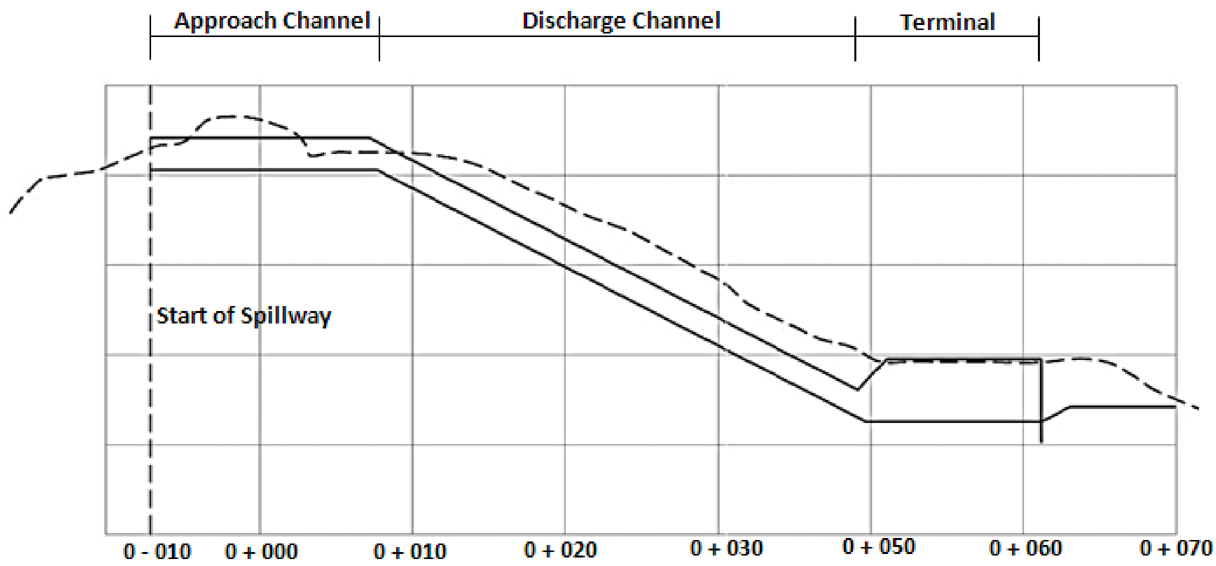

Figure 6. Spillway Diagram

7.2.2 The width of spillway shall be determined from the result of flood routing by allowing a maximum surcharge height of 1 m. Spillway hydraulics calculation is detailed in Annex C.

7.2.3 It shall be located such that the spillway discharge will not have the chance to erode or undermine the downstream toe of the dam.

7.2.4 The bounding surfaces at critical sections shall be protected with concrete lining or erosion resistant material.

7.2.5 Type of spillway

If geologic conditions will allow, side channel or chute type spillway shall be constructed. Unless excavation is excessive or too difficult, a chute spillway can be made to pass on the saddle on either left or right side of the proposed dam wherein the alignment may lead to an adjacent drainage way or to the same stream below the dam.

7.2.6 Control Section

7.2.6.1 The control section of a side channel spillway may consist of a concrete ogee weir or sill.

7.2.6.2 The use of a flat approach control for a chute spillway should be prioritized for economy, simplicity and ease of construction. The flat approach may be lined or not depending on the approach velocity or structural requirements. Other than the flat approach, the control section may consist of an ogee or sharp-crested weir.

7.2.7 Discharge Channel

7.2.7.1 The discharge channel shall have a single straight slope for hydraulic efficiency and structural stability. The slope shall be approximately equal to the general slope of the existing ground.

7.2.7.2 The cross-sectional shape of the channel may be trapezoidal, rectangular or combination of both.

7.2.7.3 The channel can be lined or unlined depending on the channel velocity.

7.2.7.4 In designing unlined channels, Table 3 showing the permissible velocities for cohesive soils shall be used.

7.2.7.5 For lined channels, linings can be of concrete, riprap or grass. Table 4 shows the permissible velocities for different types of grass. This table shall be used only as basis of comparison with similar types of grass found locally.

Table 3. Maximum Permissible Velocities for Cohesive Soils

Material	n	Clear Water V (ft/s)	Unit Tractive Force	Water Transporting Colloidal Silts	
				V (ft/s)	Unit Tractive Force
Fine sand, colloidal	0.020	1.50	0.027	2.50	0.075
Sandy loam, non-colloidal	0.020	1.75	0.037	2.50	0.075
Silt loam, non-colloidal	0.020	2.00	0.048	3.00	0.11
Alluvial silt, non-colloidal	0.020	2.00	0.048	3.50	1.15
Ordinary firm loam	0.020	2.50	0.075	3.50	0.15
Volcanic ash	0.020	2.50	0.075	3.50	0.15
Stiff clay, very colloidal	0.025	3.75	0.26	5.00	0.46
Alluvial silts, colloidal	0.025	3.75	0.26	5.00	0.46
Shales& hard pans	0.025	6.00	0.67	6.00	0.67
Fine gravel	0.020	2.50	0.075	5.00	0.32
Graded loam to cobbles when non-colloidal	0.030	3.75	0.38	5.00	0.66
Graded silts to cobbles	0.030	4.00	0.43	5.50	0.80
when colloidal					
Coarse gravel, non-	0.025	4.00	0.30	6.00	0.67
colloidal					
Cobble and shingle	0.035	5.00	0.91	5.50	1.10

Table 4. Maximum Permissible Velocities for Grassed Channels

Grass Cover	Slope Range (%)	Permissible Velocity	
		Erosion Resistant Soils	Easily Eroded Soil
1. Bermuda grass	up to 5	8	6
	5 to 10	7	5
	over 10	6	4
2. Buffalo grass	up to 5	7	5
Kentucky blue grass	5 to 10	6	4
Smooth Brome	over 10	5	3
Blue Grama			
3. Lespedeza sericea			
Weeping love grass		3.5	2.5
Yellow bluestem			
Kudzu	up to 5[1]		
Alfalfa			
Crabgrass			
4. Common Lespedeza[2]		3.5	2.5
Sudan Grass[2]	up to 5[1]		
NOTE 1 From US Conservation Service. NOTE 2 Values apply to average uniform stands of cover. NOTE 3 Velocities exceeding 5 fps are to be used only where good covers and proper maintenance can be obtained.			
[1]Not to be used on slopes steeper than 5% [2]Used on mild slopes or as temporary cover until permanent covers are established			

7.2.7.6 The flow depth along the discharge channel shall be determined based on the Manning's formula.

7.2.7.7 Freeboard channel shall be computed based on the average depth of flow within the reach.

7.2.7.8 The terminal structure of unlined and grassed channels shall be as simple as possible. It may consist of a concrete sill at downstream end of the channel and a dumped riprap from the sill to a distance downstream equal to the channel width.

7.2.7.9 The terminal structure of riprapped and concrete lined-channels may consist of an unsubmerged deflector bucket or a hydraulic jump type basin.

7.2.7.10 Formulas used and computation details related to the spillway are presented in Annex C.

7.3 Outlet Works

7.3.1 The outlet works shall be able to regulate the release of water that may be dictated by the downstream requirements.

7.3.2 It is recommended to adapt a canal outlet with closed conduit waterway flowing under pressure and gated at the downstream end. This system consists of the following:

- a concrete intake provided with trashrack for protection against debris
- steel pressure pipe waterway
- outlet with gate valve and energy dissipator

The physical arrangement is shown in Figure 7 and the methodology of design is presented in Annex D.

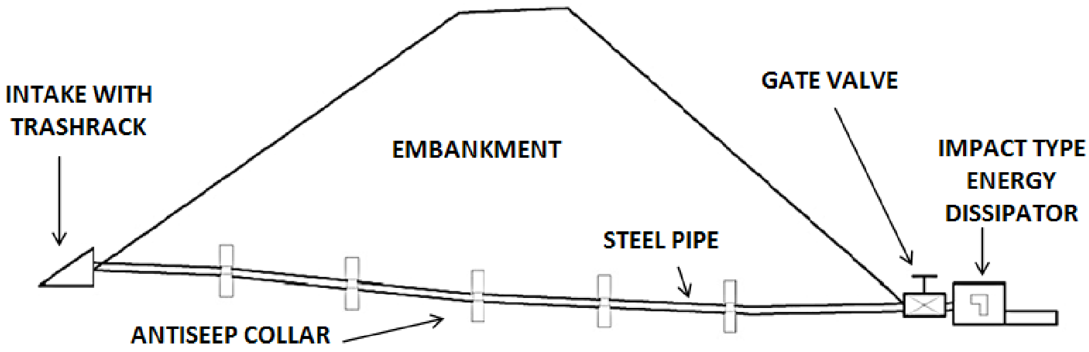

Figure 7. Physical Arrangement of Outlet Works

7.3.3 The provisions in this section shall be applicable to the above specified outlet works system.

7.3.4 The design discharge-head combination shall be based on the large discharge under low head. This combination shall be obtained from the reservoir operation studies.

7.3.5 The size of the discharge pipe shall be computed based on a full conduit flow. Details of computation are shown in Annex D.

7.3.6 The size of the impact type dissipator shall be determined as outlined in Annex D.

7.3.7 A minimum of temperature reinforcement for the concrete structural components shall be required.

7.3.8 Joints of the steel discharge pipe shall be water tight. This can be achieved by using couplings that remain water tight after movement or settlement of the pipe.

7.3.9 Methods of bedding and backfilling shall prevent unequal settlement along the pipe length and shall secure the most possible distribution of load on the foundation.

7.3.10 Tight contact between the fill and the conduit surface shall be secured.

7.4 Irrigation Works

Provisions and design procedures for irrigation works are detailed in PNS/BAFS/PAES 218: 2017 – Open Channels – Design of Main Canals, Laterals and Farm Ditches and PNS/BAFS/PAES 221: 2017 – Design of Canal Structures – Road Crossing, Drop, Siphon and Elevated Flume

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ANNEX A
(informative)

Agrohydrologic Studies and Analyses

A.1 Estimation of Runoff and Derivation of Inflow Hydrograph

Required Data:

- Drainage area
- Mainstream length from outlet to highest ridge
- Mainstream outlet to point nearest basin centroid
- Elevation difference
- Watershed Gradient
- Soil Type
- Land Cover
- Land Use

A.1.1 Determine the parameters for a synthetic unit hydrograph using an appropriate method. In this annex, Snyder's method is used.

A.1.1.1 Lag Time

$$T_L = C_t \times \left(\frac{LL_c}{\sqrt{Y}}\right)^a$$

where:

T_L	is the lag time (h)
Ct	is the coefficient with values: 1.2 for mountains drainage area 0.72 for foothill drainage area 0.35 for valley drainage area
L	is the mainstream length from outlet to highest ridge (mi)
L_c	is the mainstream length from outlet to the nearest basin centroid (mi)
Y	is the watershed gradient
a	is 0.38

If the standard rainfall duration, ΔD is not equal to $T_L/5.5$, adjust the computed T_L as shown in section A.1.1.2.

A.1.1.2 Adjusted Lag Time

$$Adjusted\ T_L = T_L + \frac{1}{4}\left(\Delta D - \frac{T_L}{5.5}\right)$$

where:

ΔD	is the standard rainfall duration (duration of 1-inch excess rainfall), h

A.1.1.3 Time of Concentration

$$T_c = \frac{T_L}{0.70}$$

where:

T_c	is the time of concentration (h)
T_L	is the lag time (h)

A.1.1.4 Time to Peak

$$T_p = \frac{1}{2}\Delta D + T_L$$

where:

T_p	is the time to peak (h)
T_L	is the lag time (h)
ΔD	is the standard rainfall duration (h)
	suggested values of ΔD: 0.5 hr or 0.4 hr where $T_c < 3$
	1 hr where $3 < T_c < 6$
	1/5 T_c where $T_c > 6$

A.1.1.5 Peak Rate of Runoff

$$q_p = \frac{0.20A}{T_L}$$

where:

q_p	is the peak rate of runoff (cm/mm)
A	is the drainage area (km^2)
T_L	is the lag time (h)

A.1.2 Compute for rainfall depth for different durations and tabulate as shown in Table A.1.

$$P = iD$$

$$i = \frac{aT^c}{(D+b)^d}$$

where:

P	is the rainfall depth, mm
i	is the computed rainfall intensity using Intensity-Duration-Frequency (IDF) Curve, mm/h
D	is the duration, h
a,b,c,d	is the regression coefficients of the IDF curve for different locations (see Table A.2)
T	is the return period

A.1.3 Rearrange the computed rainfall increments based on 3 maximization patterns. The sequence for peak at different positions are shown in Figure A.1

A.1.3.1 Peak ΔP_1 at middle time position, i= n/2

A.1.3.2 Peak ΔP_1 at 1/3 time position, i= n/3

A.1.3.3 Peak ΔP_1 at 2/3-time position, i= 2n/3+1

Table A.1. Rainfall Depth, Duration and Rainfall Increments

Sequence	Duration, D (h)	Rainfall Intensity, I (mm/hr)	Rainfall Depth, P (mm)	Rainfall Increments, ΔP (mm)
1	$D_1 = \Delta D$	1	P_1	$\Delta P_1 = P_1$
2	$D_2 = 2\Delta D_1$	2	P_2	$\Delta P_2 = P_2 - P_1$
3	$D_3 = 3\Delta D_1$	3	P_3	$\Delta P_3 = P_3 - P_2$
n	$D_n = nD_1$	n	P_n	$\Delta P_n = P_n - P_{n-1}$

Table A.2. Regression Coefficients of the Rainfall Intensity-Duration-Frequency Curve for Different Locations in the Philippines

Region	Station/Location	a	b	c	d	R
1	Vigan, Ilocos Sur	47.295	0.20	0.2710	0.577	0.9882
	Baguio City	51.414	-	0.2337	0.343	0.9800
	Laoag City	60.676	0.30	0.2370	0.554	0.9944
2	Tuguegarao, Cagayan	47.263	0.40	0.2290	0.598	0.9949
	Aparri, Cagayan	53.503	0.20	0.2780	0.610	0.9916
3	San Agustin, Arayat,	-	-	-	-	-
	Pampanga	48.749	0.40	0.2330	0.690	0.9973
	Sta. Cruz, Pampanga	41.687	0.85	0.2220	0.611	0.9976
	Dagupan, Pangasinan	53.665	0.10	0.1340	0.575	0.9959
	Matalava, Lingayen	0.890	0.10	0.2220	0.611	0.9973
	Iba, Zamabales	51.960	0.80	0.2020	0.448	0.9951
	Cabanatuan City	62.961	0.20	0.1395	0.754	0.9950
	Cansinala, Apalit,	-	-	-	-	-
	Pampanga	36.597	-	0.2280	0.568	0.9962
	Gabaldon, Nueva Ecija	43.209	0.10	0.2150	0.487	0.9942
4	Infanta, Quezon	67.327	0.30	0.2010	0.617	0.9867
	Calapan, Mondoro Or.	54.846	0.30	0.2460	0.768	0.9969
	MIA	46.863	0.10	0.1940	0.609	0.9979
	Pot Area, Manila	58.798	0.20	0.1980	0.679	0.9981
	Tayabas, Quezon	39.710	-	0.1320	0.461	0.9912
	Casiguran, Quezon	77.587	0.70	0.2380	0.717	0.9849
	Alabat, Quezon	55.424	0.20	0.2310	0.491	0.9880
	Ambalong, Tanauan,	-	-	-	-	
	Batangas	41.351	-	0.2310	0.511	0.9620
	Angono, Rizal	62.314	0.70	0.1910	0.630	0.9934
5	Daet, Camarines Norte	44.553	-	0.2240	0.570	0.9971
	Legaspi, City	55.836	0.20	0.2480	0.591	0.9958

Table A.2 continued…

	Virac, Catanduanes	49.052	0.20	0.2480	0.591	0.9958
6	Iloilo City	44.390	0.15	0.2040	0.670	0.9970
7	Cebu Airport	59.330	0.40	0.2400	0.812	0.9956
	Dumaguete City	100.821	1.00	0.2370	1.057	0.9963
8	Borongan, Eastern	-	-	-	-	-
	Samar	51.622	0.10	0.1680	0.581	0.9972
	UEP, Catarman, Samar	61.889	0.40	0.2300	0.681	0.9905
	Catbalogan, Samar	51.105	0.10	0.2020	0.620	0.9948
	Tacloban, Leyte	39.661	0.10	0.1660	0.629	0.9968
9	Zamboanga City	48.571	0.30	0.2090	0.803	0.9973
10	Cagayan de Oro	78.621	0.50	0.1950	0.954	0.9992
	Surigao City	61.486	0.60	0.2520	0.602	0.9901
	Binatuan, Surigao	-	-	-	-	-
	del Sur	57.433	0.10	0.1340	0.577	0.9932
11	Davao City	81.959	0.50	0.1740	0.945	0.9986

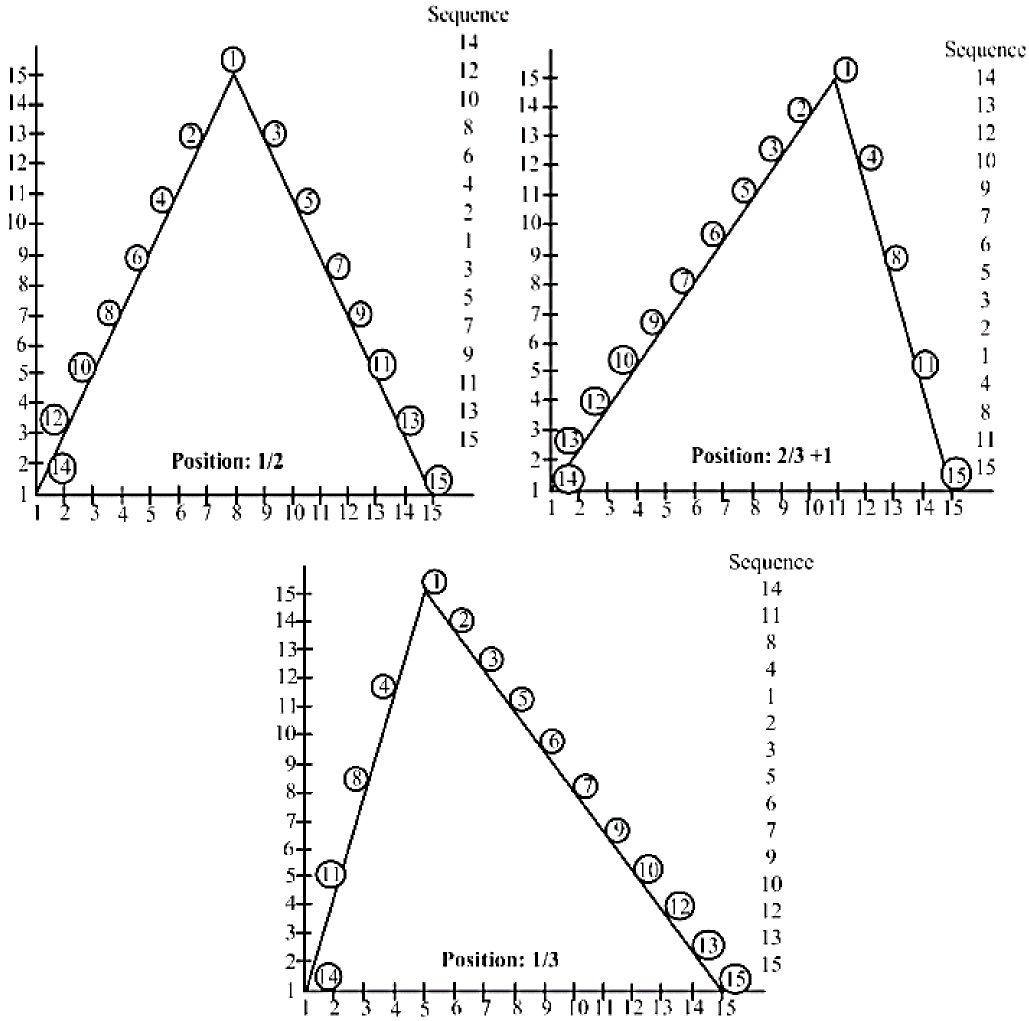

Figure A.1. Maximization Patterns for Rainfall Increments

A.1.4 The succeeding procedures, from section A.1.5 to A.1.8 shall be applied to all rainfall maximization patterns to determine which will result to maximum rainfall excess amounts. However, 2/3+1 position pattern usually result to the maximum.

A.1.5 Compute for the initial abstraction.

$$I_a = 0.2s$$

$$s = \frac{1000}{W} - 10$$

where:

I_a	is the initial abstraction (in)
s	is the maximum potential difference between rainfall and runoff (in)
W	is the watershed index or runoff curve number which is a function of: soil group (see Table A.3), antecedent moisture condition (see Table A.4), and land use cover (Table A.5) in the watershed

A.1.6 Adjust values of W for AMC I and AMC III using Table A.6.

A.1.7 Subtract he computed initial abstraction from the rainfall depth over the necessary initial number of time increment until Ia is satisfied.

A.1.8 Apply values of uniform retention rate, f, in succeeding time increments so that the retention depth subtracted each time from a rainfall increment is at most equal to f × ΔP, Applicable values are given in Table A.7. Tabulated sample computation of these values is shown in Table A.8.

Table A.3. Soil Groups for Estimation of Watershed Index W

Soil Group	Soil Characteristics	Example
A	Soils with very low runoff potential	Deep sand with little silt or clay
B	Light soils under/or well - structured soils with above average infiltration when thoroughly melted	Light sandy loam, silty loam
C	Medium soils and shallow soils with below-average infiltration when thoroughly melted	Clay loam
D	Soils with high runoff potential	Heavy soils, particularly days of high swelling capacity, and very shallow soils underlain by dense clay horizons

Table A.4. Antecedent Moisture Conditions for Estimation of Watershed Index

Antecedent Moisture Condition (AMC)	Rain in the Previous 5 Days	
	Dormant Season	Growing Season
I	Less than 0.5 in	Less than 1.4 in
II	0.5 in to 1.1 in	1.4 in to 2.1 in
III	More than 1.1 in	More than 2.1 in

Table A.5. Values of Watershed Index
(Assuming Antecedent Moisture Condition II)

Land Use or Cover	Farming Treatment	Hydrologic Condition	Soil Group			
			A	B	C	D
Native pasture or grassland	-	Poor	70	80	85	90
		Fair	50	70	80	85
		Good	40	60	75	80
Timbered Areas	-	Poor	45	65	75	85
		Fair	35	60	75	80
		Good	25	55	70	75
Improved Permanent Pastures		Good	30	60	70	80
Rotation Pastures	Straightrow	Poor	65	75	85	90
		Good	60	70	80	85
	Contoured	Poor	65	75	80	85
		Good	55	70	80	85
Crop	Straightrow	Poor	65	75	85	90
		Good	70	80	85	90
	Contoured	Poor	70	80	85	90
		Good	65	75	80	85
Fallow	-	-	80	85	90	95

NOTE:
Native pastures - Pastures in poor condition is sparse, heavily grazed pastures with less than half the total watershed area under plant cover. Pasture in fair condition is moderately grazed and with between half and three-quarters of the catchment under plant cover. Pasture in good condition is lightly grazed and with more than three-quarters of the catchment area under plant cover
Timbered areas - Poor areas are sparsely timbered and heavily grazed with no undergrowth. Fair areas are moderately grazed, with some undergrowth. Good areas are densely timbered and ungrazed, with considerable undergrowth.
Improved permanent pastures - Densely sown permanent legume pastures subject to careful grazing management are considered to be in good hydrologic condition
Rotation pastures - Dense, moderately grazed pastures used as part of a well planned, crop-pasture-fallow rotation are considered to be in good hydrologic condition. Sparse, overgrazed or “opportunity” pastures are considered to be poor condition.
Crops - Good hydrologic condition refers to crops which form a part of a well planned and managed crop-pasture-follow rotation. Poor hydrologic condition refers to crops managed according to a simple crop-follow-rotation.

Table A.6. Adjustments for Watershed Index

Corresponding Values of W for:		
AMC = II	**AMC = I**	**AMC = III**
100	100	100
95	87	99
90	80	98
85	70	97
80	65	95
75	60	90
70	50	90
65	45	85
60	40	80
55	35	75
50	30	70
45	25	65
40	20	60
35	20	55
30	15	50
25	10	45

Table A.7. Recommended Retention Rate for Hydrologic Soil Group (USBR)

Hydrologic Soil Group	Retention Rate, in/h
A	0.4
B	0.24
C	0.12
D	0.04

Table A.8. Sample Rainfall Excess Computation Using Hydrologic Abstraction

Sequence Number	Rainfall Increments (mm)	Abstraction (mm)	Retention (mm)	Rainfall Excess, E (mm)
1	7.67	7.67	0.00	0.00
2	8.06	8.06	0.00	0.00
3	8.51	1.20	0.00	7.30
4	9.64	0.00	1.22	8.42
5	10.36	0.00	1.22	9.14
6	12.37	0.00	1.22	11.15
7	13.83		1.22	12.61
8	15.81		1.22	14.59
9	23.34		1.22	22.12
10	32.20		1.22	30.98
11	56.56		1.22	55.34
12	18.70		1.22	17.84
13	11.26		1.22	10.04
14	9.03		1.22	7.81
15	7.33		1.22	6.11

A.1.9 Derive the synthetic unit hydrograph using the dimensionless-unit hydrograph.

A.1.9.1 Interpolate the values from Table A.9 until q/q_p is less than 0.001.

$$\frac{T}{T_p} = \frac{\Delta D}{T_p}, \frac{2\Delta D}{T_p}, \frac{3\Delta D}{T_p}, ..$$

where:

T is the corresponding ΔD
T_p is the time to peak (h)
ΔD is the standard rainfall duration (h)

Table A.9. Time Ratio and Discharge Ratio for Dimensionless-Unit Hydrograph

Time Ratio T/T_p	Discharge Ratio q/q_p	Time Ratio T/T_p	Discharge Ratio q/q_p
0	0	1.5	0.66
0.1	0.015	1.6	0.56
0.2	0.175	1.8	0.42
0.3	0.16	2	0.32
0.4	0.28	2.2	0.24
0.5	0.43	2.4	0.18
0.6	0.6	2.6	0.13
0.7	0.77	2.8	0.098
0.8	0.89	3	0.075
0.9	0.97	3.5	0.036
1	1	4	0.018
1.1	0.98	4.5	0.009
1.2	0.92	5	0.004
1.3	0.84	Infinity	0
1.4	0.75		

A.1.9.2 Compute the ordinate of the synthetic unit hydrograph.

$$U_i = \left(\frac{q}{q_p}\right)_i \times q_p$$

where:

U_i is the ordinate of synthetic unit hydrograph (cm/mm)
$(q/q_p)_I$ is the interpolated value from dimensionless hydrograph
q_p is the computed peak rate of runoff (cm/mm)

A.1.9.3 Determine the correction factor for synthetic unit hydrograph.

$$K = \frac{3.6 \sum U_i \times \Delta D}{A}$$

where:

K	is the correction factor
U_i	is the ordinate of synthetic unit hydrograph (cm/mm)
ΔD	is the standard rainfall duration (h)
A	is the drainage area (km^2)

A.1.9.4 Apply correction factor and tabulate results as shown in Table A.10.

Table A.10. Summary of Values for the Synthetic Unit Hydrograph

Sequence Number	Time (h)	Time Ratio T/T_p	Discharge Ratio q/q_p	U_i	Adjusted U_i (cm/mm) $Uu = U_i/K_i$
1	ΔD	$\Delta D/T_p$	Interpolated values from Table A.9	$(q/q_p)_1\ q_p$	Uu_1
2	2ΔD	$2\Delta D/T_p$		$(q/q_p)_2\ q_p$	Uu_2
3	3ΔD	$3\Delta D/T_p$		$(q/q_p)_3\ q_p$	Uu_3
4	4ΔD	$4\Delta D/T_p$		$(q/q_p)_4\ q_p$	Uu_4
5	5ΔD	$5\Delta D/T_p$		$(q/q_p)_5\ q_p$	Uu_5
n	nΔD	$n\Delta D/T_p$		$(q/q_p)_n\ q_p$	Uu_n

A.1.9.5 Use the synthetic unit hydrograph according to the convolution equations to determine the ordinates of the direct runoff hydrograph.

$$Q_1 = Uu_1 \times E_1$$
$$Q_2 = (Uu_1 \times E_2) + (Uu_2 \times E_1)$$
$$Q_3 = (Uu_1 \times E_3) + (Uu_2 \times E_2) + (U_{u3} \times E_1)$$
$$Q_4 = (Uu_1 \times E_4) + (Uu_2 \times E_3) + (Uu_3 \times E_2) + (Uu_4 \times E_1)$$

where:

Q_i	is the runoff value at nΔD
E_i	is the rainfall excess
Uu_i	is the adjusted ordinate of the unit hydrograph

A.2 Field Water Balance Computation

A.2.1 Establish the most suitable cropping pattern and cropping calendar with the following objectives:

- minimum irrigation requirements
- maximum annual production
- optimum growing conditions for the given crop and growing stages

- growing crop during wet season when water is abundant and irrigation is minimal

A.2.2 Determine the following required data

A.2.2.1 Dependable Rainfall – 10- day dependable rainfall can be determined using various hydrologic frequency analysis. Some of these methods are detailed in Annex D of PNS/BAFS/PAES 217:2017 – Determination of Irrigation Water Requirements.

A.2.2.2 Reference Evapotranspiration – determination of the reference evapotranspiration can be determined using recommended procedures detailed in Annex B of PNS/BAFS/PAES 217:2017 – Determination of Irrigation Water Requirements.

A.2.2.3 Crop Coefficient – the crop coefficient of the crop grown in the site during various stages shall be known. Details for some crops are listed in Table 4 of PNS/BAFS/PAES 217:2017 – Determination of Irrigation Water Requirements.

A.2.2.4 Seepage and Percolation Losses – determination of these losses shall be based on the soil type in the site. Estimated percolation values are shown in Table 5 of PNS/BAFS/PAES 217:2017 – Determination of Irrigation Water Requirements.

A.2.3 Determine the irrigation requirements using the data above. A step by step procedure is detailed in PNS/BAFS/PAES 217:2017 – Determination of Irrigation Water Requirements.

A.3 Estimation of 10-day Reservoir Inflow

A.3.1 For Regions I. III and IV, characterized by distinct wet and dry seasons, estimate the 10-day reservoir inflow as follows:

$$DQ_j = RC_j \times P_j$$
$$BF_j = F \times Q_{j-1}$$
$$Q_j = DQ_j + BF_j$$

where:

DQ_j	is the direct runoff in decade j (mm)
RC_j	is the runoff coefficient in decade j, equal to estimated mean monthly runoff coefficient (see Table A.11)
P_j	is the 80% dependable rainfall
BF_j	is the baseflow in decade j (mm)
F	is the 10-day reservoir factor = 0.002+(0.026×DA) where DA is drainage area in km^2 NOTE: Obtained from regression equation analysis of several small watersheds <100 km^2 in the Philippines)
Q_j	is the reservoir inflow in decade j (mm)
Q_{j-1}	is the inflow in the previous decade (mm)

A.3.2 For other regions in the country which are predominantly characterized by indistinct, short or no dry season with more or less continuous rainfall, estimate the 10-day reservoir inflow as follows:

$$DQ_j = RC_j \times P_j$$
$$BF = a + b \times DA$$
$$BF_j = BF \times \frac{\%\, monthly\ BF}{3}$$
$$Q_j = DQ_j + BF_j$$

where:

DQ_j	is the direct runoff in decade j (mm)
RC_j	is the runoff coefficient in decade j, equal to estimated mean monthly runoff coefficient (see Table A.11)
P_j	is the 80% dependable rainfall
BF	is the annual baseflow (mm)
BF_j	is the baseflow in decade j (mm)
a,b	is the coefficient of linear curve fit (see Table A.11)
DA	is the drainage area (km^2)
Q_j	is the reservoir inflow in decade j (mm)

A sample computation is shown in Table A.12.

Table A.10. Regional Runoff Coefficient and Monthly Baseflow Distribution

Region	Month	% Baseflow	Runoff Coefficient
1	January	-	0.25
	February	-	0.05
	March	-	0.03
	April	-	0.03
	May	-	0.17
	June	-	0.37
	July	-	0.64
	August	-	0.67
	September	-	0.75
	October	-	0.75
	November	-	0.61
	December	-	0.25
2	January	8.76	0.17
	February	7.91	0.17
	March	7.22	0.08
	April	7.05	0.08
	May	6.7	0
	June	6.42	0.17
	July	7.39	0.2
	August	8.18	0.34

	September	9.37	0.4
	October	10.43	0.41
	November	10.84	0.44
	December	9.72	0.37
	a = 286.021, b = -9.72 x 10^{-1}, R = 0.74		
3	January	-	0.45
	February	-	0.08
	March	-	0
	April	-	0
	May	-	0.24
	June	-	0.34
	July	-	0.58
	August	-	0.7
	September	-	0.75
	October	-	0.7
	November	-	0.4
	December	-	0.5
4	January	-	0.45
	February	-	0.44
	March	-	0.19
	April	-	0
	May	-	0
	June	-	0.19
	July	-	0.19
	August	-	0.26
	September	-	0.33
	October	-	0.47
	November	-	0.57
	December	-	0.5
5	January	9.17	0.5
	February	8.69	0.38
	March	8.28	0.3
	April	7.91	0.25
	May	7.64	0.1
	June	7.66	0.08
	July	7.86	0.15
	August	8.08	0.15
	September	8.31	0.15
	October	8.53	0.35
	November	8.79	0.39
	December	9.07	0.47
	a = 2057.31, b = 18.28, R = 0.87		
6	January	8.06	0.39
	February	8.1	0.19
	March	7.96	0.16
	April	8.1	0.16
	May	8.26	0.16

	June	8.45	0.18
	July	8.66	0.44
	August	8.73	0.44
	September	8.6	0.33
	October	8.47	0.49
	November	8.37	0.39
	December	8.21	0.39
	a = 1043.65, b = 8.221, R = 0.695		
7	January	8.23	0.26
	February	8.07	0.15
	March	8.09	0.1
	April	8.22	0
	May	8.23	0.09
	June	8.35	0.15
	July	8.47	0.3
	August	8.66	0.3
	September	8.57	0.3
	October	8.45	0.3
	November	8.37	0.3
	December	8.29	0.26
	a = 1055.85, b = 11.80, R = 0.766		
8	January	9.1	0.38
	February	8.8	0.28
	March	8.6	0.25
	April	8.3	0
	May	8.1	0.14
	June	7.9	0.22
	July	7.7	0.3
	August	7.6	0.34
	September	7.7	0.34
	October	7.9	0.51
	November	8.4	0.7
	December	9	0.7
	a = 12.52, b = 14.051, R = 0.872		
9	January	8.53	0.3
	February	8.33	0.22
	March	8.16	0.08
	April	7.94	0
	May	8	0
	June	8.13	0.07
	July	8.19	0.14
	August	8.32	0.14
	September	8.42	0.14
	October	8.53	0.24
	November	8.66	0.24
	December	8.76	0.3
	a = 1164.37, b = 30.36, R = 0.999		

10	January	8.51	0.49
	February	8.43	0.4
	March	8.36	0.37
	April	8.29	0.32
	May	8.21	0.15
	June	8.16	0.15
	July	8.21	0.15
	August	8.27	0.24
	September	8.3	0.24
	October	8.34	0.28
	November	8.4	0.25
	December	8.49	0.52
	a = 2119.90, b = 6.09, R = 0.562		
11	January	8.42	0.17
	February	8.38	0
	March	8.35	0
	April	8.31	0
	May	8.3	0.12
	June	8.25	0.12
	July	8.27	0.29
	August	8.3	0.29
	September	8.32	0.26
	October	8.34	0.26
	November	8.37	0.23
	December	8.39	0.22
	a = 152.608, b = 7.53, R = 0.751		
12	January	8.13	0.21
	February	7.99	0.12
	March	8.03	0
	April	8.13	0.13
	May	8.24	0.25
	June	8.39	0.35
	July	8.54	0.44
	August	8.69	0.45
	September	8.66	0.45
	October	8.53	0.45
	November	8.4	0.21
	December	8.26	0.21
	a = 1751.61, b = -4.018, R = 0.915		

Table A.11. Sample Derivation of Reservoir Inflow

Region: II

Drainage Area: 187 ha

Month	Decade	Mean Rainfall (mm)	Runoff Coefficient	Direct Runoff (mm)	Baseflow (mm)	Reservoir Inflow	
						mm	m^3
Jan	1	2.08	0.17	0.35	8.30	8.65	1180
	2	5.59	0.17	0.95	8.30	9.25	17295
	3	15.33	0.17	2.61	8.30	10.91	20393
Feb	4	8.62	0.17	1.47	7.49	8.96	16753
	5	8.39	0.17	1.43	7.49	8.92	16681
	6	4.99	0.17	0.85	7.49	8.34	15599
Mar	7	13.70	0.08	1.10	6.84	7.94	14839
	8	41.09	0.08	3.29	6.84	10.13	18937
	9	8.22	0.08	0.66	6.84	7.50	14020
Apr	10	28.38	0.08	2.27	6.88	8.95	16736
	11	22.22	0.08	1.78	6.88	8.46	15813

A.4 Reservoir Operation Study

The reservoir operation study must be performed to optimize the reservoir to meet water requirements. In this procedure, it is assumed that the reservoir elevation at the end of the operation will be equal to the starting elevation

A.4.1 Obtain the following data.

A.4.1.1 Reservoir Inflow – detailed in section A.3 of this standard

A.4.1.2 Reservoir Evaporation Loss – can be determined from meteorological data

A.4.1.3 Irrigation Water Requirements - detailed in PNS/BAFS/PAES 217:2017 – Determination of Irrigation Water Requirements

A.4.1.4 Reservoir Area-Capacity Elevation Curve – sections A.4.1.4.1 to A.4.1.4.5

A.4.1.4.1 From the topographic map of the delineated watershed, determine the area within each contour elevation.

A.4.1.4.2 Determine the average area between consecutive contour elevations.

A.4.1.4.3 From the average area and contour interval, compute for the incremental volume.

A.4.1.4.4 Determine the accumulated storage by adding the incremental volume between contour elevations to the preceding accumulated storage. Table A.12 shows a sample computation for determining the area-capacity elevation curve.

A.4.1.4.5 Construct plots of elevation of the reservoir versus surface area and elevation of the reservoir versus volume.

A.4.2 From the Elevation-Storage Capacity Curve, determine the normal water surface (NWS) Elevation. Assume a starting elevation lower than the NWS Elevation.

A.4.3 Follow the procedures detailed in flowchart for the reservoir operation study shown in Figure A.2. Table A.13 shows the parameters required and computed for the study.

A.4.4 Check the water surface elevation at the end of last decade of the last cropping. The starting elevation must be equal or lower than the resulting water surface elevation.

A.4.5 If two or more consecutive shortages are observed, reduce service area of the cropping period or adjust the normal water surface elevation.

Table A.12. Sample Computation for the Area-Capacity Elevation Curve

Contour Interval: 1

Contour Elevation (m)	Area Within Contour (m^2)	Average Area (m^2)	Incremental Volume (m^3)	Accumulated Storage (m^3)
82	40	0	0	0
83	80	60	60	60
84	1600	840	840	900
85	7320	4460	4460	5360
86	16420	11870	11870	17230
87	17720	17070	17070	34300
88	18920	18320	18320	52620
89	19710	19315	19315	71935
90	21250	20480	20480	92415
91	22240	21745	21745	114160
92	31800	27020	27020	141180
93	47880	39840	39840	181020
94	58920	53400	53400	234420
95	73200	66060	66060	300480
96	82880	78040	78040	378520
97	90240	86560	86560	465080
98	109240	99740	99740	564820
99	139270	124255	124255	689075
100	141440	140355	140355	829430

Table A.13. Parameters for the Reservoir Operation Study

Normal Water Surface Elevation:		**Storage at NWS:**	
Minimum Elevation:		**Minimum Storage:**	
Starting Elevation:		**Starting Storage:**	

Decade or Month	Inflow	Diversion Water Requirement	Evaporation Rate	Irrigation Demand	Evaporation Demand	Total Release	Storage	Elevation	Shortage	Spill
M	I	DWR	ER	ID	ED	R	STO	EL	SRT	SPL
-	m^3	mm	mm	m^3	m^3	m^3	m^3	m	m^3	m^3

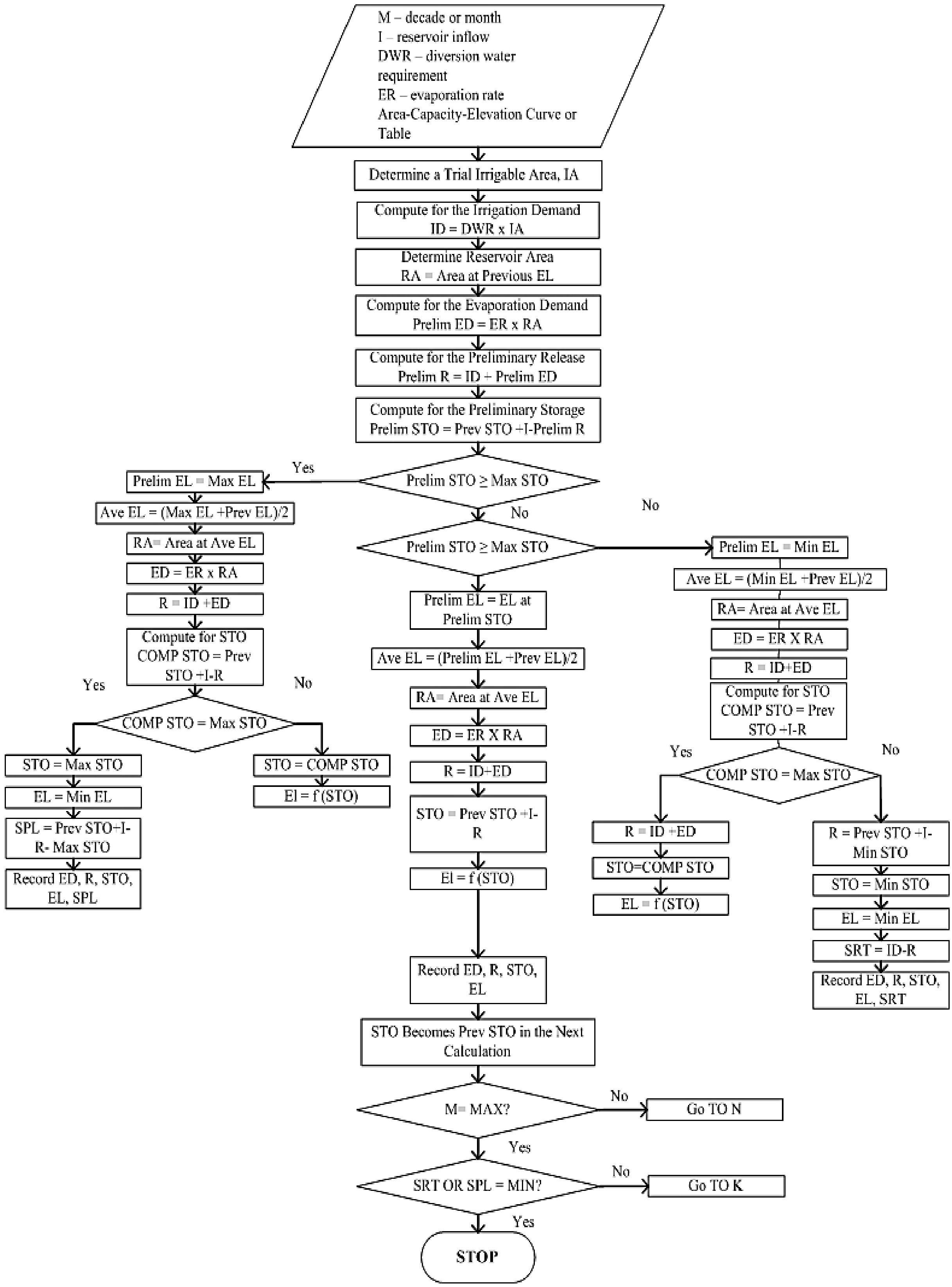

Figure A.2. Flow Chart for Reservoir Operation Study

A.5 Flood Routing

There are a number of methods used in flood routing such as Modified Pul's Method, Goodrich Method and other graphical and analog methods. In this informative annex a simple and expedient method by arithmetic trial and error will be used.

A.5.1 The following assumptions are applied:

A.5.1.1 All outlets are fully closed and all discharges are allowed to pass only over the spillway.

A.5.1.2 Water surface in the reservoir is at normal level at the start of the flood.

A.5.2 The following data are required:

A.5.2.1 Inflow hydrograph of the design flood.

A.5.2.2 Reservoir capacity-elevation curve

A.5.2.3 Spillway rating curve or equation for a broad crested weir:

$$Q = CLH^{3/2}$$

where:

Q	is the discharge over the spillway (m^3)
C	is the weir coefficient, 1.704
H	is the surcharge height (m)
L	is the spillway width (m)

A.5.3 Follow the procedures detailed in flowchart for flood routing shown in Figure A.3. Table A.14 shows the parameters required and computed for flood routing.

Table A.14. Parameters for Flood Routing

Time	ΔT	Inflow		Trial Elevation, EL_t	Outflow		Change in Storage, ΔS	Storage, S	Computed Elevation, EL_c
h	h	I_q, cm	I_{vol}, m^3	m	O_q, cm	O_{vol}, m^3	m^3	m^3	m

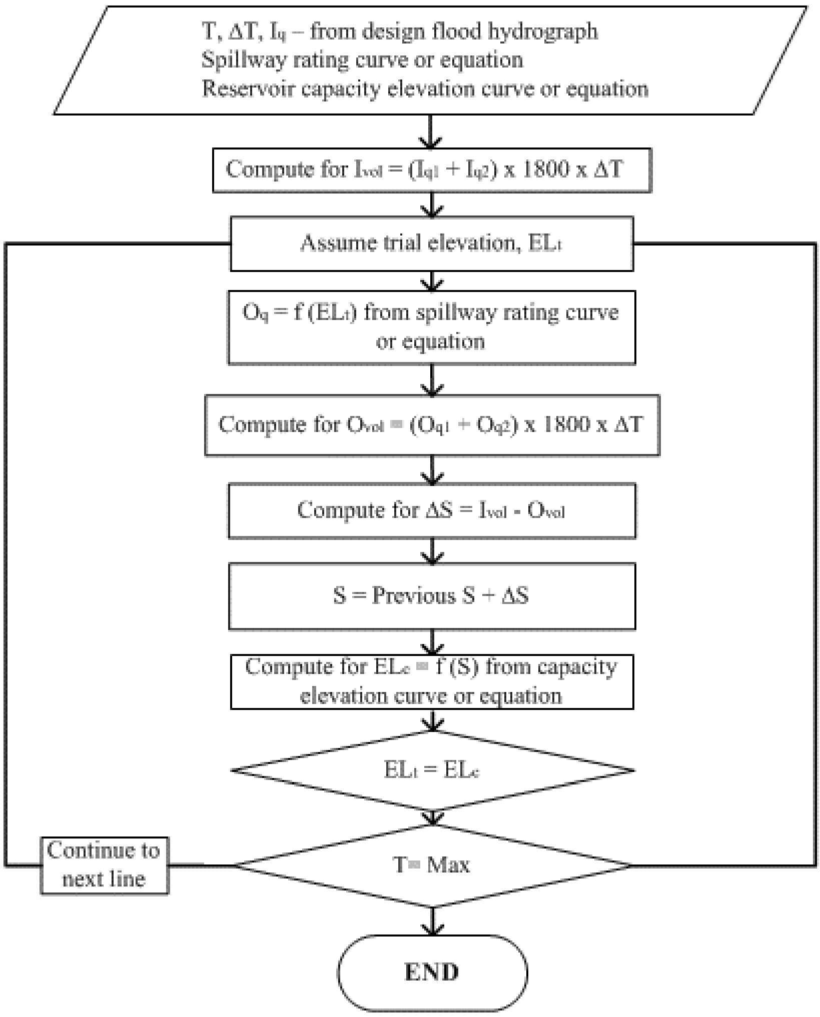

Figure A.3. Flow Chart for Flood Routing

A.6 Summary of Computation

No.	Parameter	Formula	Value (m)
1	Creek Bed Elevation	Basic data	
2	Min. Water Surface Elevation	From Dead Storage	
3	Normal Water Surface Elevation	From Reservoir Operation Study	
4	Flood Surcharge Height	From Flood Routing	
5	Max. Water Surface Elevation	Item 3 + Item 4	
6	Freeboard due to Wave Run-up	From section 7.1.1	
7	Preliminary Dam Crest Elevation	Item 5 + Item 6	
8	Preliminary Dam Height	Item 7 – Item 1	
9	Embankment Settlement	From section 7.1.1	
10	Final Dam Height	Item 8 + Item 9	
11	Final Dam Crest Elevation	Item 1 + Item 10	

ANNEX B
(informative)

Design of Embankment Components

B.1 Embankment Slope Protection

B.1.1 Size of Concrete Cube

$$W = \frac{0.0023{H_w}^3 S_r C {S_c}^3 (p - a)}{\left(\frac{S_r}{S_w} - 1\right)}$$

$$H_w = 0.032\sqrt{FV} + 0.763 - 0.271\sqrt[4]{F}$$

where:

W	is the weight of concrete (tons)
H_w	is the wave height (m)
F	is the effective reservoir fetch (km)
V	is the wind velocity (km/h)
S_r	is the specific gravity of concrete
S_w	is the specific gravity of water
C	is the riprap factor (0.54 for hand-laced and 0.80 for dumped)
p	is the 70% for dumped riprap
a	is the angle of face slope from horizontal

B.1.2 Size of Rock or Stone

$$d_m = 2.23 C H_w \frac{W}{G - W} \times \frac{\sqrt{1 + S^2}}{S(S + 2)}$$

where:

d_m	is the riprap diameter (m)
W	is the unit weight of water (ton/m^3)
G	is the unit weight of stone (ton/m^3)
S	is the slope of embankment
H_w	is the wave height (m)
C	is the riprap factor (0.54 for hand-laced and 0.80 for dumped)

B.2 Filter Drain - required between the impervious core and outer shell of zoned dams and on horizontal drainage blanket or toe drains of modified homogeneous dams to prevent migration of small particles and to screen off fine materials that flow with seepage water

B.2.1 Requirements:

- Graduation must be such that the particles of soil are prevented from entering the filter and clogging it.
- Capacity of the filter must be such that it adequately handles total seepage flow.
- Permeability must be great enough to provide easy access of seepage water so that uplift forces are reduced

B.2.2 Recommended limits to satisfy filter stability criteria:

- $\frac{D_{15}\text{ of filter}}{D_{15}\text{ of material}} = 5\text{ to }40$

  where D_{15} = size at which 15% of the total soil particles are smaller

  (provided that the filter does not contain more than 5% of material passing through No. 200 sieve)

- $\frac{D_{15}\text{ of filter}}{D_{85}\text{ of material}} = 5$

where D_{85} = size at which 85% of the total soil particles are smaller

- The grain size curve of the filter should be roughly parallel to that of the base material. If more than one filter layer is required, the same criteria are followed. The finer filter is considered as base material for selection of the gradation of coarse material.

B.2.3 Design Equation

$$Q = \frac{kt^2w}{L}$$

where:

Q	is the design seepage value (equivalent to 5-10 times the estimated embankment seepage)
k	is the average permeability of filter material
t	is the thickness of drain
L	is the length of drain
w	is the width of drain (perpendicular to flow)

ANNEX C
(informative)

Spillway Design

C.1 Determination of Flow Depth Along the Discharge Channel

C.1.1 For unlined, grassed and riprapped channel, use the Manning's Equation in a trial and error solution to detemine the flow depth

$$\frac{(bd + zd)^{5/3}}{(b + 2d\sqrt{z^2 + 1})^{2/3}} = \frac{Qn}{\sqrt{S}}$$

where:

b	is the channel bed width, m
d	is the flow depth, m
z	is the channel side slope
Q	is the discharge, m^3/s
n	is the channel roughness coefficient
S	is the channel slope

C.1.2 For concrete-lined channels where the flow is supercritical, use the Energy, Manning's and Continuity Equations in a trial and error solution to determine the flow depth. The formula below shall be satisfied.

$$M + d_c + hv_c = d_1 + hv_1 + hf_1$$

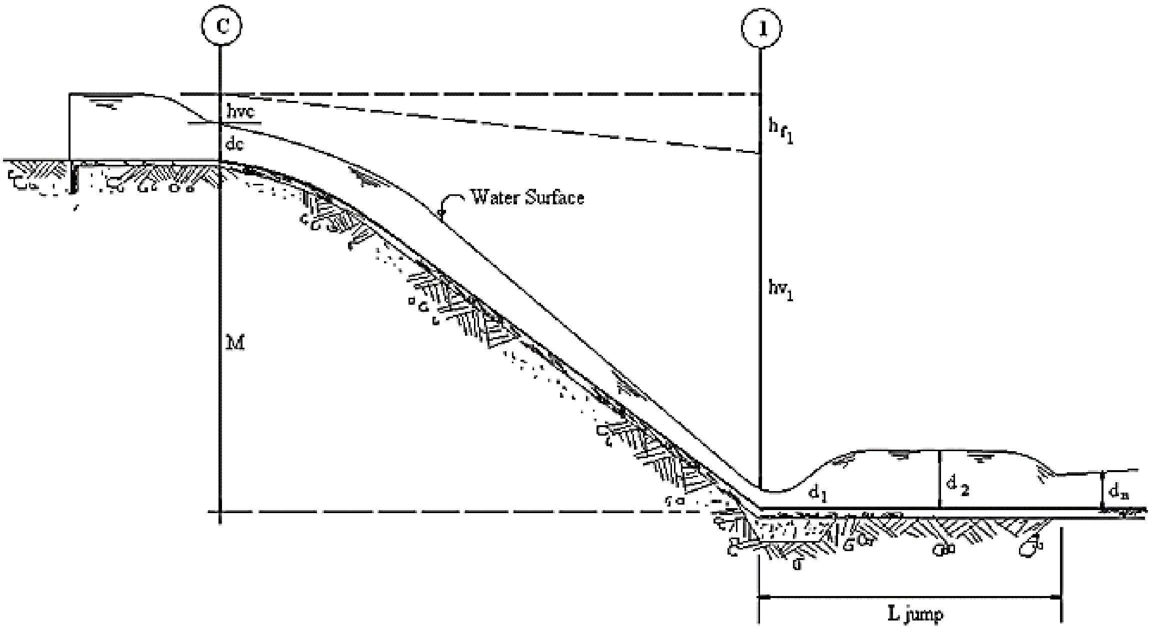

Figure C.1. Spillway Flow Profile

C.2 Freeboard Along the Discharge Channel

$$Fb = 2.0 + 0.025V\sqrt[3]{d}$$

where:

Fb is the freeboard, ft
V is the average velocity of the channel reach, fps
d is the average depth of flow within the reach, ft

C.3 Terminal Section

C.3.1 Unsubmerged Deflector Bucket

C.3.1.1 Hydraulic design considerations:

- The exit angle must not greater than 30°
- The bucket radius should be long enough to maintain a smooth and concentric flow. Minimum bucket radius should not be less than 5 times the depth of flow.

C.3.1.2 Compute the horizontal range of the jet using the formula:

$$X = 1.8\, sin^2 A(d + h_v)$$

where:

X = horizontal range of the jet, m
A = exit angle of the bucket lip
d = depth of flow at the bucket, m
h_v = velocity head, m

C.3.2 USBR Hydraulic Jump Type Basin

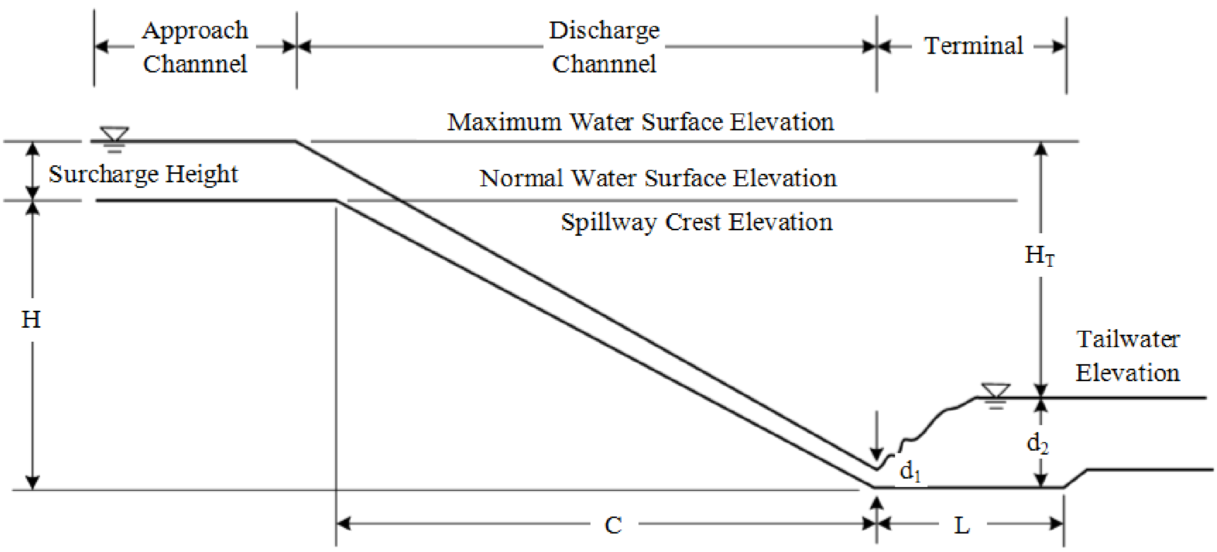

Figure C.2. Dimensions for USBR Basin Computation

C.3.2.1 Determine the jump depth, d_2 from the nomograph in Figure C.3

C.3.2.2 Compute for the velocity of flow.

$$V_2 = \frac{Q_o}{bd_2}$$

where:

V_2	is the flow velocity after hydraulic jump, m/s
Q_o	is the routed flow, m^3/s
b	is the spillway width, m
d_2	is the jump depth, m

C.3.2.3 Compute for d_1.

$$d_1 = \sqrt{\frac{2V_2d_2}{g} + \frac{{d_2}^2}{4} - \frac{d_2}{2}}$$

where:

d_1	is the depth before jump (m)
V_2	is the flow velocity after hydraulic jump (m/s)
d_2	is the jump depth (m)
g	is the gravitational acceleration (m/s^2)

C.3.2.4 Select the type of USBR Basin based on the Froude Number shown in Table C.1.

$$F = \frac{V}{\sqrt{gd_1}}$$

where:

F	is the Froude Number
V	is the velocity at entrance to the basin (m/s)
g	is the gravitational acceleration (m/s^2)
d_1	is the depth of flow at the entrance to the basin (m)

Table C.1. USBR Basin Selection

Froude Number	USBR Basin
$2.5 \leq F \leq 4.5$	Type IV
$F > 4.5$ ($V \leq 60$ fps)	Type III
$F > 4.5$ ($V > 60$ fps)	Type II

C.3.2.2 Determine the basin length from L/d_2 –F curve in Figures C.4 to C.6.

C.3.2.3 Compute for the basin freeboard.

$$Fb = 0.1(V_1 + d_2)$$

where:

Fb	is the basin freboard, m
V_1	is the velocity to the entrance of the basin
d_2	is the jump depth, m

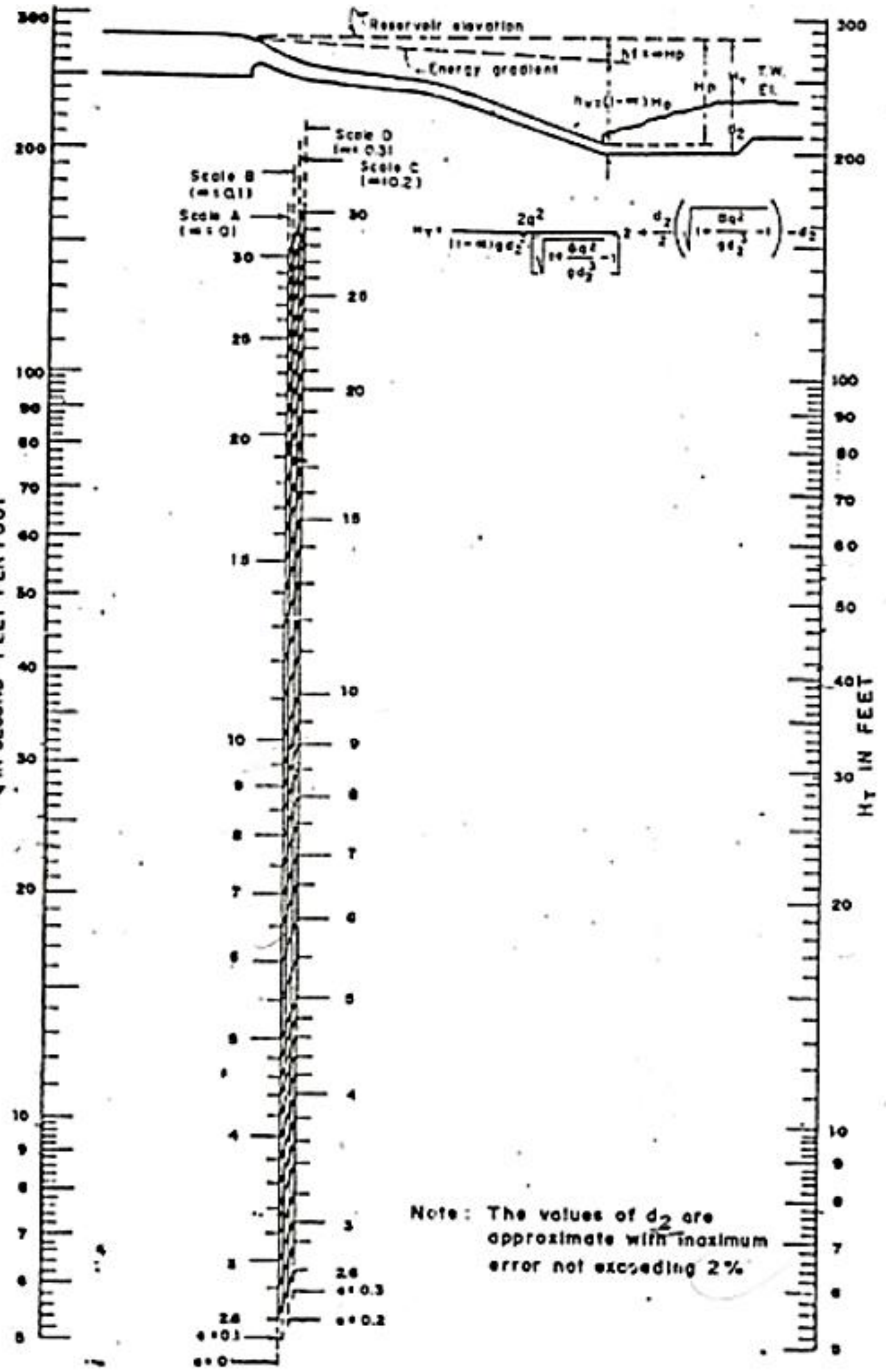

Figure C.3. Stilling Basin Depths Versus Hydraulic Heads for Various Channel Losses

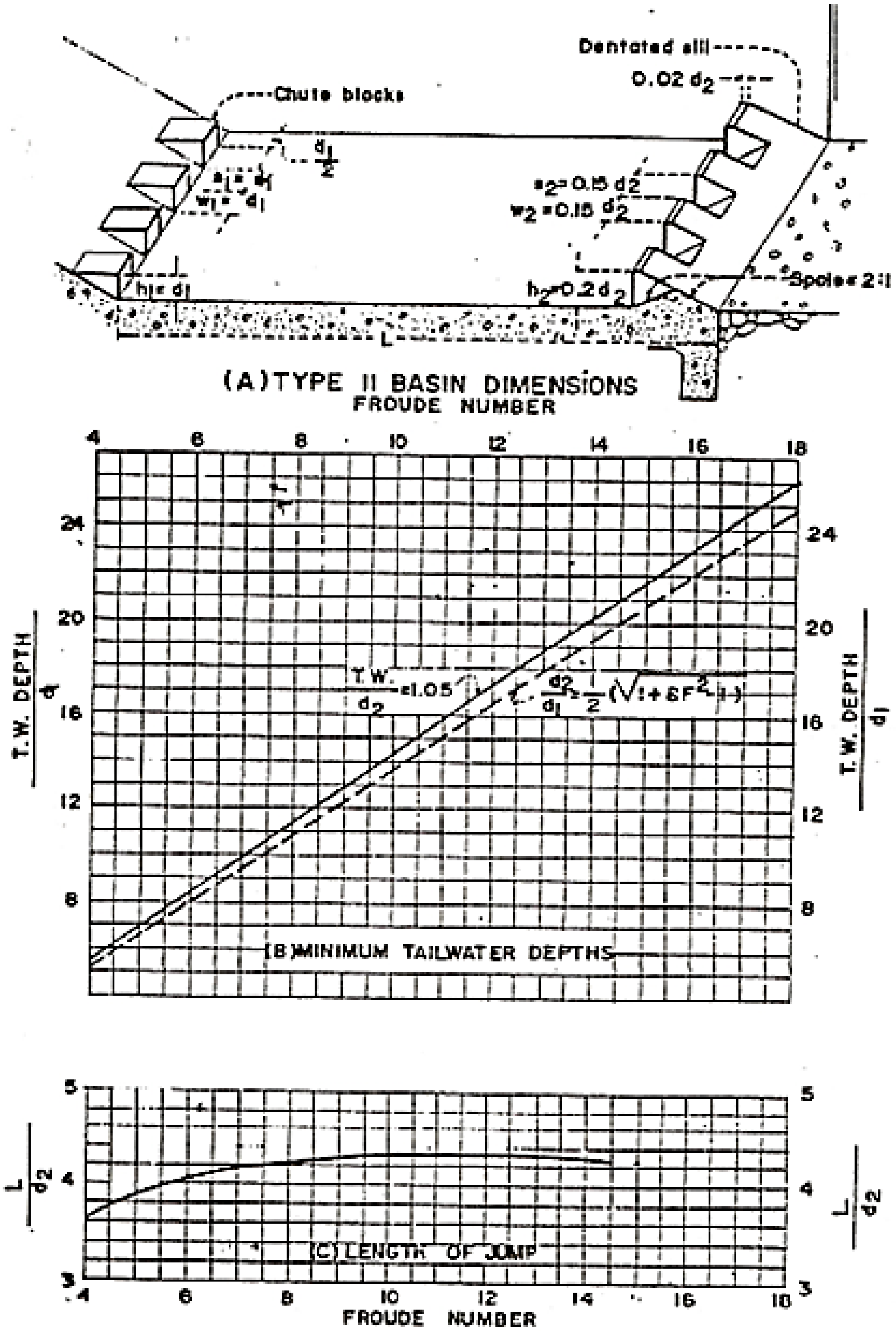

Figure C.4. Type II USBR Basin (F>45; V>60 fps)

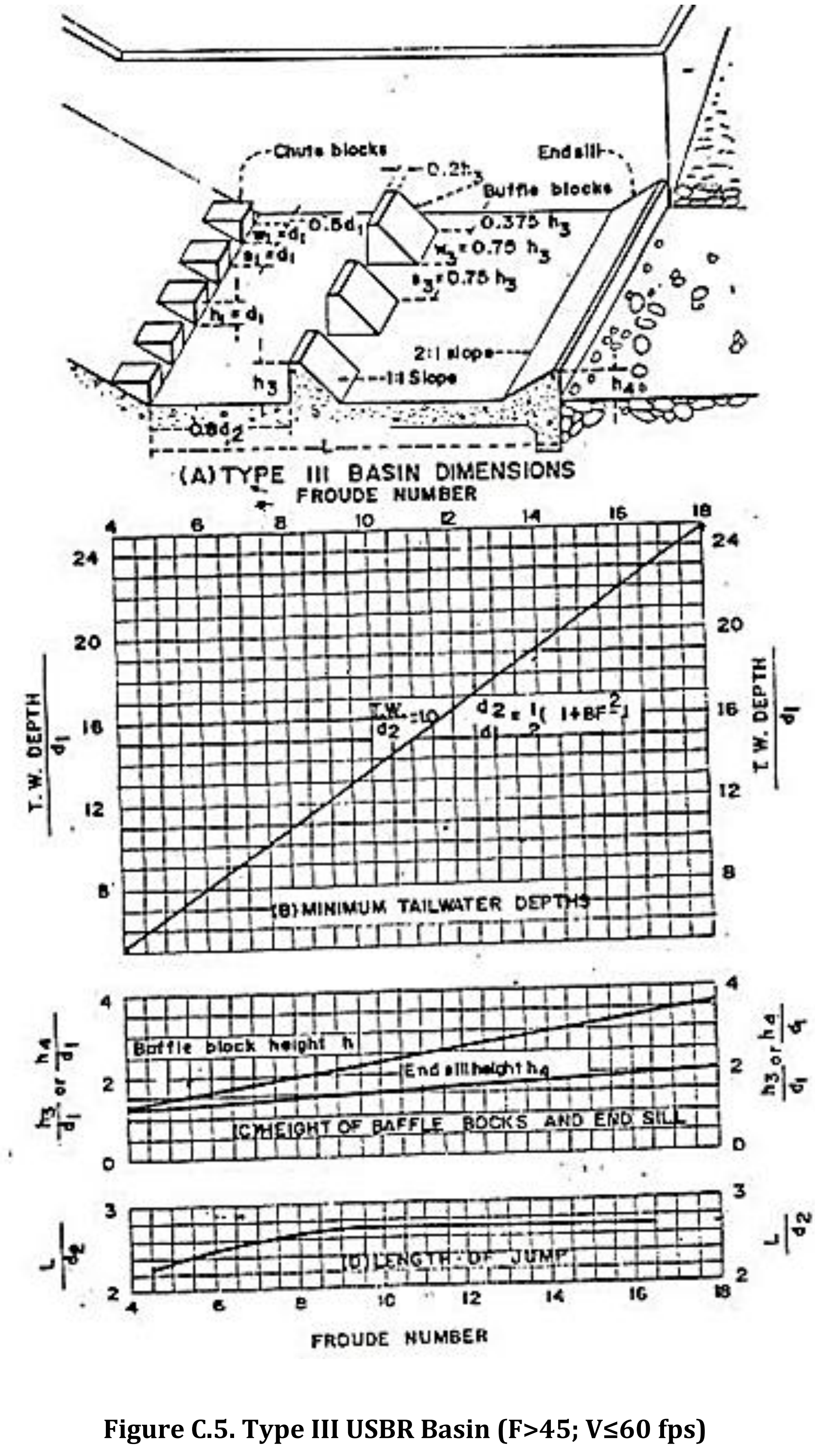

Figure C.5. Type III USBR Basin (F>45; V≤60 fps)

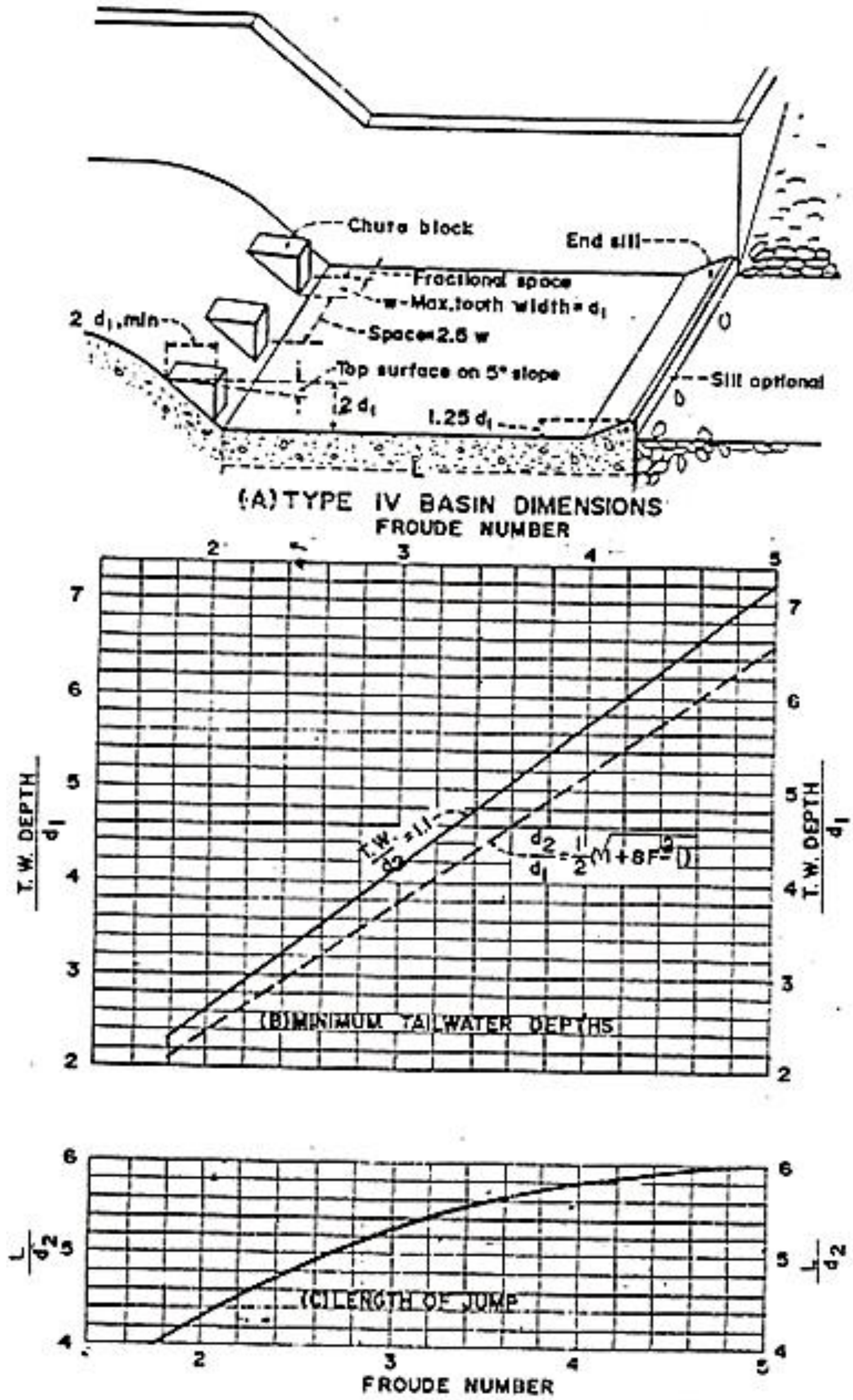

Figure C.6. Type IV USBR Basin

C.4 Structural Requirement

The design of a concrete cantilever is shown in Table C.2 Refer to Figure C.7 for the symbols used.

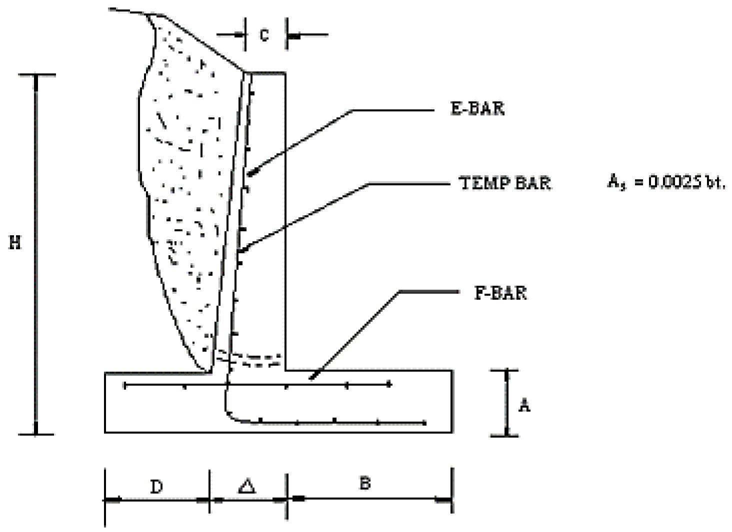

Figure C.7. Cantilever Retaining Wall

Table C.7 – Parameters for the Design of a Cantilever Training Wall

H	A	B	C	D	Bar # Spacing	
					E- Bars	F- Bars
ft	in	ft-in	in	ft-in	# - in	# - in
5	8	1 – 2	8	0 - 6	4 – 21	3 - 18
6	8	1 – 5	8	0 - 8	4 – 21	3 - 18
7	8	1 – 8	8	0 - 10	4 – 16	3 - 18
8	12	1 – 11	8	0 - 9	4 – 20	3 - 18
9	12	2 – 2	8	1 - 0	4 – 13	3 - 18
10	12	2 – 5	8	1 – 2	4 – 10	3 – 12
11	12	2 – 8	8	1 – 5	5 – 10	3 – 12
12	12	2 – 11	8	1 – 8	6 – 12	4 – 14
13	12	3 – 2	10	1 – 11	7 – 12	4 – 12
14	12	3 – 5	10	2 – 2	8 – 12	4 – 10
15	14	3 – 8	12	2 – 3	8 – 12	4 – 12
16	15	3 – 11	12	2 – 4	8 – 11	4 – 10
17	16	4 – 2	12	2 – 6	9 – 13	4 – 9
18	17	4 – 4	12	2 – 7	9 – 11	5 – 10
20	19	4 – 10	12	2 – 11	9 – 10	6 - 11
22	21	5 – 4	12	3 – 3	9 – 8	
24	24	5 – 10	12	3 – 5	9 – 7	6 – 11
26	26	6 – 4	12	3 – 8	9 – 6	6 – 9
28	28	6 – 10	12	4 – 0	9 – 5	6 - 9
30	31	7 – 3	12	4 - 2	9 – 5	

ANNEX D
(informative)

Outlet Works

D.1 Discharge Pipe Sizing

D.1.1 From the resoervoir operation study in Annex A, select a critical demand-head, Q_d –h, combination which is a considerably large discharge under low head.

D.1.2 Compute for the size of a preliminary pipe.

$$d_p = \frac{\sqrt{\frac{4Q_d}{\pi}}}{(2gh)^{1/4}}$$

where:

d_p	is the preliminary pipe diameter (m)
Q_d	is the critical demand (m^3/s)
g	is the gravitational acceleration (m^2/s)
h	is the critical head (m)

D.1.3 Select a trial size, d_t, of pipe larger than the computed d_p.

D.1.4 Compute for the corresponding velocity.

$$V_t = \frac{4Q_d}{(\frac{\pi d_t^2}{4})}$$

where:

V_t	is the flow velocity in the trial pipe, m/s
Q_d	is the critical demand, m^3/s
d_t	is the trial size of pipe, m

D.1.5 Determine the total minor losses.

$$h_m = \frac{{V_t}^2}{2g} \times (K_t + K_e + K_b + K_V)$$

where:

h_m = total minor losses (m)
V_t = flow velocity in the trial pipe (m/s)
g = gravitational acceleration (m^2/s)
K_t = trashrack loss
K_e = entrance loss
K_b = bend loss
K_v = valve loss

D.1.6 Compute for the friction loss.

$$h_f = f \times L \times \frac{{V_t}^2}{2g}$$

$$f = \frac{185n^2}{(d_t)^{1/3}}$$

where:

h_f	is the friction loss (m)
f	is the friction loss coefficient (d_t = trial size of pipe, ft)
L	is the total length of pipe (m)
V_t	is the flow velocity in the trial pipe (m/s)
g	is the gravitational acceleration (m^2/s)

D.1.7 Determine the total head loss, h_t, which is the sum of the total minor loss and friction loss.

D.1.8 Compute for the net head, h_n, which is the difference between the critical head and total head loss.

D.1.9 Compute for the corresponding discharge for the trial size of the pipe.

$$Q_t = \sqrt{2gh_n}\frac{\pi d_t^2}{4}$$

where:

Q_d	is the discharge for the trial size (m^3/s)
g	is the gravitational acceleration (m^2/s)
h_n	is the net head loss (m)
d_t	is the trial size of pipe (m)

D.1.10 If $Q_t \geq Q_d$, use the trial size as the final pipe diameter. Otherwise, assume another trial size and repeat sections D.1.4 to D.7.9.

D.2 Impact Type Dissipator

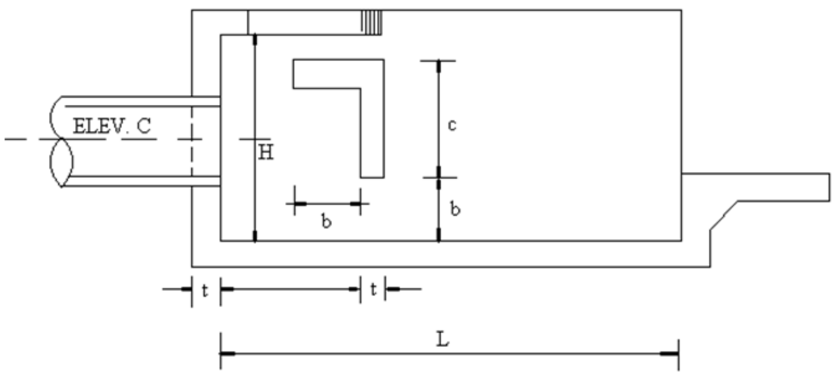

Figure D.1. Impact Stilling Basin

D.2.1 Compute for the hydraulic head.

$$h = NWS - El_{D/S}$$

where:

h is the hydraulic head, m
NWS is the normal water surface elevation, m
$EL_{D/S}$ is the downstream elevation

D.2.2 Determine the equivalent square opening of the discharge pipe diameter, its corresponding velocity and Froude number.

$$d_s = \sqrt{\frac{\pi d^2}{4}}$$
$$V_s = \sqrt{2gh}$$
$$F = V_s\sqrt{gd_s}$$

where:

d_s is the equivalent square opening, m
d is the discharge pipe diameter, m
V_s is the corresponding velocity, m/s
h is the hydraulic head, m
F is the Froude number

D.2.3 Compute for the basin width.

$$W = 2.85 \times d_s \times F \times 0.58$$

where:

W is the basin width, m
d_s is the equivalent square opening, m
F is the Froude number

D.2.4 Determine the other basin dimensions shown in Figure D.1 using the equations below. Note that all units are in meters.

$$H = \tfrac{3}{4}\,W$$
$$a = \tfrac{1}{2}\,W$$
$$b = 1/6\ W$$
$$c = 3/8\ W$$
$$L = 4/3\ W$$

Technical Working Group (TWG) for the Development of Philippine National Standard for Rainwater and Runoff management – Small Water Impounding System

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